NTNU – Trondheim
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STABILITY ASSESSMMENT OF THE UNDERGROUND SETTLING BASIN CAVERNS OF SUPER MADI HYDROELECTRIC PROJECT,NEPAL

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Hydropower Development
Submission date: June 2012
Supervisor: Krishna Kanta Panthi, IGB

Norwegian University of Science and Technology
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Submitted By : Umesh Gautam

Master Thesis

Hydropower Development Program

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for

Umesh Gautam

STABILITY ASSESSMENT OF THE UNDERGROUND SETTLING BASIN CAVERNS OF SUPER MADI HYDROELECTRIC PROJECT, NEPAL

Background

Super Madi Hydroelectric Project located in Kaski District of Nepal is under detail design stage. The project is a run of river scheme with an installed capacity of 44MW, design discharge of 18m^3/sec and net head of 295m. Major underground components of the project are two parallel settling basin caverns, 5.9km long headrace tunnel and 37m high surge shaft. In addition to these several adit tunnels connect the main underground structures.

The project lies in the lower part of the Higher Himalayas and is mainly dominated by high grade metamorphic rocks like gneiss and schist. The proposed underground settling basins are located in steep hillside to the left bank of the Madi River and are relatively close to the surface. Stability of settling basins are one of the main concerns.

MSc Project task

This MSc thesis work is hence related to the stability assessment of the underground settling basin caverns of the project. Stability assessment of the existing layout and possible alternative layout of the settling basins will be of main focus of the study, and shall include:

- Brief description of the project, selected alignment for underground structures, engineering geological conditions along the alignment etc.
- Analysis and evaluation of the input variables needed for stability assessment.
- Description and evaluation on the existing layout plan and orientation design of the settling basin caverns.
- Stability assessment of the settling basin caverns using analytical, empirical and numerical methods.
- Description and evaluation of the alternative layout, orientation and shape of the caverns.
- Stability assessment of the alternative settling basin caverns.

Relevant computer software packages

Candidate shall use *Rocscience package* and other relevant computer software for the master study.

Background information for the study

- Relevant information about the project such as reports, maps, information and data received from the supervisor and co-supervisor.
- Data and information collected by the candidate through various other sources.
- 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 and rock mechanics.

Cooperating partner

Himal Hydro, Nepal is the co-operating partner. **Mr. Pawan Kumar Shrestha** will be the ***co-supervisor*** of this thesis work.

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

PREFACE

This master thesis titled Stability Assessment of The Underground Settling Basin Caverns of Super Madi Hydroelectric Project, Nepal, submitted to the Department of Hydraulics and Environmental Engineering, Norwegian University of Science and Technology, Trondheim, Norway. It is the compulsory task to partial fulfillment of Master of Science in Hydropower Development Study 2010-2012.This study has been carried out at the Department of Geology and Mineral Resources Engineering

Thesis work has been accomplished within the period of January 2012 to June 2012.Major focus of the study are to evaluate the stability of existing layout and possible alternative layout of settling basin. Data and all the information have been provided by the Himal Hydro And General Construction Ltd, Nepal.

This academic work by author, hereby declares that the work presented here is his own and that outside input was given due acknowledgement.

Umesh Gautam

June, 2012

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I would like to express my gratitude to the Department of Hydraulics and Environmental Engineering as well as Department of Geology and Mineral Resource Engineering, Norwegian University of Science and Technology (NTNU), for providing me an opportunity to pursue my Master Degree here in Norway.

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I am thankful to the Himal Hydro and General Construction Ltd. for providing the necessary data and information to complete this work successfully.

My special gratitude to all my friends of Trondheim, Special thanks to my friends from all around the world, who are always encouraging me to study.

I am indebted to my teachers from my childhoods who are still encouraging me to do better on each step of my life.

I am deeply indebted to my parents, family members, and all relatives for their continuous encouragement, blessing, supports, and love.

I would like to dedicate this whole work to my parents, my nation-Nepal, and all Nepali.

Umesh Gautam

June, 2012

ABSTRACT

Super Madi Hydrolectric Project (SMHEP) is located in Kaski District of Nepal. It is a run of river scheme with a installed capacity of 44 MW, net head of 295m and design discharge of 18 m^3/s.It has planned to build for the fulfillment to minimize the load-shedding problem of Nepal in the current scenario. This project lies in the lower part of the Higher Himalaya, mainly dominated by high-grade metamorphic rocks like gneiss and schists.

Major task involve in this thesis work is to check for proper alignment of existing layout, assessment of stability condition with proper support system. Selection of best alternative alignment of cavern with its best shape and size are also another major work in this thesis. Optimum support estimation for the best alternative has also been done. Conclusion and final recommendations are based on stability condition and degree of rock support requirements.

Geological and topographical site condition of headworks restrict for exposed settling basin therefore underground settling basin cavern in the left hill side has been selected. Rock mass in the settling basin area is slightly deformed, foliated micaous and banded gneiss with thin layer of schist. Analysis is based on assumption of ``No significant faults and shear zones across the alignment of settling basin cavern``.

Both alternatives with axis orientation of N145E have been selected for the analysis. Shape of the caverns in both alternatives are inverted D. Existing alternative consists of two parallel settling basin caverns with a clear spacing of 9.5 m. Average width and height for both the caverns of existing alternative (Alternative I) are 8.4 m and 15 m whereas for proposed alternative (Alternative II) are 18.3 m and 20 m are respectively.

Stress-strength factor plays a vital role for overall stability condition of the cavern. Stress induced problems such as rock bursting and spalling in hard rock whereas squeezing in weak rock is assumed. Some Empirical, Analytical, and Numerical approaches have been used for stability assessment and for designing of proper rock support system.

RMR and Q-system of rock mass classification are used to classify the rock masses. Grimstad and Barton (1993) method is used in the analysis of rock bursting problem and squeezing problem. As a Semi-analytical approach ``Hoek and Marionos approach`` has been used for squeezing analysis.

Numerical approaches have many benefits over empirical and analytical approaches, specifically in complex geometry like settling basin cavern. Rocscience software for numerical analysis such as Phase^2 and Un-wedge has been used. Generalized Hoek and Brown failure criterion are used to determine the state of stresses, strength factors, and deformations around the periphery of the caverns in Phase^2 . To analyze the wedge failure due to low shear strength of joints, empirical approach suggested by Barton and Bandis is used in the numerical analysis through rocscience software-Unwedge.

Comparative study of empirical, analytical, and numerical approaches of analysis have been carried out for assessment of stability conditions. Finally, some recommendations to improve the analysis results have been performed.

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CHAPTER 1

1 INTRODUCTION

Nepal is a landlocked developing country having abundant of untapped natural resources. It is situated in between China and India with elevation ranges from 8848 m to 70 m relative to mean sea level. Great elevation difference over a short distance with extreme precipitation along the higher Himalayan zone makes it possible to develop more than 43000 MW hydroelectric powers. It is well known to all the Nepalese people that development of the country is only possible after construction of planned hydropower projects. Although Nepal is a best suitable place for developing the hydropower projects, there are many technical and non-technical challenges. Political instability of the nation is the major issue in current scenario.

In technical context, there are also many challenges that thwart the development of the projects. Extreme climatic conditions (Great temperature and precipitation variation with respect to time and locality) and active geological formation (Due to plate tectonic movements) make it difficult to construct any project within predicted time, cost, and accuracy. Lack of sufficient researches and technical experts in such active Himalayan geology makes it more tangles. In spite of those challenges, many tunneling projects are ongoing, which are more or less successful as well.

1.1 Objective and scope of study

Main objective of the study is to perform stability assessment of the underground settling basin caverns of Super Madi Hydro- Electric Project (SMHEP). Stability assessment of the existing layout and possible alternative layout of the settling basins are the focus. Main objectives of the study are listed as follows:

- Brief description of the project, selected alignment for underground structures, engineering geological conditions along the alignment etc.
- Analysis and evaluation of the input variables needed for stability assessment.
- Description and evaluation of the existing layout plan and orientation design of the settling basin caverns.
- Stability assessment of the settling basin caverns using analytical, empirical, and numerical methods.
- Description and evaluation of the alternative layout, orientation, and shape of the caverns.
- Stability assessment of the alternative settling basin caverns

1.2 Methodology

Following methodologies are used for successfully completion of the task in a chronological basis

- Collection of relevant information about the project such as reports maps, data from supervisor and co-supervisor.
- Literature review related to engineering geology and rock mechanics theory
- Review of scientific paper related to Himalayan geology
- Review of scientific papers books related to international tunneling cases.
- Project review and alternative analysis
- Evaluation of Input parameters needed for the analytical, empirical, and numerical analysis
- Performing the Stability analysis using analytical and empirical approach
- Estimation on rock supports using empirical/semi analytical approach and suggest for numerical approach
- Performing numerical analysis using Phase^2 for stability assessment and estimation of rock support and eventually suggested for wedge failure analysis
- Performing wedge failure analysis using numerical technique – Unwedge analysis
- Comparison between alternatives based on all approaches of analysis
- Conclusion and recommendation for best alternative
- General Comments
  - ✓ Rigorous study of relevant scientific papers, journals, and books along with professor`s suggestions/recommendations are done on continuous basis during study period.

1.3 Back Ground Material

Background information, which are used for successfully achievement of predefined objectives are summarized as follows:

- Feasibility study (Main report) for SMHEP
- Feasibility study report, Volume IIIA
  - ✓ Annex A ; Topographic Survey and Access Road
  - ✓ Annex B ; Geology
- Topographical maps, relevant photos
- Geological map of Nepal
- Relevant scientific Literatures and publications
- Relevant papers, publication through internet using;
  http://scholar.google.no/
  http://www.rocscience.com/

CHAPTER 2

2 PROJECT DESCRIPTION

2.1 General

Super Madi Hydroelectric Project (SMHEP) is identified by Himal Hydro and General Construction Ltd and obtained Survey License for electricity generation from Department of Electricity Development (DoED) of Ministry of Energy, Government of Nepal.

Madi River is a perennial snow fed river originating from Annapurna Himalaya region. It is one of the major tributaries of Gandaki basin, which originates from the base of Annapurna Himal. This project is the simple Run-off scheme that utilizes the flow in Madi River.

Simple boulder type- diversion weir across the Madi River is proposed to divert the water into side intake. A gravel trap settles gravels coming through the six orifices of the intake and takes to the head pond through approach box culvert. Two underground settling basins are fed with the discharge by two inlet tunnels from the head pond. An outlet pond after the settling basins feed water into headrace tunnel. The headrace tunnel has length of 5905m and internal diameter of 3.6 m - 4.4 m. It has steel penstock pipe of average diameter 2.6m and length 1381m, which allow water to the three units of vertical axis Pelton Turbines installed in the semi-surface powerhouse. The water will then flow back into the Madi River through 281m long tailrace canal. An underground surge tank is provisioned at the end of the headrace tunnel. An access tunnel to the headworks, two adit-tunnels to access the underground settling basins, three adit-tunnels to access the headrace tunnel and a ventilation tunnel to reach the top of the underground surge tank are provisioned.

In general, This project has the installed capacity of 44 MW; design discharge of 18 m^3/s ; net head of 295m and net saleable annual energy is 243.125GWh.

The electricity generated from this plant will be connected to the Integrated Nepal Power System (INPS) at the Lekhnath Sub Station of Lekhanath Municipality near Pokhara City through 22km long 132kV single circuit transmission line (Himal Hydro, 2009).

2.2 Location

Super madi Hydroelectric Project (SMHEP) is located at Namarjun and Parche Village Development Committees (VDC) of Kaski district, Nepal. Kaski district is bordered by Lamjung district in the east, Manang in the north, Myagdi in the west and Palpa, Syangja and Tanahu districts in the south. Entire project is located along the left bank of Madi River. Intake site is located at the left bank of the Madi River, at the foothill of sikles village of Parche VDC. Powerhouse is located at Sondha village. Underground headworks component is well located inside the mountain of Tantin village of Namarjun VDC.

The Project is located approximately 23km northeast from Pokhara city. Out of 7 K.m,more than half part is black topped. and the rest is graveled to Bijayapur river. The Road from Lamakhet to Chasu of about (3.5 km long) lies on the left bank of Madi River, which have to upgrade. Gravel road is constructed up to Sodha, opposite of powerhouse. There is no access road up to the headworks area because of steep topography near the headworks area. Therefore, access adit is needed to facilitate for transporting the construction material/equipment up to the intake area. Access road is also necessary to link the existing road networks to the powerhouse. Headwork site is about 2 hours walking distance from Sodha. It can also be accessible by constructing an access road of 5.55 km along the bank of Madi River, but tunnel outlet and surge tank area have to be accessed with new access road of approximately 6.66 km from Chasu (Himal Hydro 2009).

Figure 2-1 : Geographic map of Nepal with project location

Geographic map of Nepal with location of project in a global sense has been presented in Figure 2-1. The Latitude and Longitude of the project area are within 28° 19’ 02" N to 28° 21’ 39" N and 84° 04’ 45"E to 84° 08’34"E.

Figure 2-2 : Close view of Project area in Kaski District (Himal Hydro 2009)

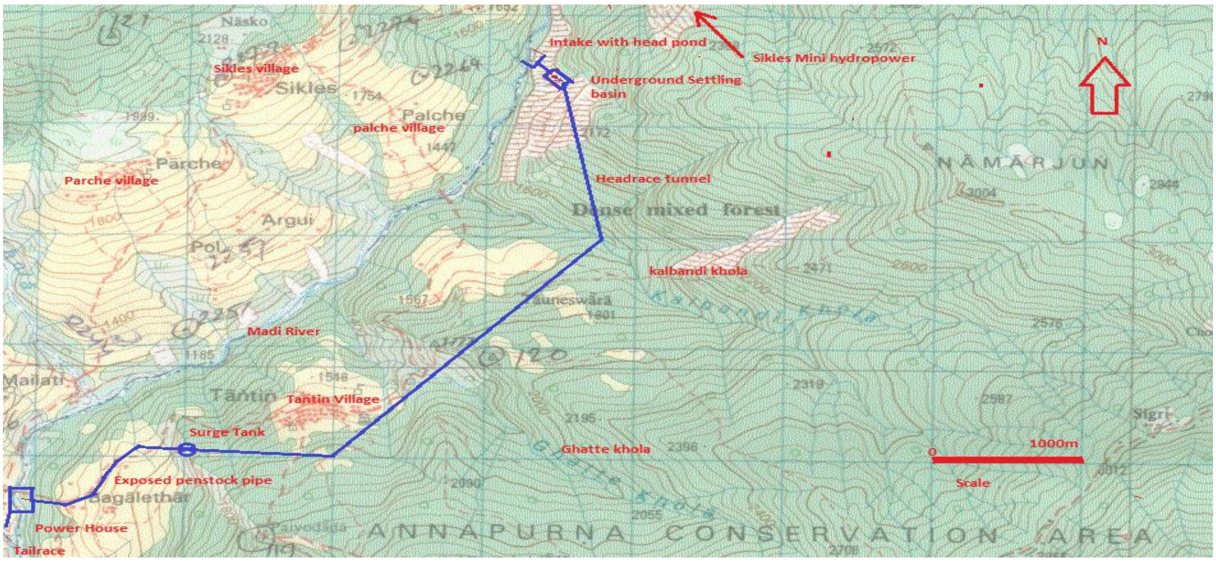


Figure 2-3 General Layout plan of the Supermadi Hydroelectric Project (Himal Hydro 2009)

A close view of project in a Kaski district and General layout plan of the super Madi Hydroelectric Project are shown in Figure 2-2 and 2-3 respectively. The above maps give the general idea. Figure 2-3 is the general layout plan of the project from intake to powerhouse. Name of different villages and locations of hydropower components are also clearly seen.

2.3 Salient Features of the Project:

Table 2-1 : Salient features of Super Madi Hydroelectric Project

General	
Type of Power Plant	Run-off River(ROR)
Gross Head	305m
Power & Energy	
Installed Capacity	44 MW
Average Annual Energy	243.125 Gwh
Hydrology	
Catchment Area	28.1 Km^2
Mean Annual Discharge	30.29 m^3/s
Design Flow	18 m^3/s
Settling Basin	
Inlet pond	31m×12.4m×4.95m
Underground De-sanding Basin	172m×7.5m×14.75m(Parallel Section 160m)
Particle Size to be settled	0.2mm@90%Efficiency
Circular Inlet Shaft to Headrace	H=13m,D=3.6m
Headrace Tunnel	
Inverted D Headrace Tunnel	D=3.6m-4.4m, L=5905m
Surge Tank	
Circular, Underground Type	D=9m, H=37m
Penstock	
Inverted D Type Penstock Tunnel	D=2.8m,L=38m
Exposed Steel Penstock	D=2.6m, L=1381m, Thickness=12mm-32mm
Power House, Turbine and Tailrace Canal	
Semi Surface Power House	L=52m, B=14m, H=28m
Number of Pelton Turbine	3
Efficiency of Turbine (%)	89.50 %
Tailrace Canal	L=160 m, W×H = 4 m × 2.5 m

CHAPTER 3

3 ENGINEERING GEOLOGICAL INVESTIGATION

3.1 Geology of Nepal

Nepal has a complex geological set-up with indeterminable rock mass conditions due to continuous folding, faulting, shearing, and fracturing of rock mass. High degree of anisotropy in strength and deformability are the results after such continuous geological activity. Combined effect of tectonic movement and instantaneous changes of climatic conditions contributed to deep weathering. This increases the permeability, provides access to water to go through discontinuities, and makes the rock mass weaker and weaker.

The Ongoing collision between Indian plate and Eurasian plate is producing the world's highest mountain belt on the earth. Since collision started, several post tectonic events took place to shape the present day Himalayas. This process was started about sixty million years ago. The collision rate is so slow, which is around five centimeter per year (Uprety 1999).One third of total length of this active Himalayan lies in Nepal. Longitudinally, Himalayan range is divided into five tectonic zones (Ganser, 1964).

1. Gangetic plain
2. Sub-Himalayan zone
3. Lesser Himalayan zone
4. Higher Himalayan zone
5. Tibetan-Tethys Himalayan zone

These zones extend from east to west and are almost parallel to each other. All have different lithology, structure, and geological history. Each of the zones is geological bounded by two different thrust planes, which are structurally unstable; hence, it becomes great challenge to all engineers to develop any infrastructure within these regions. Different geologic zone with the location of Super Madi Hydroelectric Project is shown in Figure 3.1

Gangetic Plain:

This zone is also called the terrain zone. It extends from Indian shield in the south to the sub-Himalayan (siwalik) to the north. The plain is in less than 200 meter above the sea level and usually has average thickness of 1500-meter alluvial sediments. The Northern boundary of this zone is active thrust system which is called Main Frontal thrust (MFT) with siwalik.

Sub-Himalayan Zone:

It lies between Lesser Himalayan Zone in the north and Terai zone in the south. Tectonically active thrust plane (MBT) is in north whereas MCT in the south. This Zone is extended all along the Himalaya forming the southernmost hill range with width of 8 to 50 km. The general dip of beds of Siwalik has northward trend with varying angles and the overall strike is east west. This zone is further divided into 3 sub zones:

Lower siwalik:

It consists of irregularly laminated beds of fine-grained greenish sandstone and siltstone with mudstone. The alternating mudstone beds are thickly bedded and are red, purple, and brown color.

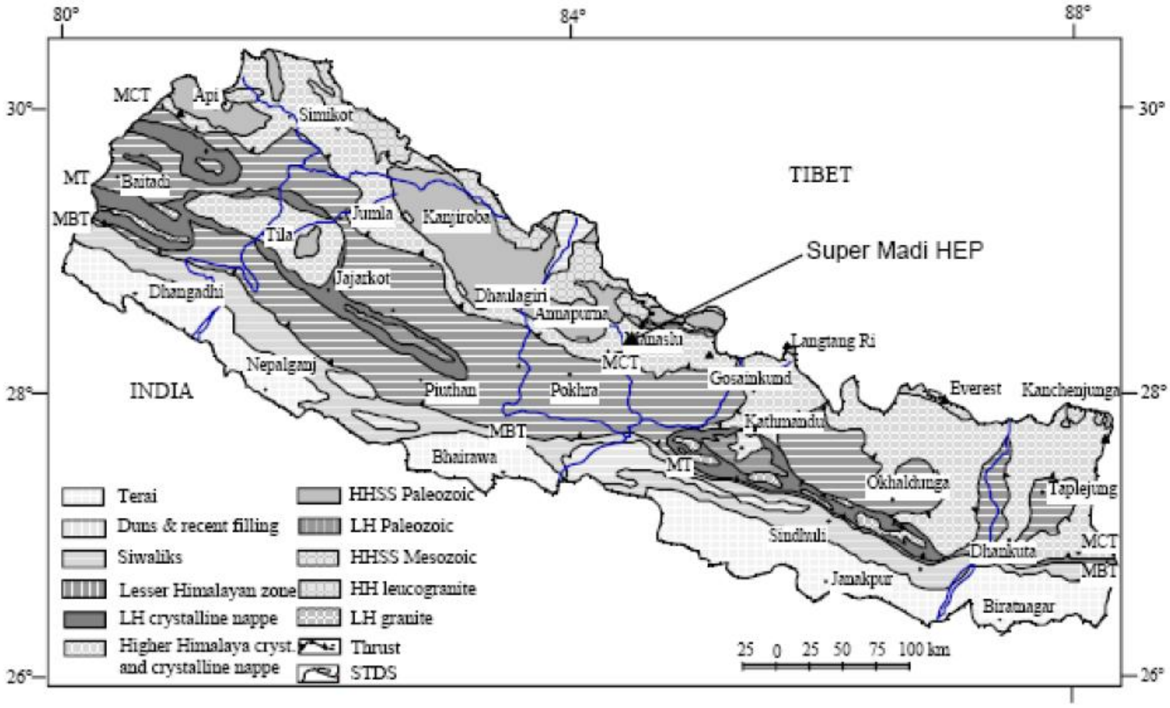


Figure 3-1 Geological division of Nepal indicating the location of project area (HH,2009)

Middle Siwalik:

It comprised of medium to coarse-grained sandstones inter-bedded with mudstone. In upper part of the Middle Siwalik, pebbly sandstone beds are also found. Thickness of sandstone beds are ranges from 1 m to 45 m.

Upper Siwalik:

It comprises of conglomerate, sand, silt beds, boulder and cobble size rounded to sub angular fragments. Mudstone beds of the Upper Siwalik are massive, irregular and contain many invertebrate fossils.

Higher Himalayan Zone:

It consists of layers of strongly metamorphosed rocks. It is separated with Tibetan-Tethys Zone by normal fault system called as South Tibetan Detachment System. It consists of an approximately 10 km thick strongly metamorphosed coarse-grained rocks. It extends continuously along the entire length of the country from east to west with varying .schist, and marble. Granites are also found in the upper part of this zone.

The SMHEP is located in the lower part of the higher Himalayan zone where banded gneiss is dominant rock type in the headwork area.

Tibetan-Tethys Zone:

It is located in northern part of the country. Most of the Great Himalayan peaks of Nepal (Manaslu, Annapurna, and Dhaulagiri) have rocks of Tibetan-Tethys Zone. This zone is composed of sedimentary rocks, such as shale, limestone, and sandstone. It begins from the top of the South Tibetan Detachment System (STDS) and extends to the north

3.2 Challenges for engineering structures in Nepalese geological context

Due to tectonic movement of continental plate, young mountains are forming continuously. These young mountain ranges are stretched approximately in east-west direction. Sheared planes are formed extensively, which decreases the rock mass strength and changes the rock stress conditions. Continuous and unpredictable changes in the quality of rock mass make it difficult to design and to construct any type of engineering structures in this region.

Rock mass in the Himalaya is relatively weak and highly deformed, schistose, weathered and altered. It is due to active tectonic movement and dynamic monsoon. Predicting the rock mass quality, analyzing stress induced problems (particularly tunnel squeezing) and predicting the inflow and leakage often have been found extremely difficult during planning phase investigation. This leads to some discrepancies between predicted and actual case and have consequences on time and cost overrun of the project (Panthi 2006).

During construction period, probing ahead of the tunnel face using geophysical means like topographic analysis and radar is not suitable in Himalayan zone (Goel et al.1995a).Therefore it is recommended to investigate in detail, prior to starting of any project.

Lack of sufficient research, experienced technical expertise, and selection of suitable technology leads to create more discrepancy between the predicted and actual condition. Very high discrepancy between pre construction and post construction works in four major projects (Khimti ,Kaligandaki A, Modi Khola and Middle Marsyangdi) of Nepal are clearly indicating the level of accuracy in pre construction phase investigation (Panthi 2006), which was leaded to extreme overrun of cost and time.

3.3 Geological setting of the project

Super Madi Hydro-Electric Project lies in the southern part of Higher Himalayan Zone, just above the active main central thrust (MCT) as shown in Figure 3.1. Project area has medium to high-grade metamorphic rocks such as silimanite gneiss, kayanite schist and gneiss, augen and banded gneiss. Headworks, the settling basins, are located inside the massive rock mass, which have slightly deformed foliated micacious and banded gneiss with thin layer of schist. A brief description of formation of Higher Himalaya crystalline rock is presented in Table 3.1

Folding and shearing are quite significant along the surface slope. Many old slides can be observed along the right hill, surface slope indicating active slope movement in the past. All these slides were due to shear failure along the topographic slope. Left hillside is much better than right hillside due to less deformation and shearing, except at kalbandi and Ghatte khola region. Some weakness zones have also been found in the left side of tunnel and penstock alignment, but insignificant with compare to right bank.

Table 3-1 : Lithostratigraphy of the Higher Himalaya (Himal Hydro, 2009)

Zone	Formation	Litho logy	Thickness	Age
Tibetan-Tethys Himalaya	Annapurna Yellow Formation	Impure Limestone, Marble and Schist	3000m	Cambrian
South Tibetan Detachment System(STDS)				
Higher Himalaya	Formation III	Augen Gneiss	3,000m	Pre-Cambrian
	Formation II	Banded Gneiss and Quartize	1,700m	
	Formation I	Schist and Gneiss	5,000m	
Main Central Thrust(MCT)				
Lesser Himalayan				

Dominating joint set in the project area have foliation-having strike of N85E to N100E and dip ranging from 40NE to 45NE. The foliation joints are relatively sheared, folded and smooth undulating in character. Furthermore, one set of almost vertical prominent cross joint and few random joints could be seen along the rock exposure of dam site. Vertical joints have strike of N120E to N130E with dip of 75SW. Cross-joints are rougher and open in character with compare to foliation joint.

3.3.1 Headwork site

Diversion Weir &Side Intake

The proposed weir location is at the elevation of 1362 Masl and the width of river width is about 50m with both side steep rock slope of approximately 70 degree. Both banks of diversion weir is dominated with banded gneiss and thin layer of schist intercalated with weak thin bands of micaceous gneiss. Rock in both banks is moderately weathered, massive to medium foliated with medium to high persistency having three sets of planner to undulating joints filled with silt.RMR and Q value of the exposed rock is about 75 to 80 and 10 to 14 respectively. Rock mass categorizes in Class II and of good quality (according to Rock Mass Classification system).

Figure 3-2 shows the location of intake with slightly weathered massive rock mass having steep slope and looks better place for intake, while considering the hydraulics of river.

Figure 3-2 : Location, Topography, Geology, and Geomorphology at Intake Area (HH 2009).

<u>Gravel Trap, Approach Canal, and Head pond:</u>

Components have been proposed in the recent deposit of alluvial, which consists of rounded to sub rounded boulder, cobble, and pebble. The bedrock is in shallow depth in this location and is approximately 5m to 30m (Himal Hydro 2009).

<u>Underground Settling Basin with Connecting Arrangements:</u>

Geology and topography of site restrict to construct the surface settling basin and other headwork components. Therefore, underground structures in the left bank of river have been proposed. Rock mass is slightly weathered, massive to medium foliated having three sets of rough and irregular, undulating, tight to moderately open joints with medium to high persistency filled with silt.RMR and Q-value of rock masses in this region are 75-80 and 10 to 14.1 respectively, which shows that the rock mass is of good quality.

Figure 3-3 : Location for Underground Settling Basin inside the massive rock mass (HH, 2009)

The alignment of whole project along with underground Cavern (Settling basin) have been fixed based on stability analysis of rock slope of three discontinuities and internal friction angle during feasibility stage of investigation. Orientations of three different joint sets and settling basin alignments have presented in Table 3-2 and Figure 3-4 respectively.

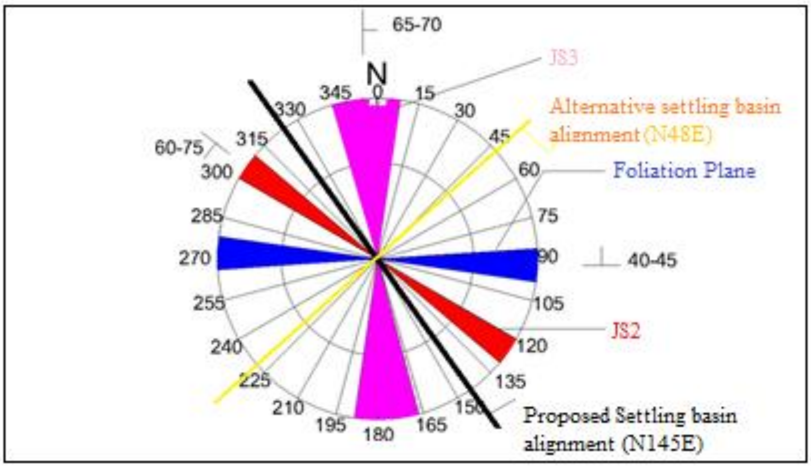


Figure 3-4: Joint rosette of major joint sets with alignment of settling basin cavern

Table 3-2 : Orientations of the three different joint sets in settling basin area (HH, 2009)

Joint	Strike	Dip Direction	Dip Amount	Remarks
Joint set 1(JS1)	N85E-N100E	175-190	(40-45)NE	Bedding/Foliation
Joint set 2(JS2)	N120E-N130E	210-220	(60-75)SW	JS2
Joint set (JS3)	N160E-N190E	100-110	(65-70)NE	JS3

This analysis has done by using lower hemisphere projection in Schmidt`s equal area net. The alignment of settling basin cavern is checked using the joint rosette concept, which is shown in Figure 3-4. Best possible alignments for the settling basin are shown in Figure 3-4 with yellow and black line. The black alignment has axis orientation of N145E,which is existing alignment proposed by Himal Hydro,2009 Another possible alignment, which have axis orientation of N48E is also taken for the possible analysis, if the first alignment is proven to be unstable and nonrealistic. The black alignment (N145E) is analyzed first because of destination direction of powerhouse. Detail Stability analysis of settling basin is done through Analytical, Empirical and Numerical approach in consequent chapters.

3.3.2 Headrace Tunnel:

Due to favorable (Better then right bank) geological feature of left bank of the Madi River, whole alignment of Headrace tunnel is kept in left bank. Banded gneiss, Garnetiferous schist and micaceous gneiss is exposed as the bed rock all the way from headworks to outlet portal of headrace tunnel. The Tunnel alignment has to cross two small streams called Kalbandi khola and Ghatte khola. ERT result at Kalbandi khola showed that the deeper part of the area seems to be predominated by shear zones. There is no sound bedrock below this river. Ghatte khola is flowing through the non-disturbed exposed bedrock of gneiss. Tantin village area and downstream to this area is covered by colluvial deposit.ERT result at the Tantin village area shows the rock is in good condition. In general, tunnel alignment is passed through predominantly fair to good quality of rock masses at the beginning section. Poor to extremely poor rock mass are also found in several locations.RMR and Q –value of rock mass at different loacation along the headrace tunnel is presented in Appendix C.

3.3.3 Surge Tank:

The surge tank is located in the left hillside near Bagalephet. Rock type of this area is mica gneiss and garnetiferous schist, which is slightly to moderately weathered and have three joint sets. Rock mass quality is fair to good. RMR and Q value of rock mass in this region are 60-65 and 2.29-4 respectively.

3.3.4 Penstock Alignment:

The penstock alignment is started with flat topography and the topography becomes steeper as the powerhouse approaches. Rock mass is poor with colluviums deposit. ERT result predicts that there are several sheared rock mass along the penstock alignment. Rock mass quality indices such as RMR and Q –value are presented in Appendix C.

3.3.5 Powerhouse:

The surface powerhouse is located above the recent thick alluvium deposits. Deposit is mainly consists of matrix form of boulder, cobble, pebble, gravels, sand and silt. The powerhouse is well located in a distance of 100m towards the hill toe. There is natural protection guide of bedrock exposure immediately upstream of the powerhouse. Ground water table is about 10m below the lower part of the powerhouse area.ERT result shows that there is thick accumulation of alluvial above the sheared rock mass Rock mass quality indices such as RMR and Q –value are presented in Appendix C.

3.4 Seismicity

Nepal is located in a geographic region prone to earthquake. Loss of lives and property due to unpredictable earthquake is the regular phenomenon. Active tectonic and geomorphic processes, young and fragile geology, variable climatic conditions, unplanned settlement, increasing population, weak economic status, and low literacy rate are the major factors for huge losses due to earthquake. Major earthquakes during 1833,1934,1980,1988 were leaded to debacle condition in Nepal. There is an urgent need to redress the proactive policies related to the seismic hazard. These natural phenomena cannot be stopped but technically, we can minimize the effect of such disaster by making appropriate engineering structures.

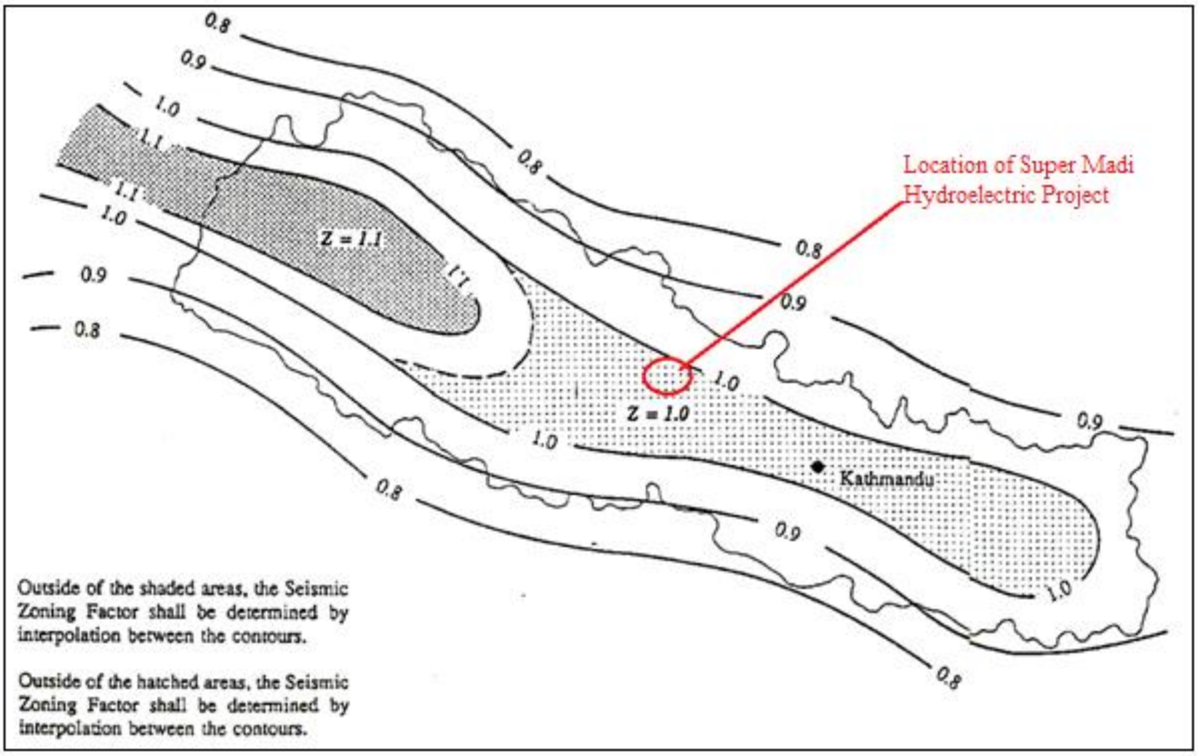


Figure 3-5: Seismic hazard map of Nepal (ASIAN Ministerial conference on disaster risk reduction, 2009, India)

Geologically, Nepal is composed of 92 individual active fault segments with three major fault systems. Nepal is divided into 3 seismic zones from north to south, which are ranges from 0.8-1.1 seismic zoning factor. SMHEP is located in the region with zoning factor of one as shown in Figure 3-5.

Seismic zone factor (Z) corresponds numerically to the effective horizontal peak bedrock acceleration (or equivalent velocity) that is estimated as a component of the design base, shear calculation. For example, Z value one corresponds to the effective peak bedrock acceleration of one times the force of gravity. Since Super Madi Hydro-Electric Project is located in the area, having Z factor 1, it is much more vulnerable. Therefore, we have to aware before /during and after the excavation, .it is highly recommended to incorporate the effect of earthquake in this zone. The effect of earthquake is considered while doing wedge failure analysis through numerical modeling in Chapter 7.

3.5 Field Investigation

After Desk study, Field visit and investigations is carried out by a team of experts. During the field visit, discontinuity survey and geological mapping of the project area including headworks, tunnel alignment, adits, surge shaft, penstock alignment, and powerhouse area have been done. Instability, mass wasting studies and necessary geological data are collected. As a field investigation, surface & subsurface investigation are carried out. After observation and recording of data, different analysis have done which includes slope instability analysis, graphical analysis and rock mass classification for predicting the strength of rock mass and rock support requirement.

3.5.1 Surface mapping

Geological site survey regarding the rock mass type and its location, joint conditions and its orientation were measured by the experienced well-organized team of ``Himal Hydro and General Construction Ltd``. Parameters that were measured at different locations along the tunnel alignment are Joint set number (J_n), Orientation of joints and bedrock, Joint roughness number (J_r), Joint water reduction (J_w), Joint alteration number (J_a), Major fault and fold, Stress reduction factor (SRF), J_v (Joint volume).

Rock mass quality indices such as RMR & Q-Value are evaluated using those parameters. Based on these indices, design method, construction technique and rock support system are determined. Results after the surface mapping are selection of best alignment of the cavern based of prevailing joints, geological map of the project area and rock quality indices, which are shown in Figure 3.4 and Appendix A respectively. Best alignment of headworks, tunnel alignment, powerhouse, surge tank were determined using ``joint rosette concept`` Stability analysis during feasibility study was done by using the lower hemisphere projection in Schmidt`s equal area net.

Based on this investigation, Rock quality indices for Settling Basin area are found as Q-value of 10-14 and RMR of 80-85, which signifies that the rock is of good quality and belongs to Rock class II. Stereographic graphic projection helps to fix the proper cavern alignment. The alignment of cavern is selected as 145 NE. Further sub-surface investigation is recommended after completion of the surface investigation and is discussed in the following section.

3.5.2 Subsurface investigation

2D-Electrical Resistivity Tomography

2D-Electrical Resistivity Tomography is one of sub-surface methods for the assessment of underground conditions. This assessment method was carried out in some critical areas of headwork, part of headrace tunnel, headrace tunnel portal, and penstock and powerhouse area.

Different response to the flow of electric current by different layers and bodies in the subsurface is the basis for electrical resistivity method of geological exploration. By virtue of different materials to conduct electricity, it is possible to separate different materials from each other. Electrical resistivity for a material depends upon the mineralogical composition, matrix (rock and /or granular) and salinity of water and degree of saturation of the pore space.

Total surface length of profile was 3713m. Twelve different 2D-ERT profiles (ERT-1 to ERT-12) were investigated in this project. Among 12 profile data, few but relatively more relevant in relation to headworks area are presented in Table 3.3

Table 3-3 : 2D-ERT Surveys Data for Headworks area (Himal Hydro, 2009)

Profile No	Location	Length (m)	Median depth (m)	Starting Point		End Point	
				Easting	Northing	Easting	Northing
ERT-1	Headworks	175	44	512278	3137673	512259	3137501
ERT-2	Headworks	126	44	512301	3137746	512311	3137621
ERT-3	Headworks	196	44	512342	3137796	512261	3137618
ERT-4	Headrace Tunnel	510	139	513143	3135556	512749	3135420
ERT-5	Headrace Tunnel	340	139	510042	3133980	510355	3134017
	(Outlet area)						

Sample of model resistivity with topography in left bank of headworks area (ERT III) has been shown in Appendix A (Upper one) whereas lower one shows the Interpretative cross-sections, with condition of bedrocks and other materials that surrounds the bedrock. From the ERT result of headworks area, Bedrock is found very close to surface, which indicates that the river is not the part of weakness zone. The rock mass around the headworks area are of good quality, not having any threatened weakness zone (.Panthi 2012).

Project area is located in Higher Himalayan Zone (mainly formed by schist and gneiss).In general. Generally, Unaltered metamorphic rock mass have very low porosity (usually 0.1% to 3% and rarely reaches up to 10%) and very few pores are interconnected to each other. Altered metamorphic rocks such as schist and gneiss have fine capillary tubes. They have high moisture holding capacity even a more than parent sedimentary rocks. Pores inside the rock mass -either in sheared zone or other weakness zone or young sedimentary or highly altered metamorphic rock- are filled by either air or water or any gouge material or combination of them. The electrical resistivity of rock mass is determined by the type and condition of those filling materials. The mineralogical composition and texture have also influenced to determine the electrical resistivity. Reliability and accuracy of manual interpretation as above are less and the time consuming.

3.5.3 Rock Mass Classification

To predict the rock mass quality in different locations of the project area, different rock mass classification systems have been used. Most popular systems are RMR, Q-value. To compute these indicators, input parameter related to rock mass system were taken during surface investigation. Calculated parameters for different chainage along the alignment of project are summarized in Table 3-4.

Table 3-4 : Estimated rock mass quality indices along possible settling basin area (HH, 2009)

Chainage	Q-Value	RMR Value	Rock Mass Quality and Support class	Description of the rock mass
0+000 - 0+635	10-14	75-80	Good rock mass (class II) Rock Support Class I	Fresh to slightly weathered, massive to medium foliated banded gneiss, gneiss and with occasional very thin bands of schist
0+635 - 0+686	4.8	70	Fair to good rock mass (Class III) Rock Support Class II	Slightly to moderately weathered, medium foliated banded gneiss with occasional thin bands of schist. Present of Possible water bearing zone.
0+686 - 0+1300	10-14	75-80	Good rock mass (class II) Rock Support Class I	Fresh to slightly weathered, massive to medium foliated banded gneiss with occasional very thin band of schist.

In conclusion, different surface and sub surface investigation were carried out during investigation period. From the surface investigation, results are the rock mass quality indices (Q –Value and RMR value) of rock masses in different location from intake to powerhouse area. Geological map of project area and suitable (Best) orientation of cavern alignment are possible to create by using measured strike and dip of different joint sets and faults at different chainage. Type of rock and possible weak zone were verified which was predicted during desk study period. As a sub-surface investigation method, 2-D ERT (Electrical Resistivity Tomography) technique is used to identify the rock quality up to certain depth from the surface .In headworks area, bedrock is found very close to surface, which indicates that the river is not the part of weakness zone. Result shows, rock mass on the left side of river is of good quality compare to right bank.

CHAPTER 4

4 ENGINEERING GEOLOGICAL CONDITION OF SETTLING BASIN CAVERN

Rock mass of the project area consists of slightly deformed, foliated micacious and banded gneiss with thin layer of schist. Since the rock mass is massive with sufficient overburden, construction of underground settling basin seems technically feasible but analysis on optimum location is needed and have been done in consequent chapters.

There are three major joint sets including foliation joint set JS1, JS2 and JS3. Joint rosettes of those joint sets are presented in Figure 3. Joints are irregular, undulating, tight to moderately open with medium to high persistency filled with silt. There are not any distinct weakness zones within the approximate section (0+000 to 0+1500 chainage). Because of its location (very close to MCT) and high competency of rock mass, more chances of high tectonic stress in this region. Best alignment is chosen based on analysis through stereographic projection and joint rosette. Ex-foliation joint due to stress anisotropic near the valley side restrict to make the settling basin very close to the surface.

Sufficient rock cover, topographic effect, stress anisotropic, type of rock mass, ground water condition, weakness zone, shape, size and spacing of caverns, construction facility, problems during operation and maintenance of settling basin, safety factor and cost are the major parameters that should take into account during selecting the location and type of settling basin.

RMR & Q-value of rock mass in the settling basin area are evaluated 80-85 and 10-14 respectively throughout the possible location of settling basin. There is no any distinct weakness zone within the chainage of 0+000 m to 0+1500 m. Ground water table is assumed to locate well below and very less chance of seepage from the surrounding of cavern. The range of rock cover for safely locating the underground settling basin is vary from 64.43m to 729.43 m within 866.5 m length of space, which is more than enough for placing a 200 m long settling basin cavern.

4.1 Existing Layout Plan and profile (Alternative I)

The Existing alignment as per feasibility report of Super Madi Hydro-Electric Project is shown in Figure 4.1.Two different caverns for two basins have been proposed at feasibility stage. Spacing between the caverns is 9.5m, which are connected by two connecting tunnel - One flushing tunnel and the other are two-adit tunnel. Orientation of cavern is fixed at 145NE based on analysis through stereographic projection of joint. The designed dimensions such as shape size location were designed based on hydraulic conditions. Geological assessments such as stability analysis in different possible location along the pre-specified alignment have been done. Different empirical and analytical approaches were used for stability assessment. It was suggested to do numerical analysis for reliability and for high accuracy. Existing, layout plan and profile of the settling basin area are presented in Figure 4.1. Cross-sectional geometry of the cavern at three different sections is kept in Appendix C.

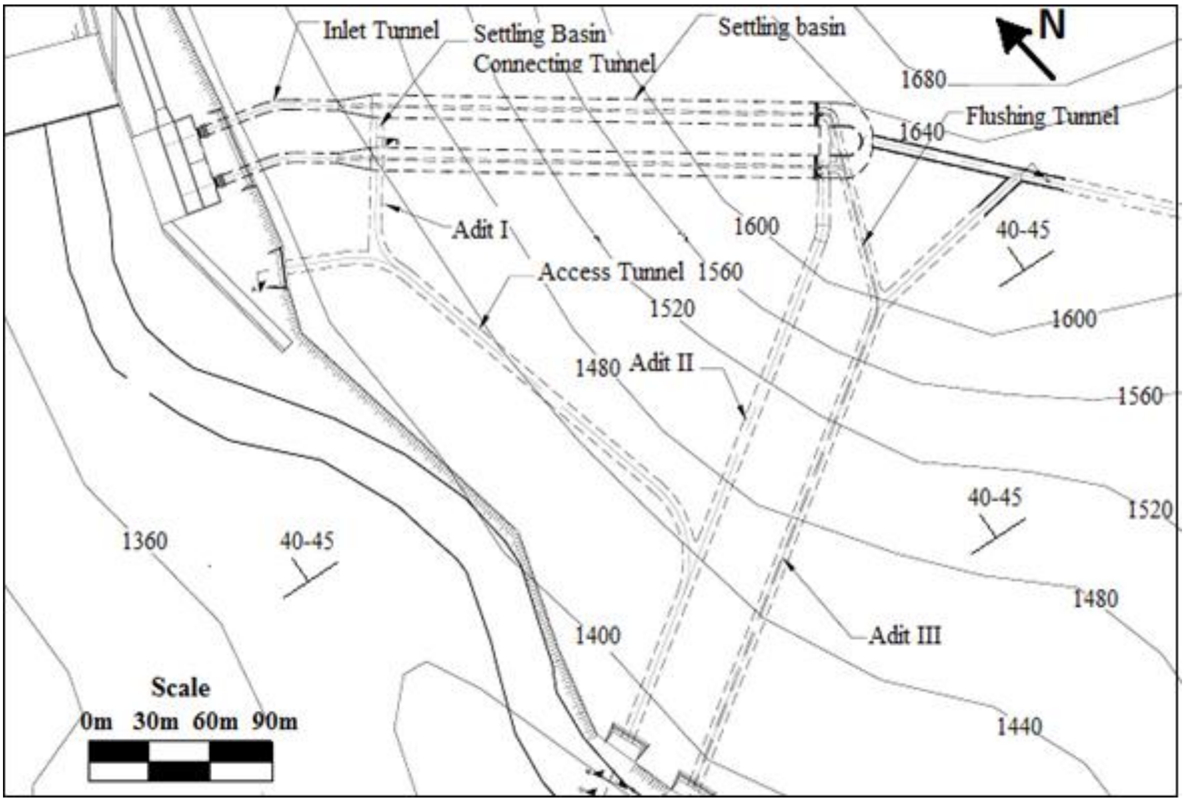


Figure 4-1 : Existing layout plan of settling basin caverns; Alternative I (HH 2009)

4.2 Proposed layout Plan and profile (Alternative II)

The alignment of settling basin cavern has kept same as that of existing one, which is selected based on analysis of orientation of joints sets (Joint rosette analysis). There are also other possible alignments but considering other parameters such as appropriate location of dam axis, other intake structures, length of adit tunnel, overburden condition, this alignment is assumed as best in this stage. The alignment of the caverns may change after stability analysis, if not stable and not feasible. Another possible alignment has shown in Figure 3.4. Pre-defined spacing between the two caverns that is 9.5 m may be technically unstable during drilling and blasting and with consideration for installation of 4m long rock bolt from both sides of cavern.

The proposed alternative is analyzed with two settling chamber in the single cavern in the same location as that of existing settling basin. Spacing between the two settling chambers is made 1.5m, which will be made of concrete after the excavation during construction of settling basin. Outlet arrangement (at Inlet of Headrace tunnel) has also been slightly changed maintaining the constant hydraulic efficiency in both alternatives. Two different inlet tunnels have been incorporated until the expansion area of settling basin. After start of expansion, both tunnels merge into one cavern.

In brief, existing (Alternative I) and proposed (Alternative II) settling basin cavern are similar except the cross-sectional geometry of the cavern. Figure 4.2 and 4.3 are the plan layout and longitudinal profile of proposed settling basin cavern. Cross-sectional geometry of the cavern at three different sections is kept in Appendix C.

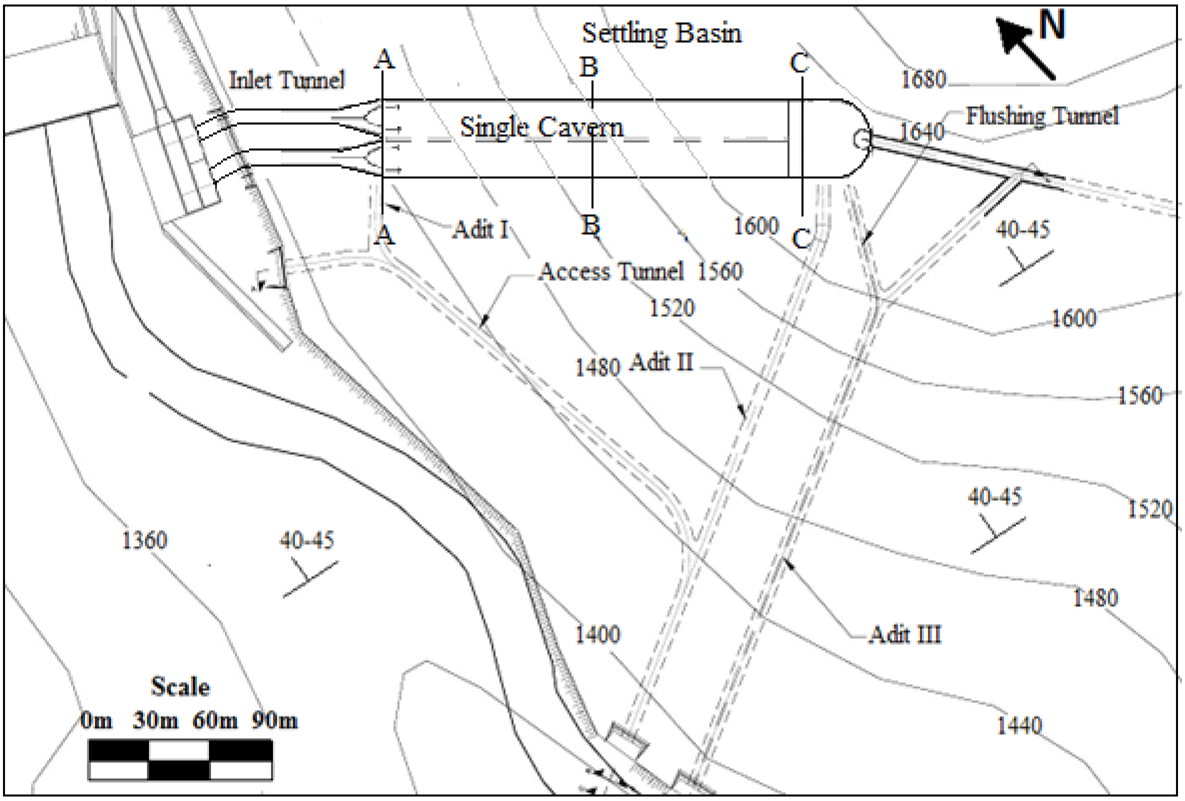


Figure 4-2 : Proposed layout plan of settling basin cavern (Alternative II)

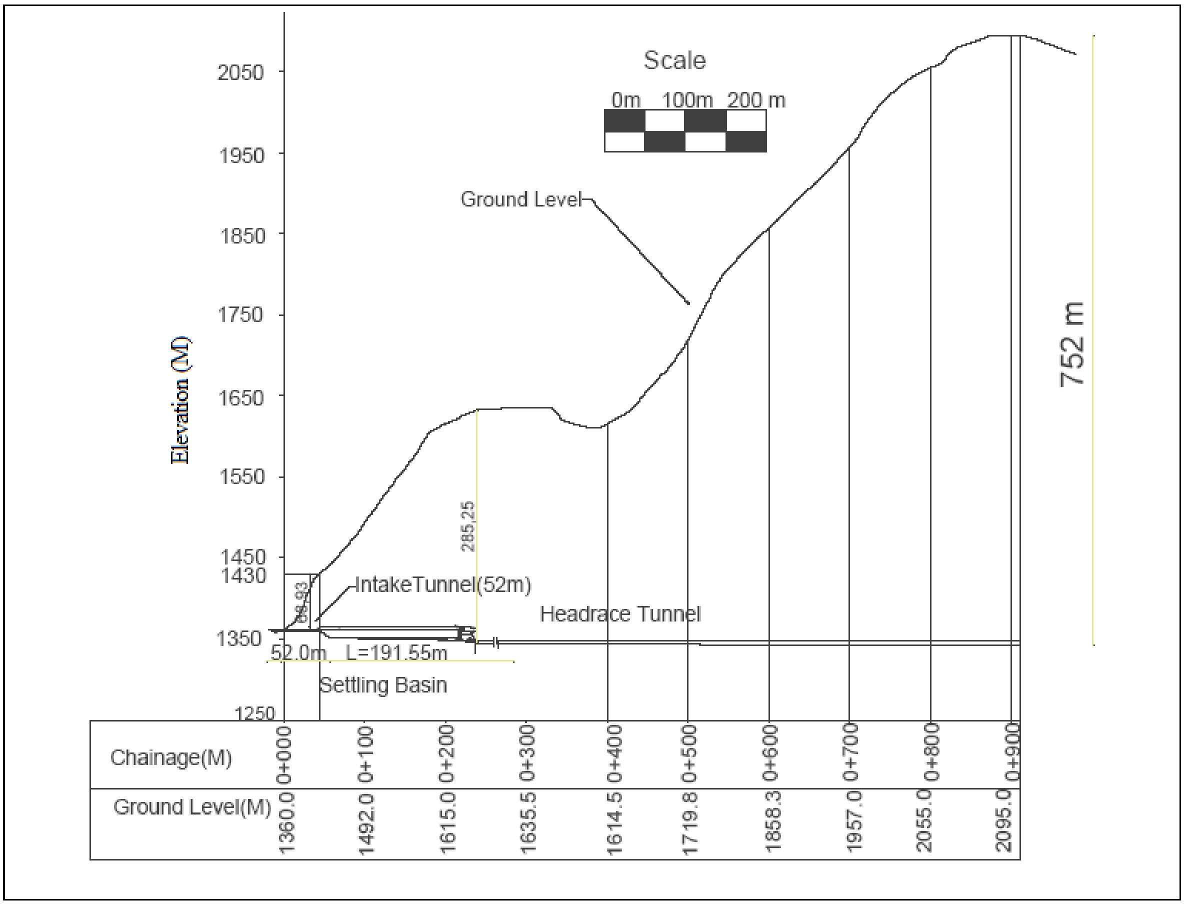


Figure 4-3 : Proposed longitudinal profile of settling basin cavern (Alternative I &II)

4.3 Design Approach of Underground Openings

Designing of any underground opening in a safe and economic way are the major issues for all engineering geologist. Selection of proper alignment, based on analysis of joint sets orientation is the first step in design phase. Consideration of stability problems and over break are the major concern. To analyze these problems, it should know the rock mass strength &local stress conditions. Shape and size of underground cavern are defined based on stability analysis, cost effectiveness, purpose of cavern and safety requirement. For analyzing and designing of caverns, analytical approach is used for simple and uniform loading caverns. Empirical approach in an initial stage with numerical approach for verification is the best way and commonly used technique.

4.3.1 Site Selection

Site should be within desired quality rock mass area. It is necessary to avoid rock type of particularly unfavorable character such as having weakness zones(faults, gouges, crushing zones),highly stressed zones, water seeping zones which may create the stability problems during and after construction and may lead to unexpected cost and time overrun.

Minimum rock cover is needed for developing the self-supporting strength. It should be well enough to establish the normal stresses on joints and fissures. In hard rock a layer of approximately 5m overburden for spans of up to 20 m is sufficient, when the thickness is measured from theoretical maximum over break above the roof line (See Figure 4.4; left in below). For an underground opening that may be regarded as deep-seated should have sufficiently de-stressed in order to avoid or to reduce the rock stress problems such as rock bursting and squeezing. De-stressed area is found outside faults, gouges or crushed zones with strike parallel to the valley and dip steeply towards the valley as shown in Figure 4.4.

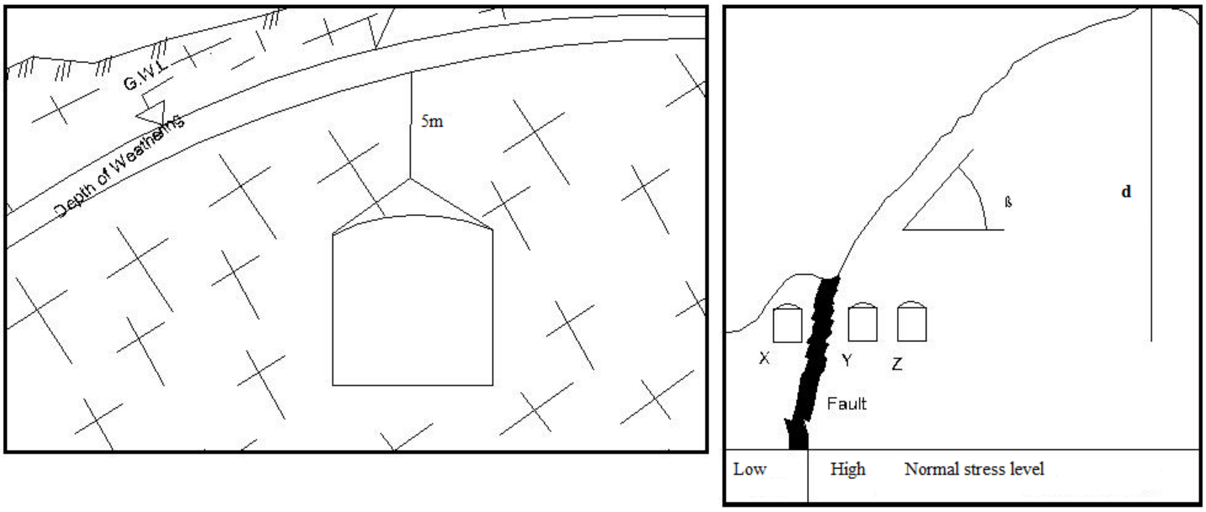


Figure 4-4 : Stress situation for a underground cavern in steep valley with fault zone (right) and minimum rock cover for shallow seated underground openings (left), (Selmer-Olsen and Broch, 1977)

4.3.2 Orientation of Length Axis:

Orientation of cavern is determined by the orientation of predominant joint sets. All possible weakness zones have to be avoided in or near the tunnel alignment. For shallow and intermediate depth, orientation should be along the bisection line of maximum intersection angle between the two predominant joint directions. Close parallelism to the third joint set should be avoided. Character of joint sets, filling material in between the joints, friction properties, joint volume, dip of the joint have also major role for stability.

In case of high anisotropic rock stress condition, tunnel contour are tangential to the plane through the major and intermediate principle stress, which are primarily exposed to rock stress problems (Rock bursting and spalling) as shown in Figure 4.5.

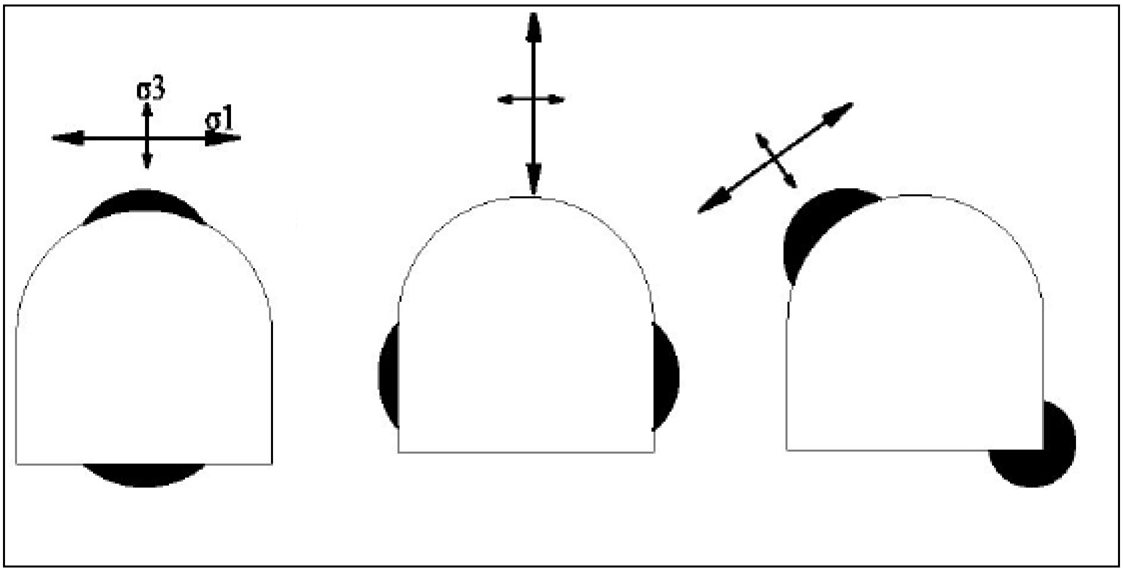


Figure 4-5 : Direction of major and minor principle stresses determining the nature of rock bursting problem (Selmer-Olsen and Broch, 1997

In the Figure 4-5, the black area indicate the location of stress related problem (Rock spalling and rock bursting).Flakes of rock piece is popped out in parallel to the direction of major principle stress. Stress concentration is more in the sharp corner than in smooth curve and have more chance of stress related problem in corners. Therefore, it is highly recommended to minimize the sharp corners in periphery of the cavern.

Most stable orientation is obtained when the length axis of the underground opening makes an angle of 15°-30° to the horizontal projection of the major principle stress. Any parallelism with the foliation or other important joint set should be avoided. If the direction of principle stress is close to the direction of bedding of foliation planes in highly anisotropic rocks such as crystalline schist and flagstones then length axis of the opening is oriented with an angle relative to the strike of the foliation plane. Length axis of the opening should be oriented with the maximum angle 5° with respect to the strike of the foliation plane and 35°should consider as an absolute minimum (Selmer-Olsen and Broch, 1977).

After construction of joint rosette diagram for the settling basin area, best axis orientation of the settling basin cavern is chosen which has shown in Figure 3.4. Two possibly best alignments are taken into consideration for the analysis (Based on the criterion described in the above paragraph).Orientation of first alternative alignment is N145E whereas the orientation of second possible alternative alignment is N48E. The second alternative alignment looks like better than the first one but this direction is not suitable while considering the location of powerhouse (where we need to direct). Therefore, it is recommended to analyze the first (N145E) option. Stability analysis has been carried out to know the stability condition of caverns, which have axis orientation of N145E.Since there is not any stability problem in this alignment of cavern, It is not necessary to analyze the alternative alignment (N48E).

4.3.3 Shape of the Settling basin cavern

Rock mass behaves weak in tensile stress condition because of its discontinuous character. It must be evenly distribute all the compressive stress along the whole periphery of the opening. This uniformity could be achieved by providing the arched roof sharp corner should be avoided to reduce the over break due to distressed in that corners and edge. Arched roof and avoidance of intruding corners are better solutions. Caverns with shallow depth with horizontal bedded rocks, flat roof is possible but for bed thickness of lesser than 0.5m, high arch is better instead of flat roof. For spans higher than 10m, rock bolts should be provided at the sides of the roof.

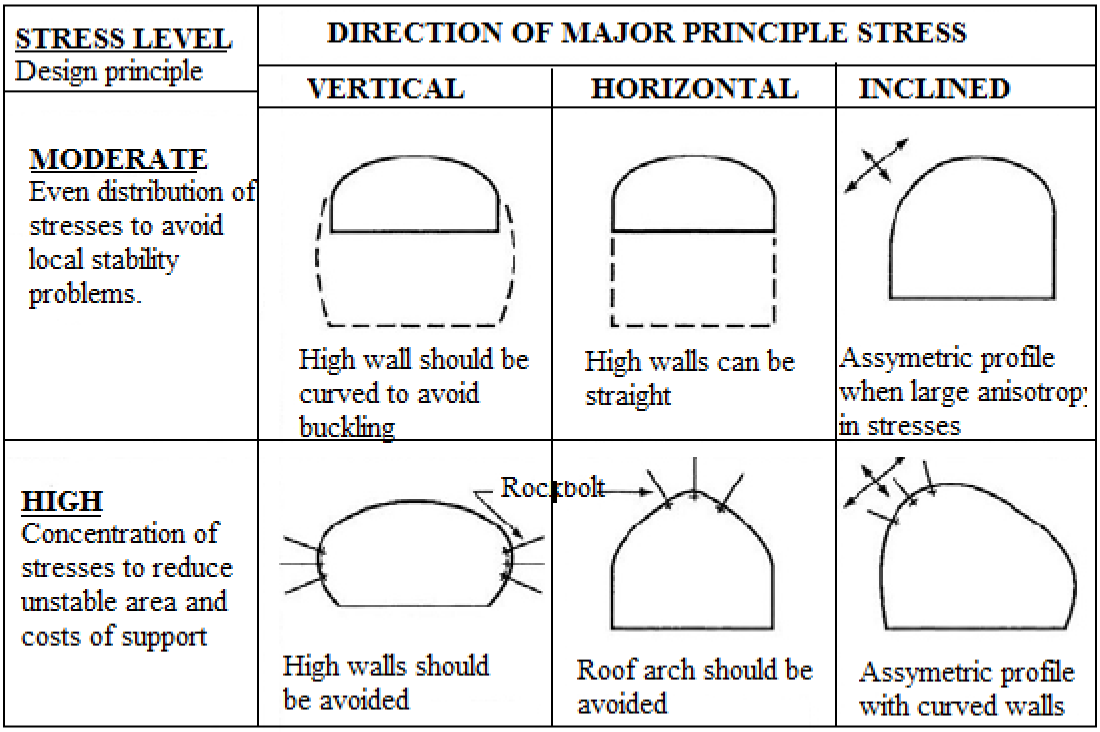


Figure 4-6 : Design principle for underground openings with varying stress and directions (Selmer-Olsen and Broch, 1997)

Wall stability depends on the wall height and joint orientation of rock. In most of cases, stable wall design leads to unstable roof systems and vice versa. Therefore, optimum shape and size of cavern is needed to minimize the rock support system and to increase the stability of cavern. Selmer-Olser and Broch, 1977 proposed some principles for underground openings with varying stress and directions which has shown in Figure 4-5.

In case of SMHEP, direction of major principle stress is almost vertical along the wall, therefore it is recommended to remove perfect vertical wall, which may fails due to buckling. Slight curve from the wall to the roof makes it possible to distribute the load evenly. To reduce the bucking effect, it can install the rock support system, which is estimated in the consequent chapters.

4.3.4 Spacing between Caverns

If the width of cavern is limited, alternative solution is to construct multiple caverns in parallel. Spacing between any two caverns should be selected by considering the stability of the each cavern. Stability of cavern is determined by the span/height ratio, rock mass quality, and stress condition. The need for large cavern should be avoided. Generally, accepted rule of thumb the thickness of the walls should be equal to the height of the caverns. For stability of walls, horizontal bedding or jointing is favorable but it might create stability problem of roof (Edvardsson and Broch, 2002).

For existing layout of SMHEP, spacing between the caverns is 9.5 m. This spacing has been used while doing numerical analysis and is suggested to modify. There might have some interference between those two caverns, if they are closer than 12 m. In numerical analysis, there are some problems while the spacing between the caverns is 9.5 m. This problem is only in the section B-B and C-C as shown in Figure 7-31.

4.3.5 Weakness Zone

It is necessary to avoid the major weakness zone, which is obtained by selecting the proper location and orientation of opening. If the space between weakness zones is too small for the opening, the different zone has to evaluate and the smallest is chosen for crossing the opening. Crossing should make as short as possible. Steep dipping discontinuities create the instability problem in wall, whereas horizontal discontinuities create the roof instability. Simple sketch of rock support related to the orientation of weakness zones is given in below in Figure 4.7 (Berthelsen, 1992)

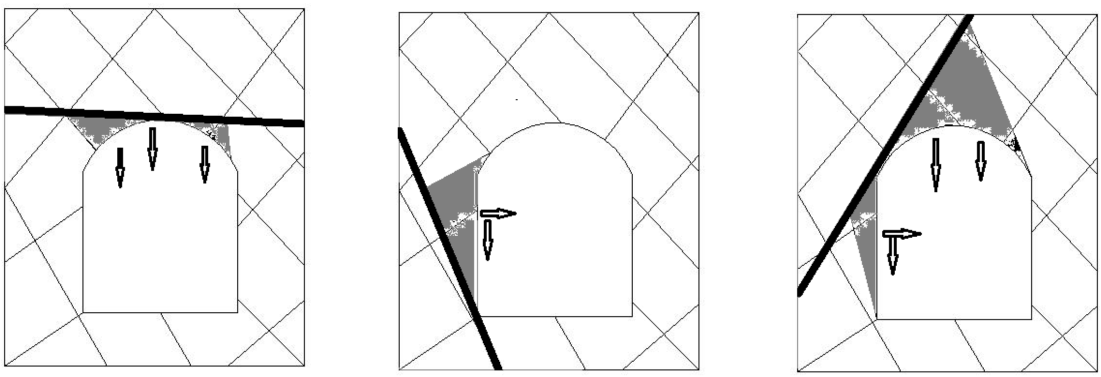


Figure 4-7 : Rock support related to orientation of weakness zones (Berthelsen, 1992)

Within Settling basin area of SMHEP, there are not any major weakness zones. 2D- ERT shows that the bedrock in the Madi River is about 35m below the current alluvial deposit. It indicates that, river is made by continuous weathering and erosion along the weaker part not along the major weakness zone.

4.3.6 Ground water (Water leakage)

Presence of large volumes of groundwater creates an operational problem in tunneling but water pressure is generally not too serious in non-pressurize tunnel. Groundwater pressure are a major factor in all slope stability problems and an understanding of the role of subsurface groundwater is an essential requirement for any meaningful slope design (Hoek and Bray, 1981, Brown 1982).

Inside settling basin of SMHEP, water will flow at atmospheric pressure that means the flow is due the gravity rather than the pressure force. Chance of losing the water from leakage is controlled by the lining or grouting around the periphery of basin. Leakage inside the cavern might create problem during and after the excavation of cavern. This possible problem should be minimized by providing pre-injection grouting or by constructing the water drainage system or combination of both, which depends upon the extent of leakage during and after the excavation

4.4 Conclusion:

After analyzing the orientation of prevailing joint set, two best alignments are taken into consideration for the analysis. Alternative I was proposed in the feasibility study where as alternative II is taken for the analysis in this study phase. Selection of best alternative will be done after the stability analysis. If both caverns are stable, then selection criterion would be based on economic and financial analysis.

CHAPTER 5

5 THEORY REVIEW AND ANALYSIS

5.1 Engineering geology and rocks

Rock mass is the heterogeneous mixture of same or different minerals/texture /composition which are bounded by some kind of joint system. Engineering properties of rock mass depends on character of their individual grains such as type, size, shape, texture, and bonds between each mineral particle. Other external factor such as weathering, anisotropy and alternation have significant role for altering the strength of parent rock mass. Gneiss are usually medium (>0.2mm) to coarse foliated and largely re-crystallized but do not have excess quantities of micas, Chlorite or other platy minerals. Band of intercalated mica schist in between the relatively strong gneiss makes it weaker. Mode of formation of rock and degree of metamorphism also define the characteristics of rock mass. Sedimentary, igneous, and metamorphic are the three-rock type based on their origin. When the rocks of all types are subjected to high temperature and pressure, they will melt and remobilize to be metamorphic rocks. During metamorphism, original minerals are changed to new minerals. Metamorphism mainly due to two way, one is contact metamorphism while another is regional metamorphism. Simple sketch of increasing order of regional metamorphism has shown in Figure 5-1.

Shale ⇒ Slate ⇒ Phylite ⇒ Schist ⇒ Gneiss

Figure 5-1 Order of metamorphism for gneiss (Panthi 2011)

Shale is the fine-grained sedimentary rock, when metamorphosed change to slate ,phylite and so on as shown in Figure 5-1.Gneiss is coarsely re-crystallized and may be as coarse as igneous rocks. Infield, Banding, and parallel orientation of minerals in rock are distinguished as Gneiss. Ortho-gneiss is from igneous origin whereas para-gneiss is results from the regional metamorphism of sedimentary rock.

5.2 Properties of rock and rock masses

There are 28 parameters, which influence the strength, deformability, permeability, and stability of rock masses. Palmstorm (1995) gives the concept of continuous and discontinuous rock mass. Size of opening compare to the size of blocks determines the stability of the rock mass after excavation. A term continuity factor (CF) is the ration of tunnel diameter (D_t) and the block diameter (D_b) to judge the rock mass is either continuous or discontinuous) If CF equals 5-100, rock mass is said to be discontinuous that means the rock mass is highly anisotropic and the individual block effect is the major concern in the analysis. If CF is less than five then the rock mass is assumed as continuous and behaves as a bulk materials. Selection of theory and analysis type is based on the type of rock mass that is either continuous or discontinuous.

Strength properties are governed by the properties of the intact rock and discontinuities in the rock mass. In-situ stresses and groundwater have influenced to change the properties of rock masses. In fundamental design and analysis of opening in the rock mass, forces acting on the rock mass should be defined to determine the reaction of the forces. Therefore, underground structure has to design to withstand the forces and the conditions that may imposed. Reliable estimates of strength and deformation of rock masses are needed for any type of analysis used during design of underground opening (Hoek, 2007). Knowledge of rock mass properties and imposed forces is vital to optimize the size and cost of construction within safe mode.

Failure mode of different types of rock mass

Failure modes of different types of rock have different as shown in Figure 5.2.Strong and brittle rock behaves like elastic material. Post failure behavior of such rock mass shows sudden drop of strength, after yielding.

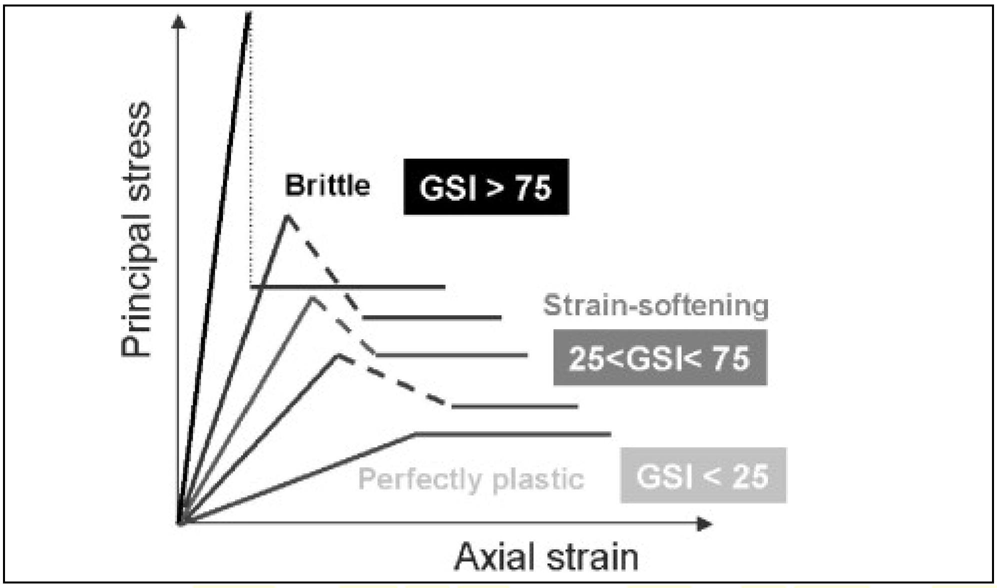


Figure 5-2 : Post-failure modes of different rock (Hoek and Brown, 1997)

Strength of rock mass, and principle stresses determine the modes of failure of rock mass as shown in Figure 5-2. According to above criterion, In case of SMHEP, post failure mode would be either strain softening or minor brittle failure, which is very difficult to predict exactly.

5.3 Hoek-Brown Failure Criterion

Reliable estimation of the strength and deformation characteristics of rock masses is required for all type of analysis in rock mechanics. Hoek and Brown (1980a, 1980b) proposed a method for obtaining estimates of the strength of jointed rock masses, based upon an assessment of the interlocking of rock blocks and condition of the surface between the blocks.

Generalized Hoek-Brown failure criterion for jointed and isotropic rock masses has been proposed in by Hoek et al. (2002), which is developed after several modifications over the years. It resolves the problems of uncertainty and inaccuracy of finding the equivalent friction angle and cohesive strength for a given rock mass. Software called RocLab makes it possible to solve and plot the following equations:

$$\sigma_1^{`} = \sigma_3^{`} + \sigma_{ci} \left(m_b \frac{\sigma_3^{`}}{\sigma_{ci}} + s\right)^{a}$$

Where,

$\sigma_1^{'}$ & $\sigma_3^{'}$ are maximum and minimum effective principle stresses at failure

σ_{ci} =Uniaxial compressive strength of intact rock; (Refer section 5.5.1.2.1)

m_b is reduced value of the material constant` $m_i$`

To evaluate the value of m_i (Refer section 5.5.1.5)

s and a are constants which depends upon the characteristics of the rock mass, and

Values of ``m_b , s and a ‘of the rock mass can be evaluated by using the following relation.

$$m_b = m_i \exp^{\left(\frac{GSI-100}{28-14D}\right)}$$

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

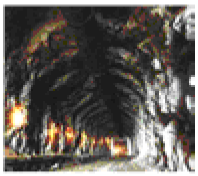
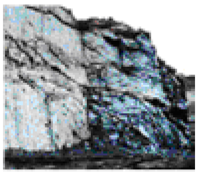
$$a = \frac{1}{2} + \frac{1}{6} \left(e^{GSI/15} - e^{-20/3}\right)$$

GSI is the geological strength index of rock mass (Refer section 5.5.1.4)

This failure criterion has been used while doing the numerical analysis through Phase^2 in chapter 7.

D is the Factor, which depends upon degree of disturbance of rock mass due to blasting and stress relaxation. Various value of D for different type of rock mass with different appearance has shown in Table 5-1.

Table 5-1 Suggested values of disturbance factor D (Hoek, 2007)

	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

Since the Rock mass in the settling basin area of SMHEP is slightly schistose, the effect of blasting is assumed as moderate. Therefore, the value of D is taken as 0.5 for this calculation.

Therefore calculated value of m_b , s and a are as follows.

$m_b = 6.71$: $s = 0.0117$: $a = 0.502$

These parameters are used further in numerical analysis in chapter seven, while using Generalized Hoek and Brown failure criterion.

Selection Criterion between Hoek & Brown and Mohr Columb

For the analysis of rock mass, two types of failure criteria are generally used. The selection criteria, which shows the suitability for particular type of rock mass is shown in Figure 5-3.According to the Figure 5-3, If the structure being large compare to the block size (Heavily jointed) then the rock mass strength have to be determined by using Hoek-Brown failure criterion. If the discontinuity spacing is larger (Either one or two joint sets) in comparison to structure dimension then Mohr-Coulomb failure criterion can be used for the stability analysis.

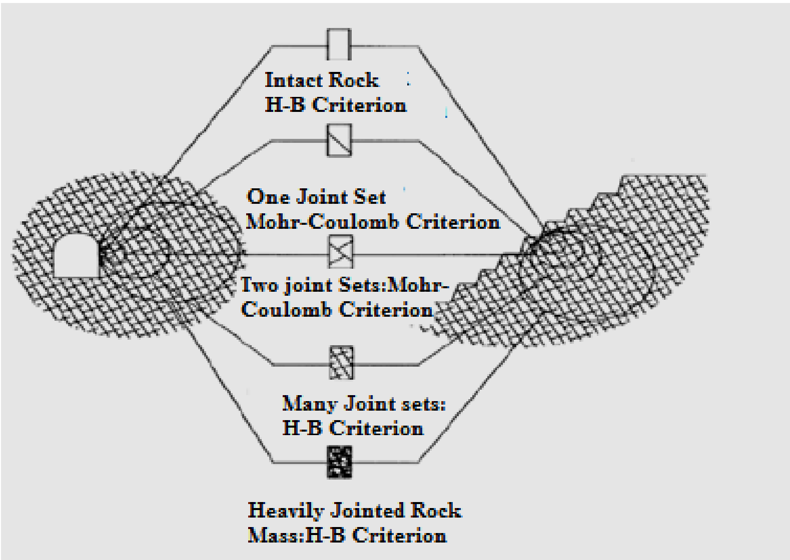


Figure 5-3: Failure criteria for different rock mass conditions (Panthi 2012)

In case of settling basin cavern of SMHEP, total number of joint set is more than two Therefore Generalized Hoek and Brown failure criterion has been used in the analysis.

5.4 Barton-Bandit Failure Criterion

Barton Bandit (1990) gives an empirical relation with consideration of joint roughness and alteration.

Revised form of original Barton equation for the shear strength of a rock joints is

$$\tau = \sigma_n \tan(\emptyset_b + \text{JRC} \log_{10}(\frac{\text{JCS}}{\sigma_n}))$$

Where,

$(\emptyset_b)$ = Basic friction angle of failure surface and is taken as 30 (Himal Hydro,2009)

JRC = Joint roughness co-efficient and is taken as 10 (Barton and choubey, 1977)

JCS = Joint wall compressive strength (78 Mpa)

τ = Shear strength

σ_n = Normal stress

In this report, this criterion is used while doing wedge failure analysis through the numerical modeling in chapter 8.

5.5 Input Variable needed for stability assessment

`The responsibility of the design engineer is not to compute accurately but to judge soundly`; (Hoek and Londe ; 1974).

Quality of input parameters, appropriate design methods, and qualified design engineer/engineering geologist are three major requirements for obtaining the reliable output. Stability of rock mass is solely depends upon the rock mass quality and mechanical processes involve in it. Major input variables to identify Quality of rock mass are rock mass strength, rock mass deformability, strength anisotropy, discontinuity, weathering, and alteration. The stability of an underground excavation is interdependent with the structural condition in the rock mass, degree of weathering of the rock mass and their relationship between rock stresses and rock mass strength (Hoek and Brown, 1980).stability of underground opening also depends upon the relative orientation of structural features and the opening Size, Shape, and the location of opening (Panthi, 2006).

5.5.1 Rock mass Quality

It is determined by evaluating the rock mass strength, deformability, strength anisotropy, degree of weathering and discontinuity. Rock mass classification system such as Q-value and RMR value of rock mass are used to describe the rock mass quality.

5.5.1.1 Rock mass classification

Rock mass classification is the tool for monitoring, recording, and comparing the predicted and actual rock mass conditions. It gives quantitative measurement of quality of rock mass. These rock mass classification systems should only be used for preliminary/planning purpose not for the final tunnel support. Two main rating methods, (RMR & Q-system) are essential for monitoring rock condition during construction but integration of them with a descriptive (NATM) would be most effective.

Commonly used Classification systems are on empirical basis. Analytical methods for classifications are used only to a minor extent. Most commonly used classification systems are as follows:

1. Descriptive Methods (New Australian Tunneling Method (NATM))

2. Rating Method (Bieniawski`s RMR-System (1973, 1974) or Q-system by Barton, Lien and Lunde (1974)

Descriptive Methods: (Terzaghi`s rock mass classification, New Australian Tunneling Method)

Terzaghi(1946) classified the rock mass in a descriptive way such as intact, stratified, moderately jointed, blocky and seamy, crushed, squeezing and swelling. Laufer(1958) gave the idea of stand-up time, unsupported span and quality of rock mass and their descriptive relationship. Concept of stand-up time is ``An increase in the span of the tunnel leads to a significant reduction in the time available for the installation of support`` .NATM gives some technique for safe tunneling in rock conditions in which the stand up time is limited before failure occurs. It suggests us to use of smaller heading and benching or the use of multiple drifts to form a reinforced ring inside the tunnel. This technique is best suited for soft rocks and not recommended for hard rock (Hoek 2012)

Q –System (Barton, Lien and Lunde(1974)

This is an quantitative classification system for the evaluation of the rock mass quality. It depends upon the mainly six ground parameters such as Rock Quality Designation (RQD), Number of joint sets (J_n), Joint roughness (J_r), Joint alternation number (J_a),Joint water reduction factor (J_w) and Strength reduction factor (SRF). Mathematical relationship between these parameters to evaluate the Q-value of rock mass is as follows (Grimstad and Barton, 1993)

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

Rock Quality Designation (RQD) is the length in Percent of measured length of the un-weathered drill core bits longer then 10cm (Deere (1963)).

RQD in volumetric joint basis by Palmstrom (1974) has been presented below:
RQD = 115 - 3.3 J_v (RQD = 0 for J_v > 35, and RQD = 100 for J_v < 4.5)

Palmstrom (2005) has found that
RQD = 110 – 2.5 J_v (For J_v = 4 to 44)

Where
J_v = Joint volume.

This second relation (Palmstrom (2005)) gives the better correlation, but still with several limitations.

RQD is insensitive when the rock mass is moderately fractured. On the other, it is the function of the total frequency, which is highly sensitive to sampling line orientation.

The second term ($\frac{J_r}{J_a}$) represents the roughness and frictional characteristics of the joint walls with or without filling material. This ratio is weighted in favor of rough, unaltered joints in direct contact. It is to be expected that such surfaces will be close to peak strength, that they will dilate strongly when sheared, and they will therefore be especially favorable to tunnel stability (Hoek, 2004).

Q-system has certain limitation because it does not include the joint orientation, joint size, joint persistence, joint aperture, rock strength. If we desire to cover the field other than stability and rock support, we have to consider the effect of rock strength during calculating the Q-value. Hence, new relation has been proposed instead of Q that is Q_c (Arild Palmstrom & Elinar Broch).

Table 5-2: Estimated Q-value for settling basin cavern of SMHEP (Himal Hydro 2009)

Rock Type	RQD	J_n	J_r	J_a	J_w	SRF	Calculate Q value	Rock class
Banded Gneiss	80-85	9-12	4-3	1-2	1	1	10-14.1	B (good)

In settling basin area of SMHEP, average value of Q is calculated based on above field parameters and have found in between 10 to 14.1. For further calculation in this analysis, least value of Q is taken.Lowest value of Q is taken by considering the fact that the cavern will be stable in all cases, if it is safe at weakest section. RMR value with rock support estimation is presented in chapter 6 of this report. Detail description of each input parameters for evaluation of Q-value and rock support requirements are presented in Appendix C.

5.5.1.2 Strength of intact rock and rock masses

Rock mass is composed of several intact rocks and discontinuities. Strength of rock mass is mainly depends on the conditions and properties of those variables. Strength of rock mass is the measure of resistance to deformation in a variety of stress conditions.

Strength of intact rocks

Unit-axial compressive strength (σ_{ci})

It is determined either by directly unit-axial compressive strength test in the laboratory or indirectly from the point load strength test (ISRM 1972). Hoek and Brown suggested that the unit-axial compressive strength (σ_{ci}) of rock specimen with diameter of d (mm) is related to the uni-axial compressive strength (σ_{c50}) of a 50 mm diameter sample by following equation (Hoek & Brown 1997)

$$\sigma_{ci}=\sigma_{c50}\left(\frac{50}{d}\right)^{0.18}$$

Where,

$\sigma_{c\ 50}$=uniaxial compressive strength of 50 mm diameter sample

d = Diameter of actual sample (mm)

Compressive strength of rock specimen also depends upon the angle of schistocity to the loading direction (degree), which is shown in Figure 5.4.

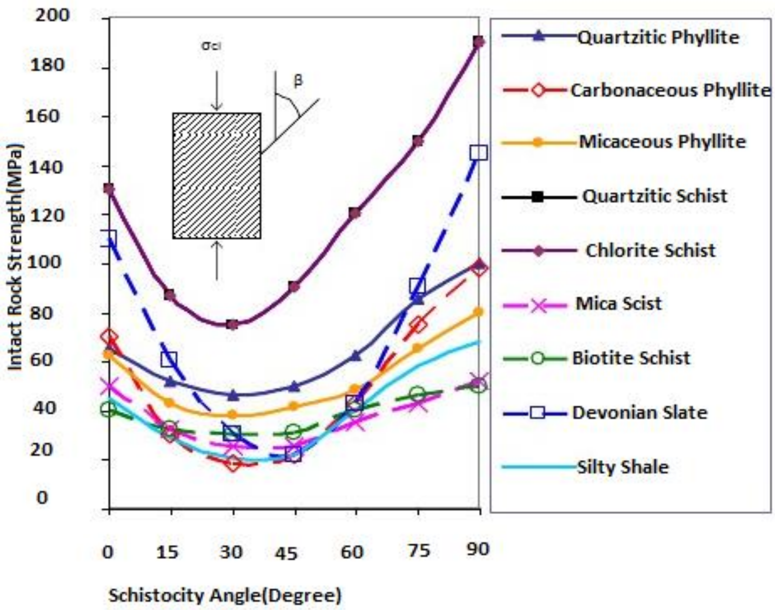


Figure 5-4 : Uni-axial compressive strength of intact rock measured at different angle of schistocity (Panthi 2006))

Difference in the strength of 50 mm diameter specimen and actual sample is due to variation in faults, joints and other discontinuities. If the rock mass has high strength anisotropy, it is recommended to take uni-axial strength test in several directions to get more accurate results. But, In case of settling basin area of SMHEP, There are not any laboratory test to measure UCS value of intact rock.

Point Load Test:

This is another method to evaluate the uni-axial compressive strength of intact rock (σ_{ci}) in the laboratory. Point load strength (I_s) is determined by using the following relation.

$$I_s = \frac{P}{D_e^2}$$

Where,

P = Measure load in failure (KN)

D_e=Equivalent sample diameter

If the sample diameter is different from the 50mm then some adjustment is needed. Adjustment for the test where the sample diameter is different from 50mm is

$$I_{s\ 50} = F.\ I_s$$

Where, $F = (\frac{D_e}{50})^{0.45}$

Strength anisotropy and water content are the influencing parameter to alter the actual point load strength. This method is a form of indirect tensile strength test. $I_{s\ 50}$ Is approximately 0.8 times the unit-axial tensile test (Brazilian tensile strength)

Uni-axial compressive strength (σ_{ci}) of rock is determined by correlating it to point load strength (I_s) as follows (Nilsen & Palmstrom)

$\sigma_{ci} = K.\ I_s$, or

$\sigma_{ci}50 = K_{50}.\ I_{s50}$

Values of K depend on the strength of rock. It is lesser in weak rock then strong rock. Based on point load strength the values of k and σ_{ci} is suggested, which has shown in Table 5.3.

Table 5-3 : Suggested values of K_{50} and compressive strength (σ_{ci}) (Nilsen and palmstrom)

Compressive Strength σ_{ci} (MPa)	Point Load Strength I_{s50}(MPa)	Suggested Value of K_{50}
25* -50	1.8-3.5	14
50-100	3.5-6	16
100-200	6-10	20
>200	>10	25

* Point load strength test is not valid for rocks having σ_{ci}< 25 MPa (Bieniawski(1973)

According to Nilsen & Palmstrom, Tensile strength of banded gneiss For settling basin area of SMHEP, is calculated as:

$IS_{50}/0.8 = 3.5/0.8 = 4.3$ Mpa

For SMHEP, there is no uni-axial compressive strength test data in the settling basin area. Table 5-4 gives some good approximation of the UCS value of intact rock. These values are used for the guidance to estimate the UCS value of intact rock in settling basin area

Table 5-4 : Value of UCS (σ_{ci}(MPa)) of intact rock for different part of the world.

Type of Rocks	Test of rocks worldwide (Nilsen & Palmstrom)		For Himalayan (Panthi 2006)	For Himalayan(UTKHEP and NHP)	ISRM,1978(Field estimate value)
	σ_{ci}(MPa)	Number of Tests	σ_{ci}(MPa)	σ_{ci}(MPa)	σ_{ci}(MPa)
Gneiss	130	27	(50-100) Banded gneiss)	35(for schist and gneiss)	100-250
Micaschist	104	16			

There might be some deviation in UCS value of banded gneiss due to local parameters. Worldwide measure strength is somehow higher than the UCS value in Himalayan region, which may be due the highly fractured and young rock, weathering condition of sample piece, laboratory methodology, non-homogeneity of minerals in the rock etc.

SMHEP and UTKHEP are located at the same geological area with same rock type (Higher Himalayan region), but the rock mass in the settling basin area of SMHEP is stronger than UTKHEP. It is due to less weathered and massive rock mass in SMHEP. Difference in UCS of SMHEP and worldwide measure value may be due to the schistosity, non-homogeneity, anisotropy, and intercalation of mica-schist in between relatively strong gneiss in SMHEP. In this analysis, UCS value is calculated by reducing the weathering effect from the world test results presented in Table 5-4.. According to Bearie 1985, Gupta and Rao 2000, The strength of rock mass in Himalayan region is reduced by 40% from the completely fresh rock(Refer Figure 5.5).Value of UCS based on their suggestion for Himalayan weathered rock becomes 78 MPa. This value is also similar to Panthi (2006) `s suggestion.

<u>Strength of Rock masses</u>

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

In SMHEP, Settling basin cavern is located within the area having high-grade metamorphic rock. Rock mass is slightly deformed foliated micaous and banded gneiss with thin layer of schist. The rock is slightly weathered and massive having three sets of rough, irregular, undulating, tight to moderately open joints with medium to high persistency filled with silt. Therefore, strength of rock mass is always lesser then the strength of intact rock.

Analytically, it is almost impossible to evaluate the rock mass strength. For practical use, some authors and scientists have suggested some empirical formulae. Table 5-5 gives some useful relation.

Table 5-5 : Empirical formulae for estimation of rock mass strength (σ_{cm})

Developed/proposed By	Relationship	Calculated Values(Mpa)
Barton(2002)	$\sigma_{cm} = 5\gamma * {Q_c}^{\frac{1}{3}} = 5\gamma(\frac{\sigma ci}{100} * Q)^{\frac{1}{3}}$	25.78
Bieniawaski(1993)	$\sigma_{cm} = \sigma_{ci} * \exp(\frac{RMR-100}{18.75})$	12.06
Panthi(2006)	$\sigma_{cm} = \frac{{\sigma_{ci}}^{1.5}}{60}$	11.48
Hoek at al(2002) and Hoek(1994)	$\sigma_{cm} = \sigma_{ci} * s^a = \sigma_{ci} * (\mathrm{Exp}^{(\frac{GSI-100}{9})})^a = \sigma_{ci} * \mathrm{Exp}(\frac{RMR-105}{9}))^a$	11.07

Where,

σ_{cm} = Unconfined compressive strength of rock mass in MPa

σ_{ci} = Uni-axial compressive strength of intact rock of 50mm core Diameter (in MPa).

RMR = Bieniawaski`s rock mass rating (1973, 1974)

S & a are the material constant related to Hoek & Brown failure Criteria (value of `a` ranges from 0.5 to 0.58 for GSI value of 100 to 10)

GSI= Geological Strength Index

γ =density of Rock mass ton/m^3

Q_c =Normalized rock mass quality rating

Q = Rock mass quality rating

Values of Q , GSI and RMR have been discussed in sections 5.5.1.1 , 5.5.1.4 and 6.2.1 respectively.

For Himalayan region with highly schistose, foliated, thinly bedded and anisotropic rocks of metamorphic and sedimentary origin with low compressive strength the Co-relation given by ``Panthi 2006`` may be useful (Panthi 2006).

5.5.1.3 Rock mass discontinuities

Discontinuities are the structural feature of the rock mass that makes it non-homogeneity. Joints, weak schistose plane, faults, weak bedding plane, and all other weakness zone are called discontinuity, which reduces the tensile strength of rock mass. Discontinuities are mainly result of tectonic activity. Joints in the metamorphic rock are called foliation joints whereas in sedimentary rocks called bedding joints. Strength of rock mass is determined by the number of joint set (joint density), length and continuity of joints, condition of filling materials and roughness /waviness of joint wall. Degree of jointing is measured in terms of Rock quality designation (RQD).

Weakness Zone and Faults

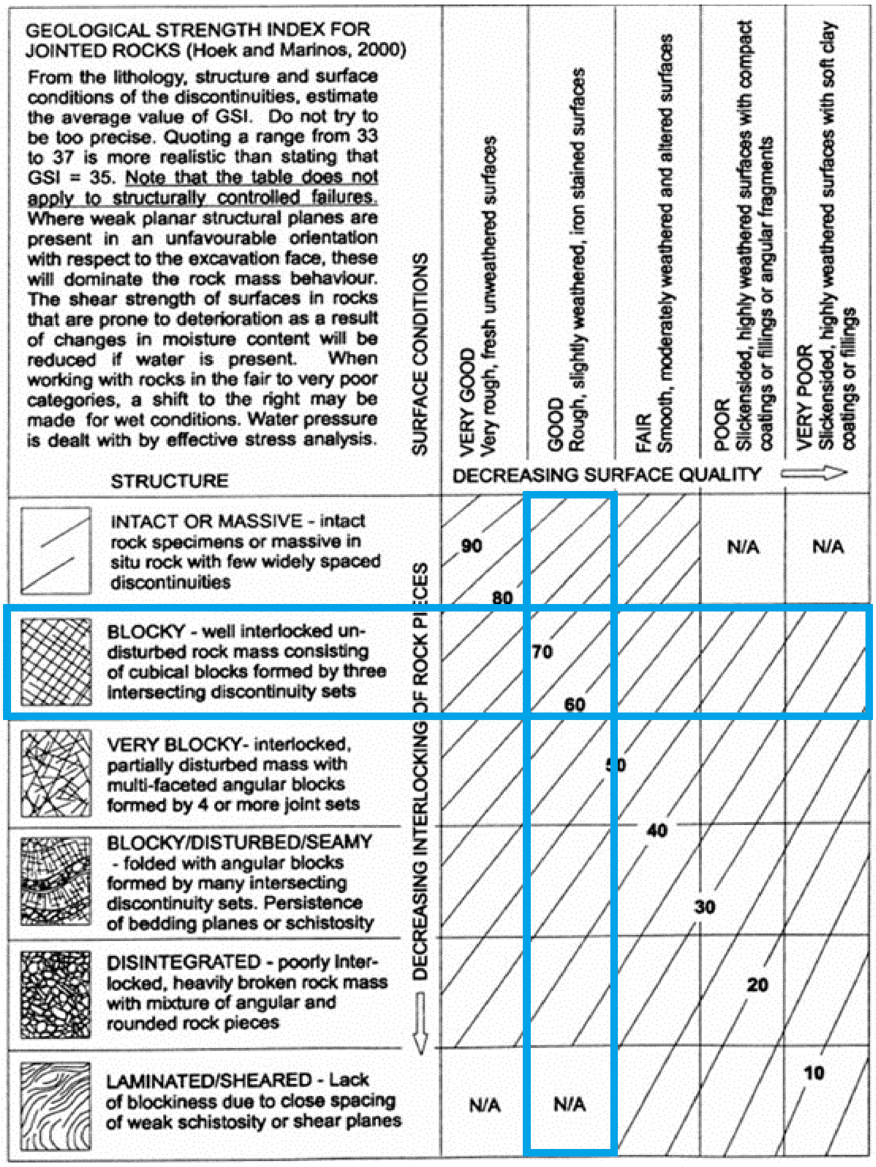
Weakness zone are the part of rock mass having relatively low strength than the surrounding rock. It may be faults zone, thrust zone, shear zones or weak mineral layers etc. Faults are the result of ruptures throughout the geologic time. Its width is ranging from centimeter to hundreds of meters. Walls are often polished and striated because of shear displacement. Rocks on the both side of fault are shattered and weathered or altered, resulting in fillings like gouge and breccias. There are mainly four types of faults, which are normal, reversed, shear/strike-slip and wrench fault.

In settling basin Area of SMHEP, there is not, any significant weakness zone that will creates problem but of course thin layer of filling material into the joints. It is assumed that will not create any significant problem for stability of underground cavern. Presence of those filling material is taken into consideration during evaluation of rock mass quality indices.

5.5.1.4 Geological Strength Index (GSI)

It determines the reduction in strength of rock mass (σ_{cm}) and material constant (mi) from in-situ condition to the laboratory value. This variation is due to the presence of discontinuities and condition of discontinuity in in-situ condition.(Hoek and Marinos 2000). To estimate the geological strength index(GSI) for jointed rock mass Hoek and Marinos,2000 gives simple chart which consists of input variables such as ; Structure of rock mass(focus on discontinuity) and surface quality of rock mass(Refer Figure 5.5)

Figure 5-5 : Geological strength index for jointed rock-mass (Hoek and Marinos, 2000)



For the case of settling basin area of SMHEP, rock mass in the cavern area is BLOCKY type with three sets of discontinuities. Surface of rock mass is rough and slightly weathered. Therefore, Suggested value of GSI from the Figure 5-5 is in a range of 55 to 75.For further analysis of cavern, GSI value of rock mass is taken as 60.

5.5.1.5 Hoek & Brown Constant, (m_i)

It has been computed from the statistical analysis of tri-axial tests, if available, but in case of SMHEP there is not any test result. Therefore, it would be better to use the value of this parameter based on (Hoek 2000)`s suggestion. Value of this constant depends upon the texture and structural origin of the rock mass. Figure 5.6 below, shows the approximate value of M_i and recommended for further analysis.

Figure 5-6 : Approximate value of `$m_i$` for different type of Intact-Rock (Hoek and Brown)

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

For SMHEP, banded gneiss is the metamorphic rock with few intercalation of mica schist. Therefore, for this type of schistose banded gneiss, value of M_i is taken as 28 for further analysis as an input parameter.

5.5.1.6 Rock mass deformability

Deformation modulus (E_m) of rock is an important parameter in any form of numerical analysis. It is the ratio of stress to corresponding strain during loading of rock mass under elastic and inelastic behavior. Whereas Modulus of elasticity or young modulus (E_{ci}) is ratio of stress to the corresponding strain during loading of rock mass only within elastic region.

Jointed rock mass does not behave elastically and hence it needs to formulate new terminology which covers both the elastic and plastic behavior of the rock mass, which is termed as deformation Modulus (E_m). (Bieniawski, 1978).Some useful empirical relations to evaluate the rock mass deformability are presented in Table 5-6.

Table 5-6 : Empirical formula for estimation of deformation modulus (E_m).

Proposed By	Deformation Modulus(E_m) (GPa)	Restrictions	Calculated value(Gpa)
Bieniawaski (1978)	$E_m = 2\,RMR - 100$	$RMR > 50, \sigma_{ci}$>100MPa	30
Palmstrøm(2002)	$E_m = 8\,Q^{0.4}$	1<Q<30	20
Serafim and Pereira(1983)	$E_m = 10^{\frac{(RMR-10)}{40}}$	RMR<60	23.7
Hoek et al.(2002)	$E_m = \left(1-\frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}} * 10^{\frac{GSI-10}{40}}$	$\sigma_{ci} < 100$ MPa	21.35
	$E_m = \left(1-\frac{D}{2}\right) * 10^{\frac{GSI-10}{40}}$	$\sigma_{ci} > 100$ MPa	
Barton(2002)	$E_m = 10 * (\frac{Q * \sigma_{ci}}{100})^{\frac{1}{3}}$		19.83
Panthi(2006)	$E_m = \frac{1}{60} * E_{ci} * \sigma_{ci}^{0.5}$		3.97

Where,

E_{ci} =modulus of elasticity of intact rock

E_m =Rock mass deformation modulus

D=Degree of Disturbance due to blast damage and stress relaxation (Refer Table 5.1).

Theoretically, for homogenous, isotropic, and massive rock mass the ration between rock mass strength and intact rock strength is equal to the ration between deformation modulus and elasticity modulus. Then rock mass deformation modulus (E_m) is expressed as follows:

$$E_m = E_{ci} * (\frac{\sigma_{cm}}{\sigma_{ci}})$$

For Himalayan region with highly schistose, foliated, thinly bedded, and anisotropic rocks of metamorphic and sedimentary origin with low compressive strength the Co-relation given by Panthi (2006) may be more relevant. Among all, Panthi (2006) did his extensive research in Himalayan region and is able to co-relate the rock mass deformation modulus with the young`s modulus of elasticity of intact rock. Since laboratory value of young`s modulus of elasticity have better reliability then other predicted parameters such as Q-value, RMR and GSI value which are used by other authors, It is recommended to use Panthi approach to evaluate the rock mass deformability (E_m) in Himalaya and is used here as well. Hoek et al.(2002) is used the term disturbance factor(D) which is also very relevant and practical. His approach may be un-doubtfully useful for other region of the world.

5.5.1.7 Rock Weathering

Weathering is the natural process of disintegration and decomposition of rocks. It is the consequence of changing environment of the earth surface. Both the physical and chemical phenomenon involves during weathering of the rock for example frost wedging, thermal expansion, unloading and organic activity leads to physical weathering whereas oxidation, dissolution and hydrolysis process help for chemical weathering. Rate of weathering of the rock mass is influence by several factors such as rock composition (minerals), structure, climate, topography etc. (Brattli, 2002).

Weathering reduces the properties of rock mass such as rock mass strength (tensile strength, unit-axial compressive) strength, elasticity modulus, deformability, frictional resistance, porosity, density and slaking durability. It may increase the permeability considerably (panthi 2006).Intensity of weathering is measured in terms of grade, which is a relative terminology. As Intensity of weathering increases, the grade will also increase. Decrease in strength of rock masses due to weathering has shown in Figure 5-7.

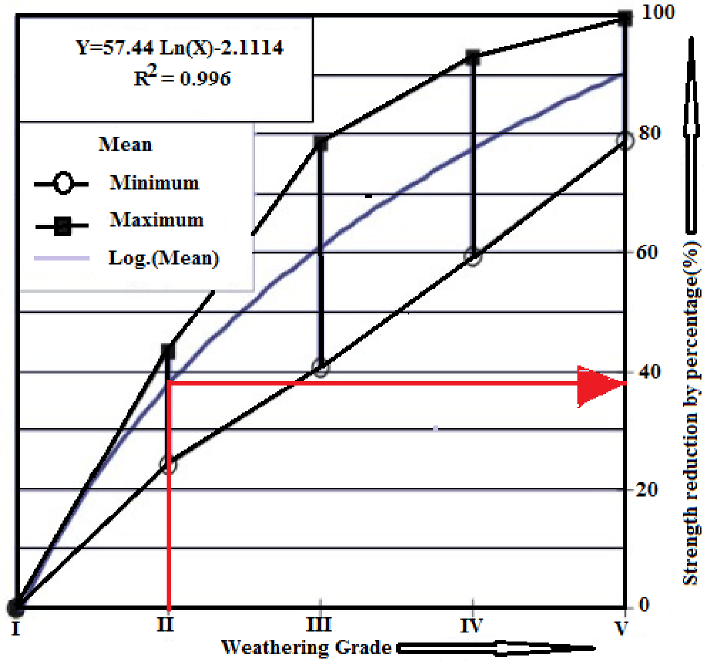


Figure 5-7 : Reduction in strength (percentage) due to weathering (Panthi 2006).

In ground surface of settling basin area of SMHEP, Rock mass is slightly weathered. In field, Rock material is discolored with some discontinuous surfaces. It is assumed that the Surface rock is weaker then fresh rock``. Therefore weathering grade for this rock is II (according to ISRM; 1978).In above Figure 5.5, weathering grade of II, means reduction in strength is about 40% from its ideally non weathered parent rock.

5.5.1.8 Other mechanical Properties

In SMHEP, laboratory test for evaluation of Poisson`s ration, bulk density and modulus of elasticity are not carried out. Therefore, those parameters have taken from the similar projects where laboratory test has been carried out. In this respect, UTKHEP and SMHEP have many geological similarities such as rock type (Banded gneiss) and geological location (Higher Himalaya).Some mechanical properties of banded that are used in the Himalayan geology are shown in Table 5-7.

Tabell 5-7 : Mechanical properties of Banded gneiss (Panthi 2006/Neupane 2010)

Mechanical Properties	Values (Neupane 2010)	Panthi 2006
Poisson`s ration	0.2	0.12
Bulk density(g/cm^3)	2.7	2.68
Young's modulus of elasticity(GPa)	27	25±3
Porosity (%)		0.87±0.1

In case of SMHEP, following values are taken for the analysis:

Poisson`s ration : 0.2

Bulk density (g/cm^3) : 2.68

Young's modulus of elasticity (GPa) : 27

5.5.2 Rock stresses

Stress surrounding the underground opening depends up on the stress situation prior to excavation and the geometry of the opening. Virgin stresses are the resultant of the vertical stress caused by gravity, tectonic stresses, topographic and residual stresses. Distribution of this virgin stress will be changed after the excavation of openings. Vertical stress is induced because of the overlying strata. If we assumed the homogenous and isotropic rock mass the vertical stress due to the overlying strata, (gravitational stress) is calculated by using following relation.

$\sigma_v = \gamma.h = \rho.g.h$

Where

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

ρ =Average density of rock mass (ton/m^3)

h =Depth of overburden (m)

g =acceleration due to gravity ($^{m}/_{s^2}$)

Stress due to plate tectonic movement is in horizontal direction therefore total horizontal stress is the vector summation of tectonic stresses and horizontal effect of gravity stresses. Mathematical form of total horizontal stress is as follows:

$$\sigma_H = \frac{\vartheta}{1-\vartheta}\,\sigma_v + \sigma_{Tec} = K.\sigma_v + \sigma_{Tec}$$

Where,

`$\vartheta$` is the poison ratio - Ratio of transverse strain to the axial strain of intact rock

Sheorey (1994) suggested a relation to evaluate the value of `K` by considering the curvature of the crust and variation of elastic constants, density, and thermal expansion co-efficient through the crust to mantle. According to him:

$$K = 0.25 + 7\,E_h\,(0.001 + \frac{1}{z})$$

Where E_h is the average deformation modulus of the upper part of the earth crust measured in a horizontal direction and z (m) is the depth below the surface.

For SMHEP, E_h is taken as 3.97 GPa (Refer Table 5.8) for banded gneiss. The following figure shows the different value of K at different depth.

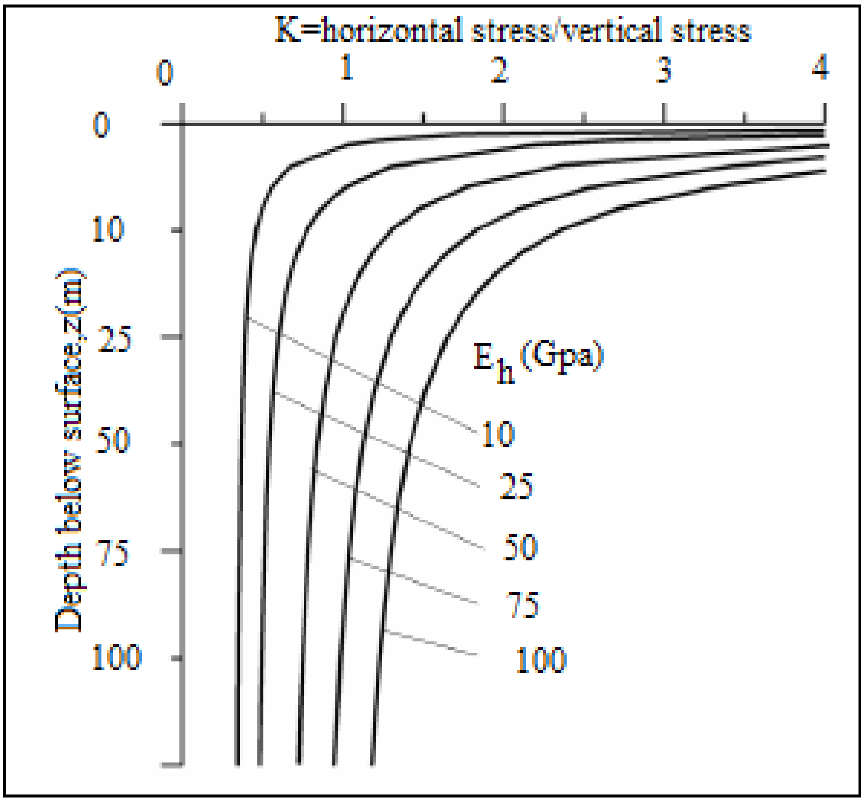


Figure 5-8 : Different Value of K at different Depth (Sheorey (1994)

Table 5-8 : Calculated value of rock stresses

Sections	Z(m)	E_h(GPa)	K	σ_v	σ_H (In plane)	σ_H (Out plane)
A-A	92	3.97	0.58	2.48	2.73	7.37
B-B	218	3.97	0.40	5.88	3.7	8.32
C-C	272	3.97	0.38	7.344	4.1	8.72

To determine the magnitude and direction of tectonic stress at any particular location, three dimensional stress measurements data are needed. In SMHEP, there is not any test carried out to measure the tectonic stresses. It is assumed that the horizontal stress due to northward movement of Indian plate is almost same as that of UTKHEP, where three dimensional stress measurements had been carried out. Tectonic stresses at UTKHEP along the tunnel alignment are 3.5MPa and 14.4MPa respectively. Stress map of Nepal shows that the major horizontal stresses are almost north south direction as shown in Figure 5-9..

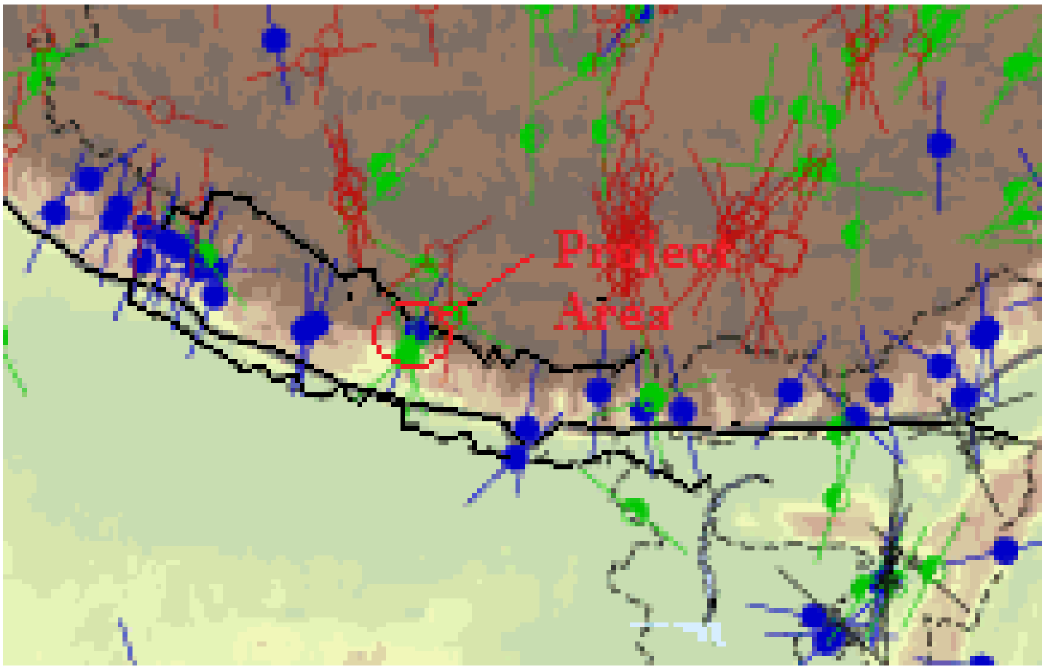


Figure 5-9 : Stress map of Nepal (Source: www.world-stress-map.org, 2012)

Tectonic stresses measurement based on UTKHEP gives abnormal values therefore has not taken for this analysis. In case of SMHEP, value of tectonic stress is taken as six Mpa in a north south direction (Panthi 2010). The resolving components of the tectonic stresses along and in perpendicular to the cavern alignment are 1.29 MPa and 5.93 MPa respectively. These values are adopted for further analysis in this thesis work.

5.5.2.1 Stress distribution around the circular tunnel

Virgin Stresses in the rock masses are changed due to excavation. Stress set up around the opening depends on the magnitude and direction of principle stresses and geometry of the opening. For homogenous, isotropic and elastic rock mass with iso-static virgin stress ($\sigma_1 = \sigma_2 = \sigma_3 = \sigma$), radial and tangential stress is calculated by using the following relations.

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

$$\sigma_R = \sigma \times (1 - \frac{r^2}{R^2})$$

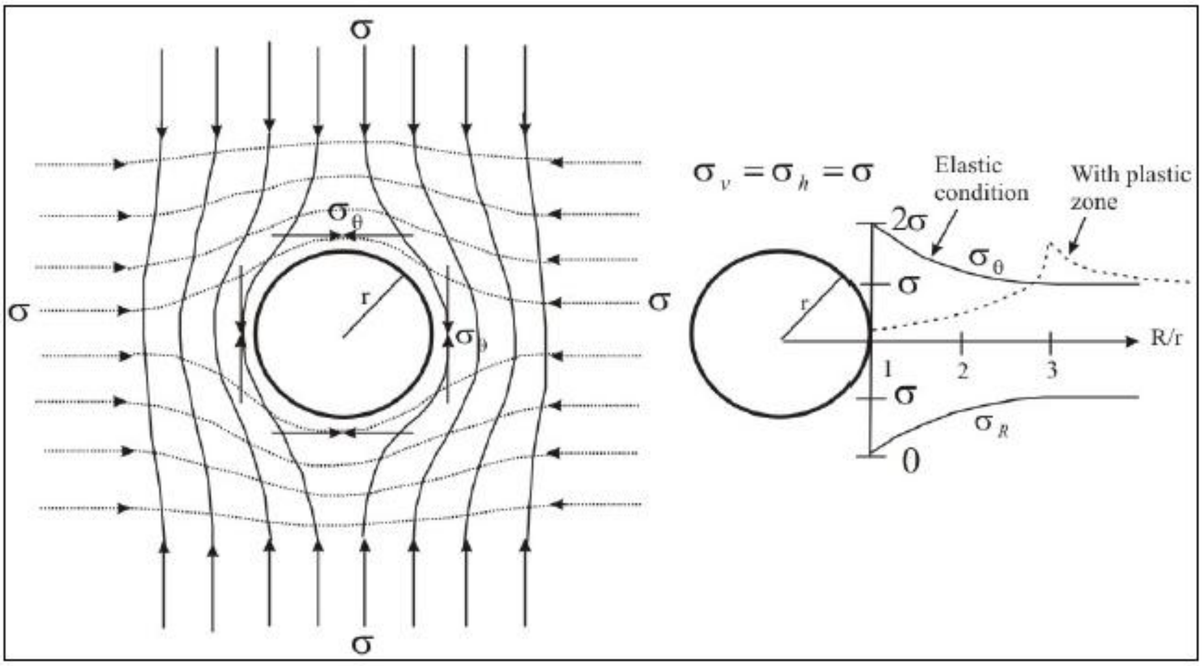


Figure 5-10 : Stress Trajectories in rock mass surrounding a circular opening(left) and tangential and radial stress distribution in elastic and non elastic conditions(right); (Panthi,2006).

Where, σ_θ and σ_R are tangential and radial stresses at the periphery of the circular opening in an iso-static stress condition of elastic rock. R and r are the radial distance and opening radius.

For anisotropic rock mass, Kirsch`s equation may be useful to evaluate the tangential stress around periphery of the opening after excavation in the circular opening of elastic material. Maximum and minimum values of tangential stresses are calculated by using the following relations.

$$\sigma_{\theta(max)} = 3\sigma_1 - \sigma_3$$

$$\sigma_{\theta(min)} = 3\sigma_3 - \sigma_1$$

Where,

σ_1 = Major principle stress; which is calculated analytically by resolving the stresses along the valley slope.

σ_3 = Minor principle stress; which is calculated analytically by resolving the stresses perpendicular the valley slope (perpendicular to theσ_1).

Kirsch`s equation may give some good result in this case because the cavern is located near by the slope face of valley. Calculation of stresses for cavern of SMHEP based on this approach is presented in Table 5.9.

Table 5-9 : Tangential stresses at cavern depth for both alternatives at three different sections (According to Kirsch`s equation)

Section	Alternative I		Alternative II	
	$\sigma_{\theta(max)}$ (MPa)	$\sigma_{\theta(min)}$(MPa)	$\sigma_{\theta(max)}$(MPa)	$\sigma_{\theta(min)}$(MPa)
A-A	7.529331367	4.787694878	7.529331367	4.787694878
B-B	15.63123096	4.609077681	15.63123096	4.609077681
C-C	20.57040702	4.043549659	20.57040702	4.043549659

In Table 5-9, Virgin stresses in both the alternative at corresponding sections are same. But in actual case, there is redistribution of stresses due to excavated geometric profile. Therefore, practical approaches that consider the cross-sectional geometrical profile of excavated cavern, is described in the following section.

5.5.2.2 Practical method to estimate the magnitude of tangential stresses

After excavation, the stress is redistributed along the periphery of the opening. If this induces stress after the excavation exceed the strength of the rock mass, there will be chance of instability. In-stability does not only occur in the area of maximum tangential stress but also in low stresses area due to jointed rock mass. To estimate the magnitude of tangential stress around the periphery of the different shape of openings, Hoek and Brown (1980) gave some empirical relations and have discussed below:

Tangential stress in the roof ($\sigma_{\theta r}$) = (A × K – 1) σ_v

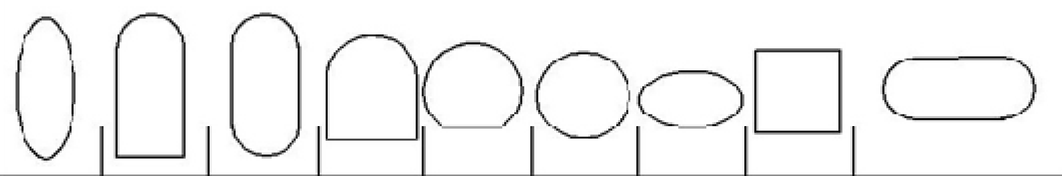
Tangential stress in wall ($\sigma_{\theta w}$) = (B-K) σ_v

Where,

A and B are the roof and wall co-efficient for the various excavation shapes as shown in Figure 5.9.

K : Ratio of horizontal and vertical stress

σ_v : Virgin vertical stress



	Tunnel Shape								
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

Figure 5-11 : Different values of Roof and Wall coefficients for different shape of tunnel (Hoek and Brown (1980)

For settling basin area of SMHEP, The value of A and B are taken as 3.2 and 2.3 for alternative I. and 3.0 and 2.4 for alternative II.

This approach is used to calculate the tangential stresses at wall and roof of cavern. Calculated values are compared with the strength of rock mass, to know the stability conditions. Stability analysis based on this approach has been carried out. Stresses condition of wall and roof in both alternatives at three different sections are shown in Table 5.10.

Table 5-10 : Stability analysis of caverns (Criterion based on Hoek and Brown (1980)

Alternatives	Section	$\sigma_{\theta r}$	$\sigma_{\theta w}$	$\sigma_{\theta w}/\sigma_{cm}$ (Wall)	$\sigma_{\theta r}/\sigma_{cm}$ (Roof)
I	A-A	2.144	4,2	0.37 (No rock burst)	0.18 (Usually favorable)
	B-B	1.77	11.145	0.97(Moderate slabbing)	0.154 (Favorable stress condition)
	C-C	1.6	14.09	1.22 (Heavy rock burst)	0.14 (Favorable stress condition)
II	A-A	1.85	4.5	0.39 (No rock burst)	0.16 (Favorable stress condition)
	B-B	1.29	11.73	1.1 (Heavy rock burst)	0.11 (Favorable stress condition)
	C-C	1.05	14.83	1.29 (Heavy rock burst)	0.09 (Favorable stress condition)

Rock bursting criterion to show the types of stability problems are presented in Table 5-11. Above results show that roof is favorable in all sections of both cavern whereas heavy rock bursting problems occur at section B-B and C-C of both alternatives.

5.6 Stability analysis

5.6.1 Stability problem due to tensile stress

Because of discontinuity in the rock mass, it cannot resist high tensile stress. Very small tangential stress may create radial crack in periphery of the opening. Generally, minor cracks does not create problem of stability. In case of high-pressure tunnel, it is more important that secondary jointing and opening of existing joints may create the risk of water leakages out of the tunnel. In case of settling basin cavern, this type problem may not arise because of non-pressure flow inside the cavern.

According to Nilsen and palmstrom,(2000), tensile strength of intact piece of banded gneiss is taken as $I_{s\,50}/0.8 =$ 4.3 MPa. (Refer Section 5.5.1.2).

5.6.2 Stability problem due to High compressive stress

In the contour of the underground opening, normally there are diametrically opposite area of tangential stress concentration. Stability problems normally occur in the high stress concentration area but very low tangential stress may also create the problems.

Rock burst/spalling/popping out

If the rock mass is hard and brittle, rock burst problem may arises in the area of high compressive stress when the rock mass strength is lesser than the imposed compressive stress. If the fracturing is accompanied by strong noises then it is called spalling. Main cause of rock spalling is asymmetric shape of the tunnel profile. Location of spalling/popping out of rock indicates the direction of major principle stress. This problem generally occurs just after the excavation takes place, if the strength of rock is low compare to the stresses on it. Instability problems may arise after few years of excavation. (Due to reduction on strength of rock mass cause by water pressure, creep etc).

in SMHEP, rock mass is banded gneiss which is hard and brittle and is assumed to have rock burst problem in great depth but due to intercalation of weak rock inside the hard rock, squeezing problem may occur. Nature of stress related problems are predicted based on semi analytical approach, which is presented in Table 5-11.

Table 5-11 : Prediction of stress related problems according to the Q-system (Grimstad and Barton, 1993).

Consequence of stresses	σ_c/σ_1	σ_θ/σ_c
1)In competent, massive, hard rock		
Moderate slabbing after >1hr	5-3	0.5-0.65
Slabbing and rock burst after a few minutes	3-2	0.65-1
Heavy rock burst(strain burst) and immediate dynamic deformation	<2	>1
2)In squeezing rock		
Mild squeezing		1-5
Heavy squeezing		>5

Based on the above criteria develop by Grimstad & Barton, 1993, Stress related problems for both alternatives at different sections are presented in Table 5.12.Stability analysis is done based on tangential stresses and is more relevant for weak rock where tangential stress (in periphery) is somehow less compare to hard rock. For hard rock, stability analysis is also done based on Major principle stress. Based on this approach, following result comes after the calculation.

Table 5-12 : Prediction of stress related problem for hard rock according to the Q-system based on Grimstad and Barton, 1993).

Alternatives	Sections	σ_c/σ_1	Consequences
I	A-A	3.3	Moderate slabbing after >1hr
	B-B	1.78	Heavy rock burst
	C-C	1.4	Heavy rock burst
II	A-A	3.14	Moderate slabbing after >1hr
	B-B	1.66	Heavy rock burst
	C-C	1.37	Heavy rock burst

Table 5-12 shows that section B-B and C-C have heavy bursting problem whereas minor rock spalling problem occurs at section A-A.

Factors affecting the stress problems are rock mass properties such as jointing systems, strength properties, anisotropy, and elastic properties. Orientation of major principle stress relative to the direction of major joint sets and structural features, such as bedding and schistosity have a major influence on rock bursting and spalling. Major problems may occur if the schistosity runs parallel to the cavern axis, and the major principle stress acts perpendicular to the axis and in the dip direction of schistosity Rock of rock burst/spalling is varying along the tunnel because of variation of stresses(it relative direction), type of rock and elastic properties.

For gneiss, tunnel section having rich in mica are often characterized by stress relief, while the rock burst is confined to more quartz and feldspar rich sections (Panthi 2006).

Tunnel Squeezing:

When weak rock mass, having many discontinuities, is subjected to rock stresses and inflow of water, there is high chance of collapsing the tunnel by squeezing phenomenon. In such condition, rock mass deformed plastically. Weak rock behaves in a different way than isotropic and stronger rocks, when subjected to tangential stresses. When the strength is less than the induced tangential stresses along the tunnel periphery, micro cracks along the schistose or foliation plane will be formed. It generally happens in weak rock such as shale, phylite, and slates or at weakness zone. In case of SMHEP, rock mass seems (from surface) strong and brittle, there is less chance of squeezing. However, it may happen because of unpredictable nature of Himalayan geology. Some useful and accepted approaches for predicting the tunnel squeezing are discussed below.

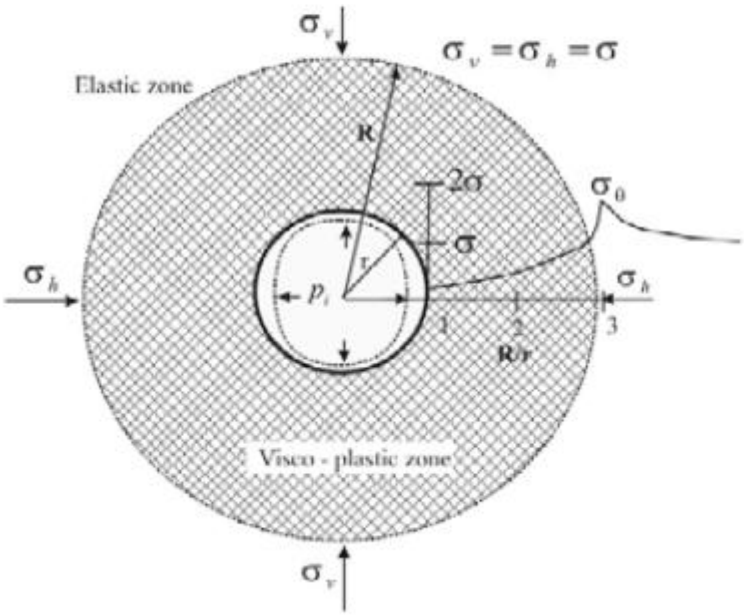


Figure 5-12 : Illustration of Squeezing (Panthi, 2006)

Figure 5.12 shows, how the plastic deformation happens, in a weak rock mass. Conditions of stresses (vertical stresses and horizontal stress) during squeezing are also shown in this figure. Tunnel squeezing occurs prior to the stresses reaches it maximum tangential stress. Rock mass within the visco-plastic zone will squeeze. Figure 5.13 shows an example on how the squeezing occurs around the tunnel periphery in weak rock mass.

Figure 5-13 : Squeezing in tunnel kaligandaki HEP (left) and Modi (HEP) right. (Panthi 2000)

Some Prediction for tunnel Squeezing based on empirical and semi analytical approach has described below.

<u>**Empirical approach**</u>

Singh et al approach:

In this empirical approach, squeezing problem is defined by the rock mass quality (Q) and overburden for Himalayan tunnels. In figure 5.14, Squeezing problem will only occur above the straight line. Following relation defines whether the squeezing problem occurs or not.

$$H => 350Q^{1/3}$$

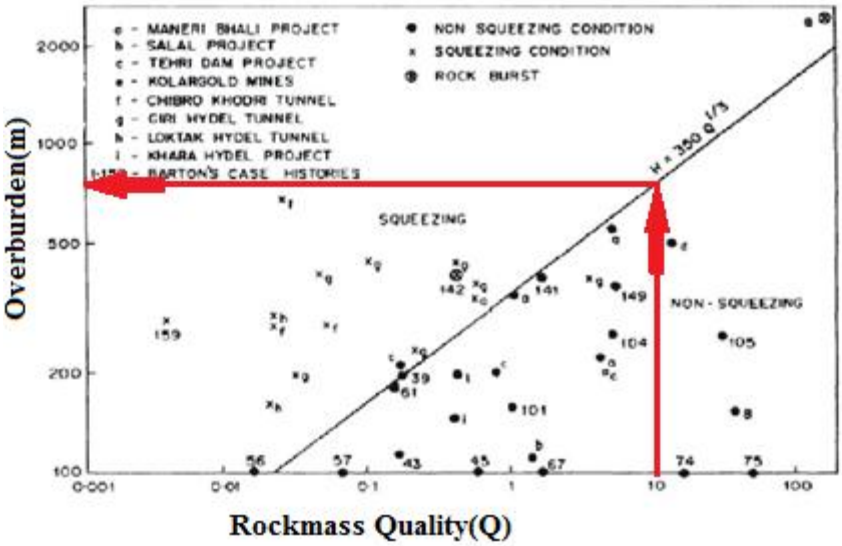


Figure 5-14 : Empirical criterion for rock mass squeezing (Singh et al (1992))

If the Q-value is greater than ten then Squeezing problem may only occur below 800m depth of overburden in weak rock. This is shown in Figure 5.14.

In case of settling basin area of SMHEP, maximum overburden is only about 300 m. Therefore, according to this empirical approach, there is no instability problem due to squeezing.

Semi-Analytical Approach:

Hoek and Marionos approach:

Semi-analytical approach of predicting the squeezing problem given by Hoek and Marinos (2000) is more reliable than purely empirical approach because deformability and strength change after the excavation of rock mass over time have greater consequence on squeezing than only the overburden(Kovari,1998).Total inward tunnel strain(ε_t) in percentage can be evaluated using the following relation.

$$\varepsilon_t = \left(0.2 - 0.25 \times \frac{p_i}{\sigma_v}\right) \times [\frac{\sigma_{cm}}{\sigma_v}]^{(2.4 \times \frac{p_i}{\sigma_v} - 2)}$$

Where,

p_i , σ_v and σ_{cm} are the support pressure, vertical stress and rock mass strength in MPa.

When support pressure (p_i) is zero then the relation becomes as follows. This is useful to determine the total strain before the support is installed.

$$\varepsilon_t = 0.2 \text{ x } (\frac{\sigma_{cm}}{\sigma_v})^{-2}$$

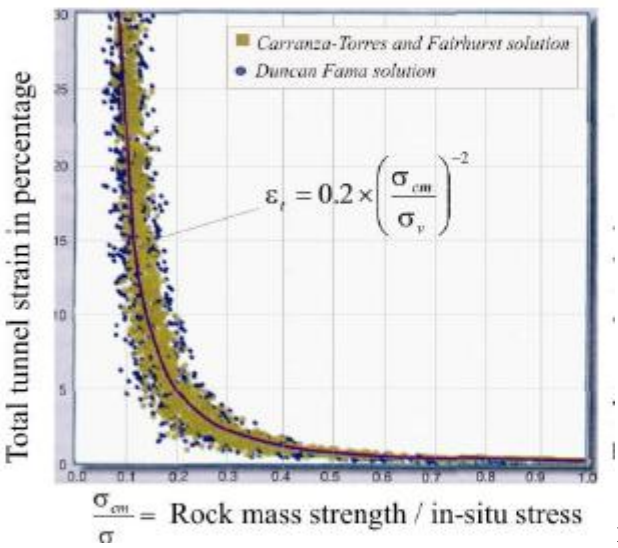


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Figure 5-15 : Total Tunnel strain (convergence) against the ratio of rock mass strength and in-situ stress (Hoek and Marinos, 2000)

Figure .5.15, shows the rate of change of total strain with respect to the ration of rock mass strength and in-situ stress. Degree of difficulty also increases as the tunnel strain increases.

Very weak rock masses are incapable of sustaining the differential stresses and failure occurs when the in-situ horizontal stresses and vertical stresses become equal. Since tangential stress is always higher than other stresses, failure (squeezing) occurs before the stresses reaches the tangential stresses. Therefore, for evaluating the strain in squeezing rock, vertical stress is consider instead of tangential stresses.

Hoek and Marinos also define the size of plastic zone (R), with support condition follows.

$$R = r * \left(1.25 - 0.625 * \frac{p_i}{\sigma_v}\right) * \left(\frac{\sigma_{cm}}{\sigma_v}\right)^{\left(\left(\frac{p_i}{\sigma_v}\right) - 0.57\right)}$$

Since this relation is developed based on test result of several circular openings, it works well for circular opening in an iso-static condition of stresses. But it can also be used for non-circular opening (like cavers of SMHEP) as well with certain deviation in accuracy. Calculated values of total strain based on Hoek and Marionos approach have been presented in Table 5-13

Tabell 5-13 : Calculation of Total strain (ε_t) based on Hoek and Marionos approach:

Alternative	Sections	σ_{cm}/σ_v	Total Strain (ε_t) in %
Alternative I	A- A.	4.63	0.0094
	B-B	1.95	0.053
	C-C	1.56	0.082
Alternative II	A- A	4.63	0.0094
	B-B	1.95	0.053
	C-C	1.56	0.082

Table 5.13 shows the total strain in percentage for both alternatives at three different sections. Strains in all section are insignificant. Therefore In case of settling basin cavern of SMHEP, there is very less chance of occurring a squeezing problem.

5.6.3 Ground water and leakage in rock masses

Presence of large volumes of groundwater creates an operational problem in tunneling but water pressure is generally not too high in non-pressurize tunnel. Groundwater pressure is a major factor in all slope stability problems and an understanding of the role of subsurface groundwater is an essential requirement for any meaningful cavern design (Hoek and Bray, 1981, Brown 1982).Therefore it is necessary to estimate the amount of ground water in flow or outflow from the cavern. Some relevant techniques to measure the flow in underground openings are Lugeon test, Water pressure measurements, Water inflow registration etc. For

Himalayan geology, a useful empirical formula has been developed which is based on some specific project of Nepal. This empirical formula may suit well for SMHEP, when needed and is discussed below.

Panthi`s approach (2006)

Panthi (2006) proposed a relationship, to determine the specific leakage, which is based on the assumption that the leakage is function of the rock mass quality index (Q) and hydrostatic head. Location of ground water table (Which determines the hydrostatic head (H_{Static})), degree of jointing (J_n), joint roughness (J_r) and joint alteration (J_a) have major role to determine the extent of leakage from the tunnel. Specific leakage (q_t) in the tunnel is estimated by using following empirical relation (Panthi 2006).

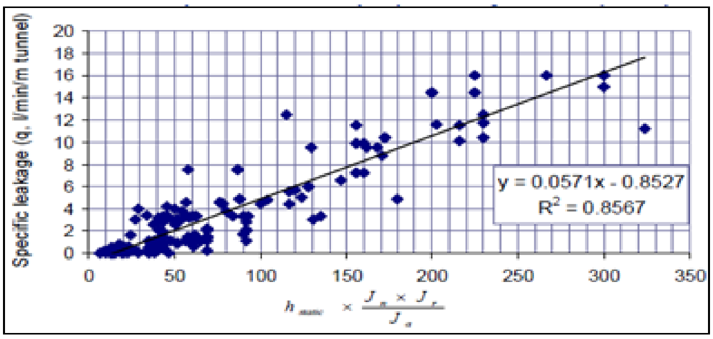

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Figure 5-16 : Relationship between Specific leakage and Static head (Panthi 2006)

$$q_t = f_a \times h_{static} \times \frac{j_n \times j_r}{j_a}$$

Where,

q_t =Specific leakage in the tunnel in liters /minute/ meter

f_a = joint permeability factor (0.001-0.15) in (l/minute/m^2)

h_{static} =Static head at that particular section (m)

J_n =joint set number

J_r= Joint roughness numb

J_a= Joint alteration number

Panthi`s relation may be best suited in Himalayan region but It is strongly recommended to incorporate another parameter RQD(Rock Quality Designation) which is the only one term that can be measured physically in laboratory to represent the actual strength of rock mass. Other parameters that are use to evaluate the Q value are only based on assumption.

In case of settling basin cavern of SMHEP, problem due to water leakage is assumed as trivial. Therefore, detail analysis related to leakage problem is not carried out.

5.7 Comparison, Discussion, Conclusion and Recommendation

Different analytical and empirical approaches are used - either in combine or in individual form - to analyze the problems of stability in underground settling basin cavern of SMHEP.

As an empirical approach, Rock mass classification system is used to predict the quality of rock mass either in the form of Q-Parameter or RMR parameter. Empirical equation suggested by Singh et al, Grimstad &Barton, 1993(Based on Q –value) and semi-analytical approach suggested by Hoek and Marionos have been used. Based on all above approach, none of the section of both alternatives have squeezing problem. Maximum total strain is at section C-C for alternative II and equal to 0.082%.

Rock burst criterion suggested by Grimstad &Barton, 1993 (Based on Q –value) and Hoek and Brown(1980) are used. Stress parameter (σ_1) is evaluated by resolving the stresses along the valley slope (considering the stress anisotropy in valley side), whereas σ_c and σ_θ are evaluated based on suggestion by Panthi`s (2006) and Hoek and Brown (1980) respectively. Analysis shows that rock bursting and spalling problems is high in section B-B and C-C of both alternatives. Section A-A has also minor rock spalling problems. Rock-bursting problem is increases as we go more inside the hill because of increasing rate of overburden. There is not more space available near the valley side of the cavern to shift the position of cavern closer to the valley side. Therefore, it is suggested to install the rock support system to make the cavern more stable and safe. Estimation of rock support system have been suggested and is carried out in chapter six.

It is strongly recommended to perform numerical analysis, which may give more reliable results than empirical and semi-analytical approaches. Numerical analysis has been carried out in chapter seven.

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CHAPTER 6

6 ROCK SUPPORT ESTIMATION

6.1 Introduction

Installation of rock support is to improve the stability conditions of the underground openings. Type and extent of rock support is determined by actual condition of ground after excavation. Flexible support systems such as bolt, lining (concrete, RCC, Geo-textile etc), or combination of them are used to accommodate the continuous changes of rock mass quality. Normally initial support is used for providing safe working condition whereas permanent support is installed to meet the requirements for satisfactory functioning of the opening during its lifetime. Prediction of rock support requirements are very important and difficult task for any engineering geologist because of uncertainly in the geological condition and lack of development of exact methodology to choose the rock support type, pattern and total quantity. This uncertainty leads to overrun the cost of project. There are some approaches to evaluate the rock support requirements, which are presented below.

6.1.1 Empirical approach

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

6.1.2 Analytical Approach

Based on the analysis of stress and deformation rock support requirements are predicted. Numerical approach such as finite element, finite difference, boundary elements, and physical modeling technique are common in this approach.

6.1.3 Observational approach:

Actual monitoring of ground movement during excavation to measure instability and analysis of ground support interaction are the major task in this approach. New Austrian tunneling method (NATM) and convergence-confinement methods are the commonly used approach for estimating rock support.

6.2 Rock mass classification for estimation of rock support

6.2.1 RMR system of classification for estimation of rock support

Bieniawski (1973) developed a system of rock mass classification based on his experience in many tunnel projects. According to him, five major parameters affect the strength of rock mass, which are listed below. Detail criterions for rating the rock mass based on these parameters are presented in Appendix C.

1) Uni-axial compressive strength of intact rock material
2) Rock quality designation (RQD)
3) Condition of discontinuity
4) Spacing of discontinuity
5) Ground water condition

Bieniawaski, 1989 has also given the idea for adjustments for relative orientation of discontinuity and tunnel alignment. Tunnel alignment is assumed as good. Rock mass in the cavern area is banded gneiss with average compressive strength of 78MPa (Refer Chapter five). Average predicted Q-value is lies between 10 to 14.1.RQD of rock mass lies in between 80-85.Ground water inflow is less than 10 l/s. Using the above rock mass parameters, calculated value of RMR is found to be 65 to 70 (Refer Appendix C)

RMR values are also computed directly by using the following conversion equation (Bieniawski(1989) and Barton(1995))

$RMR = 9 \ln Q + 44$ (Bieniawski, 1989)
$RMR = 15 \log Q + 50$ (Barton, 1995)

By using the above relationship, value of RMR is found to be in between 65 to 68 in the settling basin area. Rock mass class determined from total rating is II, which means good quality. Since this prediction is only based on surface measurement and may not reflect the rock mass quality inside the hill (Where cavern is located). For, the case of cavern at SMHEP, RMR value calculated from the above equations is taken into consideration. It has been used for further calculation. To estimate the rock support system and stand up time based on RMR value, Figure 6.1 has been used. Stand up time is the term used to figure out the estimation of rock support systems. According to the figure 6-1; Stand up time, RMR value, Roof span, and required support system for both the alternatives have been evaluated as follows:

<u>Alternative I (Two different caverns for each settling basin):</u>

For this alternative, roof span for each cavern is about 8.4m with average calculated RMR value of 66.Based on this RMR value and span length; standup time is approximately 4.5 month. This time is sufficient to install the rock support system during / just after the excavation.

<u>Alternative II (One single large cavern for both basins):</u>

In this alternative, both the settling basin cavern are incorporated into one single big cavern with span length of 18.3 m. Rock mass type, quality of rock mass, location of cavern are all similar to that of alternative I. Chances of instability problem is high compare to alternative I because of larger span. Calculated value of stand up time based on Bieniawski, 1989 is found to be approximately one month. Therefore, it is suggested to put the required support system within one month of excavation. For good quality of rock mass, requirement of rock support system with excavation and corresponding way of installation of rock support is shown in Table 6.1, (Bieniawski, 1989).

Following figure 6.1 shows the relationship between the stand up time, RMR value, roof span, and required support system.

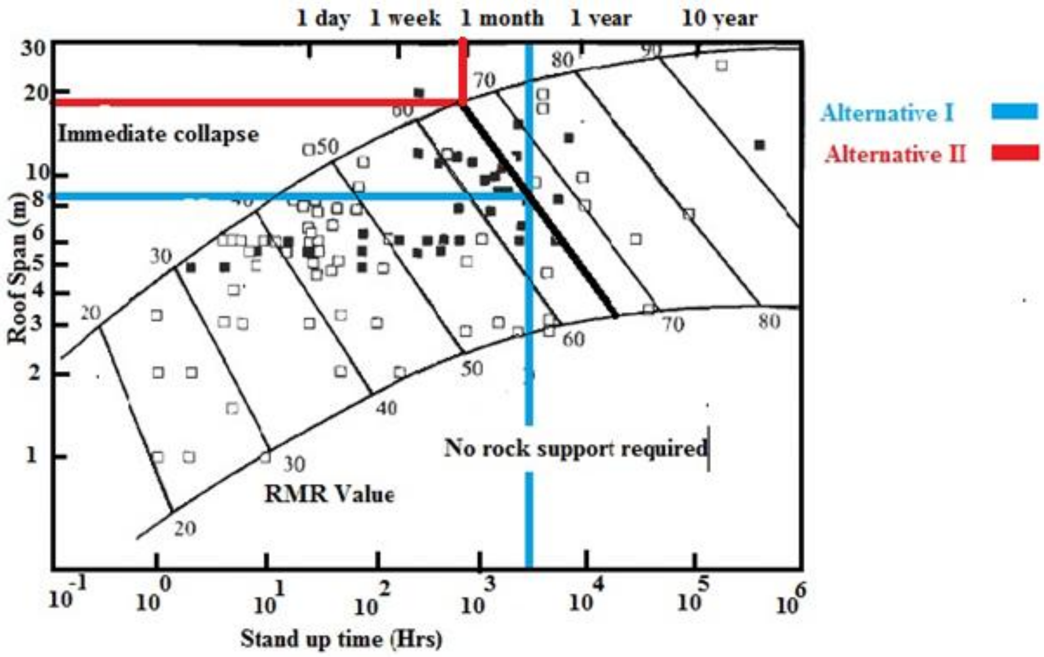

Figure 6-1 : RMR classification chart of rock masses at settling basin caverns (After Biewniawski, 1989)

Based on the above figure, Rock support estimation for both alternatives has been done and are discussed below.

Table 6-1 : Guide for excavation and supports in tunnel for 10 m span based on RMR value (Bieniawski, 1989).

Rock mass class	Excavation	Supports		
		Rock bolt(20mmdia,fully bonded)	Shotcrete	Steel sets
Good Rock(RMR:61-80)	Full face 1.0-1.5m advance Complete support 20m from face	Locally bolts at crown,3m long spaced 2.5m with occasional wire mesh	50 mm in crown where required	None

Support systems that are presented in table 6.1 may be useful for alternatives I but is not suitable for the alternative II due to its limitation on span (10 m only) of opening.

6.2.2 Q-system classification for estimation of rock support

After analysis and evaluation of approximately 200 tunnels cases Barton et al (1974) of the Norwegian Geotechnical Institute (NGI) proposed the Q system of rock mass classification. For SMHEP, Six parameters that influence the value of Q are presented in Table 6-2. Detail description about the Q system is presented in Appendix D.

Table 6-2 : Rock mass parameter for cavern area (Himal-Hydro, 2009)

Rock Type	RQD	J_n	J_r	J_a	J_w	SRF	Calculate Q value	Rock class
Banded Gneiss	80-85	9-12	4-3	1-2	1	1	10-14.1	B (good)

After using, the relation suggested by Q system (Refer section 5.5.1.1) Average Q-value of rock mass in the settling basin area is found to be in between 10 to 14.1. Based on the Q-value, support requirements of the underground excavation have been determined. Barton et al (1974) has suggested chart to evaluate the rock support in any type of underground openings, which is function of two variables-Equivalents Dimension (De) of the excavation and Q-value.

Where,

$$D_e = \frac{\text{Excavation span or diameter or wall height(m)}}{\text{Excavation support Ratio ESR}}$$

Value of ESR is depends upon the purpose of excavation and degree of security needed (stability).Some values for different excavation category have been proposed by Barton et al (1974).Which has shown in Table 6.3.

Table 6-3 : Rating of excavation support ratio (ESR), from Barton et al. (1974)

Type or use of underground opening	ESR
Temporary mine opening	3.5
Vertical shafts, Rectangular, and Circular respectively.	2.0-2.5
Water tunnels, permanent mine openings, adits, drifts.	1.6
Storage caverns, road tunnel with little traffic, access tunnels etc.	1.3
Power stations, road tunnels and railway tunnels with heavy traffic, civil defense shelters , etc.	1.0
Nuclear power plants, railroads stations, port arenas, etc.	0.8

Here, Average value of `Q` is taken as 12 for estimation of rock support. Rock mass with Q-value of 12 is presumed as good quality with rock class B (Grimstad et al.; 2002).

Alternative I:

For alternative I, Width of cavern is taken as 8.4 m whereas the maximum height is 15m. Based on these geometric figures, Equivalents Dimension (considering both wall and span) are calculated. Value of ESR is taken as 1.6 as described in Table 6.3.Average Q-value of 12 is taken for calculation, which gives the average quantity of rock supports. Figure 6.2 and Table 6.4 show the rock support chart and calculated support system for alternative I respectively.

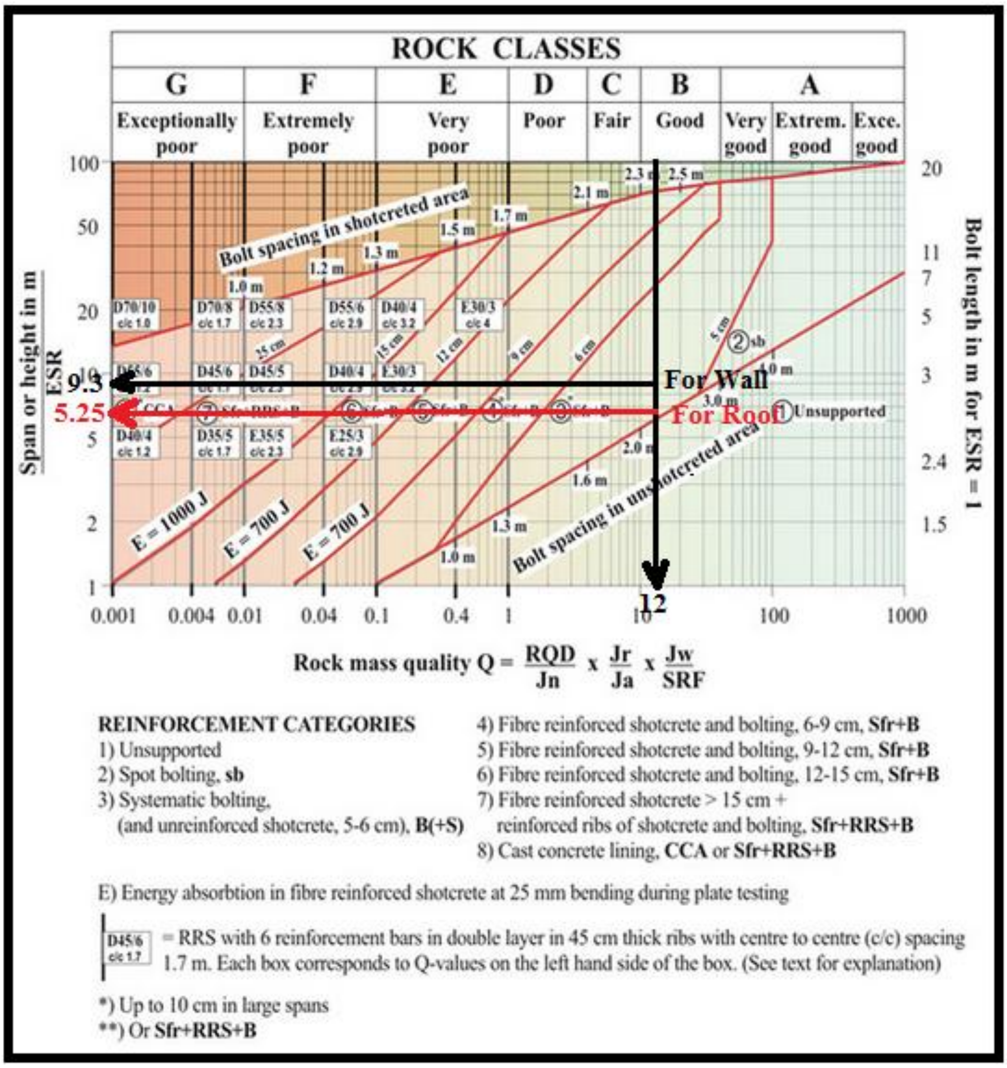

Figure 6-2 : Rock support chart according to Q-system for settling basin cavern-alternative I (Grimstad et al., 2002).

Table 6-4 : Calculated amount of support system (for alternative I)

Alternative I					
Span (m)	Average height (m)	D_e(Roof/Invert) (m)	D_e(Wall) (m)	Support system at wall	Support at roof and invert
8.4	15	5.25	9.3	-Systematic bolting with spacing 2.5 m -Bolt length is 3m -Steel-reinforced shotcrete 5.75 cm)	-Systematic bolting with spacing 2.2m -Bolt length is 2.8 m -Steel-reinforced shotcrete 5.5 cm)

From the Figure 6-2, although required rock support system for both roof and wall seems similar, wall is more sensitive than roof. Therefore, it is recommended to put few more support system in wall than roof (Refer Table 6-4)

Alternative II:

Heights of cavern at different section vary from 15m to 20m whereas width is 18.3 m. Calculated values of rock support requirements based on Figure 6.3 for both wall and roof at all section of consideration are presented in Table 6.5.

Table 6-5 : Calculated amount of support system based on figure 6.2, for alternative II

Alternative II					
Span (m)	Average Height (m)	D_e(Roof/Invert)	D_e(Wall)	Support system at wall	Support at roof and invert
18.3	18	11.6	9.4	-Systematic bolting with spacing 2.3 m -Bolt length is 5m -reinforced shotcrete 7.5 cm)	-Systematic bolting with spacing 2.2m -Bolt length is 5 m -reinforced shotcret 7.5 cm)

Note: For large cavern, Up to 10 cm thickness of shotcrete may have to apply (Grimstad et al., 2002).Therefore It is suggested to installed 10cm thick shotcret at roof but 7.5 cm is sufficient for wall.

Detail description of each input parameters for evaluation of Q-value are presented in Appendix C.During installation of rock support, it is suggested to instal shotcrete first then only rock bolt because shotcrete takes the load and gives the confinement to the cavern promptly. (Panthi 2012)

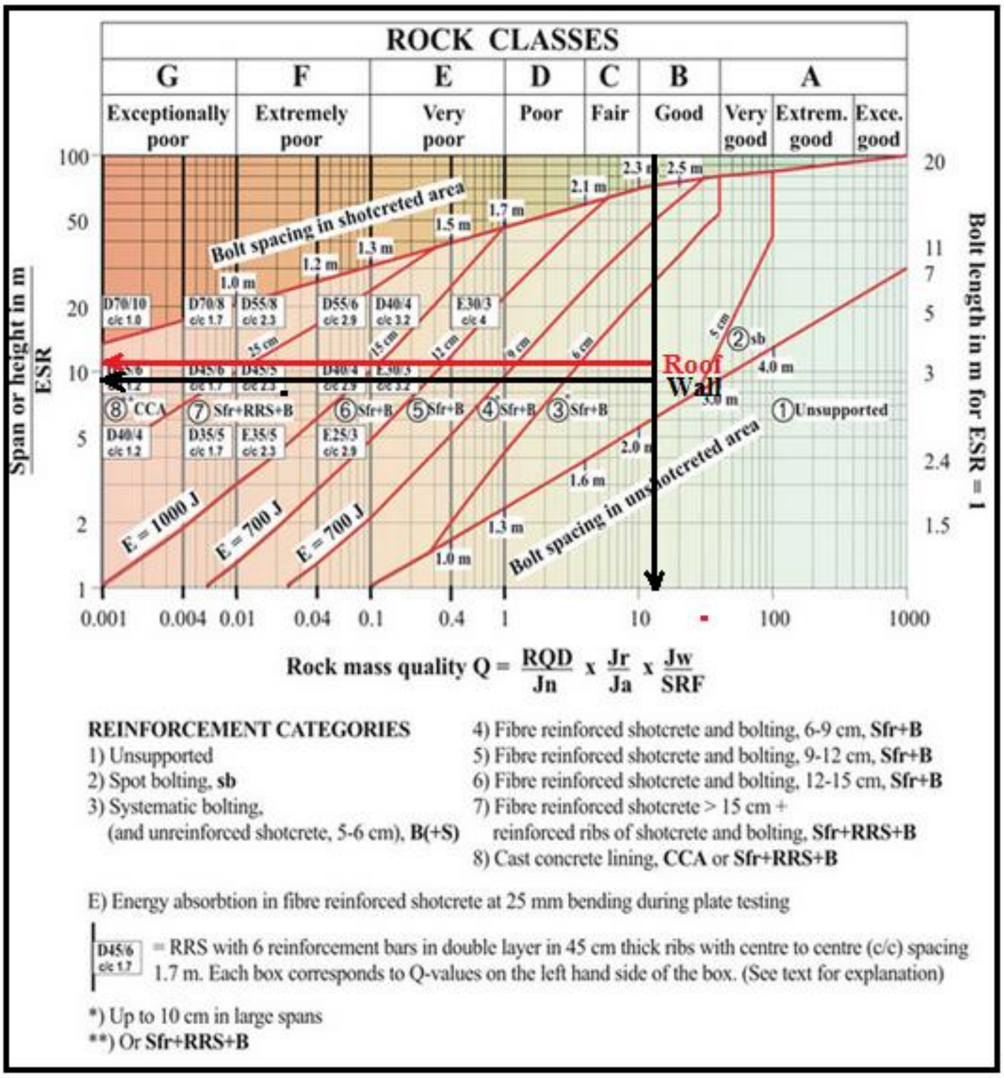

Figure 6-3 : Rock support chart according to Q-system for settling basin cavern (Alternative II) (Grimstad et al., 2002).

6.2.3 Rock support estimation by NGI Method

According to Norwegian Geotechnical Institute (NGI), Permanent support pressure required for rock mass is given by following relation.

$$P_{roof} = \frac{2\sqrt{J_n}\, Q^{-\frac{1}{3}}}{3J_r}$$

For the settling basin area of SMHEP, rock mass quality (Q-Value) lies between 10 to 14.For average value of 12, Required permanent support pressure is equal to;

$$P_{roof} = \frac{2\sqrt{9}\ * 12^{-\frac{1}{3}}}{3 * 3} = 0.29\ \ Kg/cm^2 = 29\, KN/m^2$$

Stability of opening is largely depends on the geometry of the opening. Minimum length of support (L) of rock bolt is estimated from the excavation width (B) and Excavation support ratio (ESR) as follows:

$$L = (2 + 0.15B)/ESR$$

Therefore for Alternative I, B = 8.4 m and calculated length of rock support becomes 2.1 m.

Whereas for Alternative II, B = 18.3 m and calculated length of rock support equals 2.97 m

Maximum Unsupported span is calculated by using the following formula;

Unsupported span = 2 × ESR × $Q^{0.4}$ = 8.64 m.

According to this method, However there is not required any support system for alternative I to be stable itself, for safety and practical purpose, few support system is needed.

6.3 Rock Bolts:

Rock bolt increases the degree of confinement of the opening and eventually increases the stability of the opening. Both classification system (RMR and Q) have suggested for required length, diameter and spacing of rock bolts through support charts/tables.
(Schach(1979)) suggested a relation to estimate the length of rock bolt and are often used;

$$L_b = 1.4 + 0.184\, D_t$$

Where,
L_b is the length of bolt in m

D_t Is the diameter or span of opening in meter (m)

But in ideal case, rock bolt should be designed based on rock mass conditions, especially average block size of the actual location. Palmstrom (2000) suggested the following relation for estimation of length of bolt for both wall and roof individually.

$$Lb_{roof} = 1.4 + 0.16\, D_t\, (1 + \frac{0.1}{D_b})$$

$$Lb_{wall} = 1.4 + 0.08(D_t + 0.5\, W_t)(1 + \frac{0.1}{D_b})$$

Where,
W_t = Height of tunnel wall
D_b =Block diameter
D_t =Diameter of span

For SMHEP, Calculated bolt lengths using the relation suggested by Schach (1979) are approximately 3m and 5m for alternative I and alternative II respectively. Rock bolt analysis may also be possible to be done for wedge stability but it requires the known direction of discontinuities and identification of potential wedge failure (Mao Dawei, 2005)

6.4 Conclusion and recommendation

Comparative studies of RMR, Q-system, and NGI methods for rock support estimation have been carried out. All the approaches have some uniqueness, such as RMR value gives the idea for stand up time whereas Q-system does not. Since Q-system consider the more parameters (Including Strength reduction factor) and have better support chart for estimation, it is highly recommended to use Q-system for estimation of permanent rock support. To know standup time to aware, how quick the support system have to install, short overview of RMR system is also recommended.

Different combination or rock bolts and lining for wall and roof have been estimated. For alternative I, wall is the critical part, which needs better support system than roof whereas for alternative II, the case is reversed due to longer span. Selection or rock bolt and lining type are also depending upon their market availability in the vicinity area.

For rock masses with swelling and squeezing ground, none of the rock mass classification system works well. RMR system does not consider the stress parameters, which is vital. For more accurate and optimum solution to estimate the rock support system, numerical analysis technique is suggested and is discussed in chapter seven. For the analysis through numerical approach, initial values of rock supports for the simulation are needed. In this regard, Suggested values of rock supports from the Q-system would be better choice and are used. Various combination of lining (shotcret, Concrete, RCC) and Bolts are analyze through numerical analysis and have eventually estimated the optimum values of support combinations which provides the required degree of safety and have relatively least cost. In numerical analysis, this requirement is achieved by reducing the maximum number of yielded elements.

CHAPTER 7

7 NUMERICAL ANALYSIS OF SETTLING BASIN CAVERNN

7.1 Introduction

Rock mass in numerical modeling is discretized into a large number of individual elements and are analyzed for rock stresses and deformation (Nilsen and Palmmstrom, 2000).
Basic Numerical modeling applied in rock mechanics problems are as follows

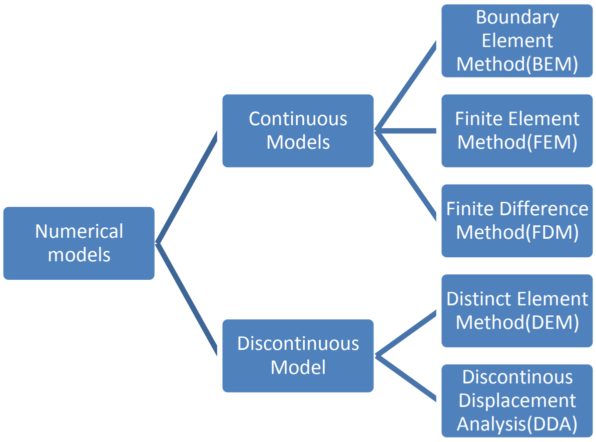

Figure 7-1 : General Classification of Numerical Methods (After Nilsen and Palmstrom, 2000)

Beside above methods, some useful coupled modeling methods are as follows:

Coupled Modeling:

- FEM+BEM
- DEM+BEM
- DDA+FEM

7.2 Finite element Method ($PHASE^2$)

It is a numerical technique for finding approximate solutions of partial differential equation as well as integral equations. This approach is based on eliminating the differential equation or rendering the partial differential equations into an approximating system of ordinary differential equation, which are numerically, integrated using standard techniques. This is the most popular method of numerical modeling, which is best suited for solving anisotropic, discontinuous and heterogeneous or non-linear rock masses. It consists of mainly three steps, domain discretization, local approximation, assemblage, and solution of global matrix equation. It involves the representation of continuum as an assembly of elements which are connected at discrete points ‘Node`. Problem domain in divided into discrete and displacements for the problem domain (source: http://w3.civil.uwa.edu.au). It Is the two dimensional elastic-plastic finite element program for estimating the stress and displacement around the underground openings, and can be used for solving complex geotechnical, rock engineering, mining and civil engineering problems for any time of earth materials (Source: $PHASE^2$ Reference Manual Version 8.0)

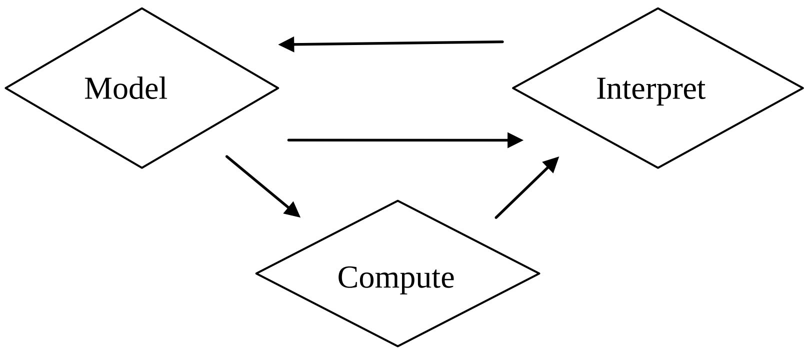

Figure 7-2 : Inter-relationship between three different modules in $PHASE^2$ program.

This program consists of three different independent program modules. They are inter-related with each other in a systematic way as shown in Figure 7.2.Compute and interpret can be started from within the model

Assumptions

In Plane strain analysis: It uses a plane strain analysis where two principle in-situ stresses (σ_1 and σ_3) are in the plane of excavation whereas the third principle stress (σ_z) is out of plane. Plain strain assumes that the excavations are of infinite length normal to the plane section of analysis. In this analysis, In-plane displacement and strains are calculated whereas out plane strain and displacements are assumed as zero.

In Axis-symmetric analysis: It is the 3-dimensional analysis for excavation which are rotationally symmetric about an axis (for example, end of a circular tunnel).Although the input is 2- dimensional, the analysis results can apply to the 3-dimensional problem. Both type of analysis are possible through powerful software$PHASE^2$.

7.2.1 Different Modules Used In $PHASE^2$

Model Generation

It is the pre-processing module for entering and editing the model, where the following tasks have been performed in a sequential way.

Project setting: The solver type determines how compute solves the matrix in the system of equations. Methods of solver used in $PHASE^2$ are as follows:

1. Gaussian elimination
2. Conjugate gradient iteration
3. Pre-conditioned, conjugate Gradient Iteration.

Gaussian elimination is the default method in $PHASE^2$ but for solving large problems conjugate iteration methods is best suited. For elastic type, material conjugate gradient techniques are suitable. Different units such as metric and imperial are possible to select. Metric, stress as MPa has been used in this analysis. Model can be generated in different stages of excavation based on requirement. Large cavern is needed large number of stages whereas for small cavern few stages are sufficient. It can also edit stages afterwards.

Entering and editing Boundary conditions: There are seven-boundary type, which can be generate and edit as per need. They are excavation, external, material, stage, joint, structural interface and piezometric line. Four first are mostly useful in our analysis. Excavation boundary is formed by closed polylines. In case of staged excavations, an excavation boundary represents the maximum extent of an excavation. Intermediate boundaries within the excavation being represents by stage or material boundaries. Material boundaries may be open or closed polylines that are used to define boundaries between the different material types. External boundaries define the extents of finite elements mesh that encompasses all other boundaries. In our analysis, external boundary is drawn with scale factor of five.

Meshing and discretization: After defining all boundaries, the finite element mesh can be created. Finite element mesh generator generates the meshes either triangular or quadrilateral finite elements. Meshing and discretization option can be used either individually or together.

Loading the field stresses: In $PHASE^2$, field stress can be gravity or constant. `Gravity field stress option` is used to define a gravity load, which varies linearly with depth from user defined ground surface. It is useful for surface or near surface excavations. For settling basin cavern of SMHEP, this option has been used to evaluate the principle stresses from valley model. This is needed for further confine analysis. Total stress ratio used in the model does not consider of locked horizontal stress (Tectonic stresses).It can also used the effective stress ratio where effect of pore water is considered and would be more accurate, if data are available.

Load split option is used to split the field stress (and /or body force) during Loading of the finite elements between any stages of the model, rather than applying the whole load in a first stage. It allows to gradual applying of the field stress load as excavation progresses. Load split determines how the field stress load is applied to the solid elements of the finite element mesh. Load split does not directly determine the load that is applied to the support elements. Support elements will only take the load, if deformation of finite element mesh occurs.

Defining and assigning the Material Properties

Physical and hydraulic properties of rock mass are the input parameters for this option. There are two steps to specify the material properties for the model are defining the properties and assigning the properties to the various regions of the model. Material physical properties such as unit weight, strength, and elastic parameters are the input to define and assign. First of all name and color of the material has to be chosen. As a initial element loading there are four options to select in different conditions which are field stress only, field stress & body force, body force only and None. Field stress loading, is of two types either gravity field stress or constant field stress which can select in option ``Loading`` in model. `Gravity field stress` option is used to define an in-situ stress field, which varies with depth. Gravity field stress is generally used for surface or near surface excavations. In cavern analysis of SMHEP, ``Gravity field stress`` option is used to evaluate the in plane principle stress (σ_1 and σ_3), its direction and out plane major principle stress (σ_z).Later, these calculated principle stresses for a particular point inside ground is assumed as a constant field stresses in a confined model. Unit weight of rock mass is used in case of gravity field stress

Four elastic models are used to define the material elastic properties, which are Isotropic, Transversely Isotropic, Orthotropic, and Duncan-Chang Hyperbolic. In isotropic material (Rock mass in settling basin area of SMHEP), material properties such as young`s modulus and poison’s ratio do not vary with the direction but in other material type those parameters are varied in different directions. The option- young`s modulus- inside the elastic properties dialog box is the modulus of elasticity of rock mass (not the intact rock).Residual value of young`s modulus is only useful when the material is assumed as plastic and the material is subjected to a change in load state (Unloading and reloading) after yielding.

In strength parameter dialog box, there are failure (strength) criterion for a material and type of material (elastic or plastic).If we choose the elastic material, failure criterion parameters such as m_b,s,a,σ_{ci} are only useful to calculate and plot the strength factor(strength/stress)within the material. Materials will not fail in elastic assumption because it allows more deformations and stresses then the actual case. That means failure envelope allows a degree of overstress. Elastic analysis is used only to find the strength factor and to know whether the rock mass will fail in plastic condition or not. If the strength factor in elastic analysis is less than 1 then it is assumed that the rock mass will fail in actual case. In real case, the rock mass is not completely elastic and hence in $PHASE^2$ it is assumed as elastic plastic material. It can be proved from the computed value of strength factor (never goes below one) of rock mass after plastic analysis. Strength factor equal to one after the plastic analysis signifies that the rock mass fails in that particular strength and stresses conditions. If the strength factor is very close to one then it is necessary to provide support system until it increases to a factor of safety that will need for the design of underground opening. During analysis, other parameters such as GSI Value, material constant (m_i), disturbance factor (D), Young`s modulus of elasticity of intact rock (E_i) are taken either from empirical/analytical approach (see in chapter five) or directly from the roc data or roc lab.

There are many failure criterions in the options box; all have their own scopes and limitation. Selection of failure criterion is depends upon the available input parameters, type of rock masses, accuracy, and scopes of each criterion(See Figure 5.3).Mohr-Columb failure criterion is best suited for more cohesive rock mass which requires the input parameter such as cohesion, friction angle, tensile strength and dilation parameter(if the material is plastic). For SMHEP, Rock mass is best suited to use Hoek-Brown Failure Criterion and available parameters are useful to use this criterion. Input parameters for Hoek and Brown Failure Criterion are evaluated in chapter five. In$PHASE^2$, most of the parameters can directly take from rocdata, roclab, or using GSI calculator. For good to reasonable quality rock mass Hoek-Brown failure criterion is best suited where the parameter a is taken as 0.5 directly. But for lesser quality of rock mass, General Hoek & Brown can be used. In this analysis I have used the General Hoek & Brown Failure Criterion because of availability of `a` parameters. Dilation parameter defines the increase in volume of rock mass due to shear. For plastic analysis residual parameters are taken as same as peak value because the rock mass is assumed as elastic-plastic. After defining the material properties, it is compulsory to assign it in respective stages.

Installation of Support system: Final stage of model generation in $PHASE^2$ is to define the support system. Requirement of rock support will be know after the analysis of opening without support conditions. If the rock mass fails in absence of support we have to installed it in different stages. Optimum selection of Type, quality and degree of support system is based on the safety requirements, available budget, and the expertise of designer. In $PHASE^2$, two different type of rock support is used which are bolts and liner. Bolts are of different types such as fully bonded, end anchorage, swallex, plain stranded cable, and tie/back. Bolts are generally installed either in a random spot or in a systematic pattern. Liner are also different types such as single layer liner, multiple layer liner (composite liner) and as model piles, geo-synthetics etc. Different properties of liner that are used in this analysis are presented in Table 7.2. In practice, generally, shotcrete is installed first and then rock bolt (Panthi 2012). Rock support systems that are suggested from the empirical approach (Q-System) are used for the initial input parameter for the simulation of rock support system.

Assign Properties: After defining the each material property, staging sequence and rock support of the excavation, it is necessary to assign those properties using assign properties option from properties toolbar.

After generation of the model, it is necessary to Save the generated model as ``*.fez `` extension file before compute. Main sequences that are used in the software to generate the model and run for analysis are described above but still some other parameters are there to be addressed. Detail description of each parameters are describe in ` $PHASE^2$ 8.0 Program: Reference Manual`, Rocscience Inc. (2008).

Compute

After generation of model, analysis and computation of model is done before interpretation. The model that has generated must be saved in a (*.fez) file extension before computation.

Executable files to compute engines are found in phase2 installation folder and have the file name as`` feawin.exe or .feawin_seq.exe``.

Phase2 automatically store all input and output files for a model in compressed file format. After analysis it stores in a single compressed (zip) file with extension *.fez. Detail description of this module are describe in ` $PHASE^2$ 8.0 Program: Reference Manual`, Rocscience Inc. (2008).

Interpret

This module is use to view the result of analysis after the computation. Interpret allows to contour analysis results on the model such as strength factor, Total/horizontal/vertical displacements, major and minor principle stresses etc.

Results in a various interpretation way such as graph, line and charts. We can also do the load analysis from the interpretation module such as how much load is taking by which bolts and liner in different stages. Other several way of visualizing the output can be found in this module, which are describe in ` $PHASE^2$ 8.0 Program: Reference Manual`, Rocscience Inc.(2008).

7.2.2 Stability analysis of settling basin cavern

7.2.2.1 Model description

Rock mass in whole the cavern area is banded gneiss of relatively high strength. Location of cavern is chosen based on topographical and geological consideration. To make analysis simpler, two most feasible alternatives are selected for further analysis in this report. In both alternatives, caverns are excavated in three consecutive stages as heading, middle bench, and final bottom except for section C-C. To find the optimum support system, each excavation stages are analyzed with different support combination. Three critical sections (A-A, B-B, and C-C) are chosen based on overburden depth and opening size/shape of cavern. A-A is the inlet section of settling basin, B-B is the middle section and C-C is the outlet section of the settling basin cavern

<u>Alternative I:</u>

This alternative layout is proposed during the feasibility study of the project (Himal Hydro 2009).Two caverns are put together in parallel to each other. Average overburden for the cavern section is varying from 92 m to 272 meter. The alignment of the cavern is fixed at N145E after stereographic and joint rosette analysis (Refer Chapter 4). In plane and out plane tectonic stresses are taken as 1.29 MPa and 5.93 MPa respectively (Refer Chapter 5). Cross-sectional dimensions (Shape and size) of the caverns for all section are shown in Appendix C whereas the plan layout is shown in Figure 7-3.

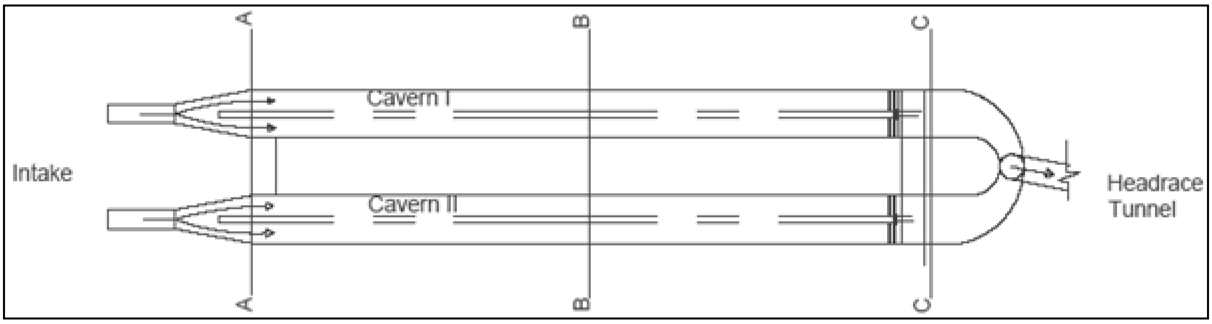

Figure 7-3 : Plan Layout of settling basin cavern for alternative I, Showing different sections

<u>Section A-A (Chainage 0+057)</u>

This is the inlet section of the cavern. Cross-sectional dimensions for both the caverns in this section are similar. This section is considered as critical because it has lowest overburden with relatively bigger size of cavern. Net lowest overburden is 92m with ground elevation of 1452 m.

<u>Section B-B (Chainage 0+142m)</u>

This is the middle section of the cavern.Net overburden in this section is 218m with ground elevation of 1577m.Other parameters for the analysis are same as section A-A except dimensions of cavern.

<u>Section C-C (Chainage 0+218m)</u>

This is the end section of the settling basin cavern, which has the largest opening. Inlet of the Pressurized Headrace tunnel is just after this section.Net maximum overburden is 272 m with ground elevation of 1630 m.

<u>Alternative II (Proposed alternative)</u>

This is the proposed alternative for the analysis. Location and orientation of proposed alternative is similar to existing alternative. All the three sections are taken similar to alternative I. Major difference in alternative II and alternative I is the cross-sectional geometry of the cavern. Simple layout plan, which shows the three distinct sections, has been presented in Figure 7.4. For both alternatives, Centre line for the cavern is taken as the base line for measuring the net overburden.

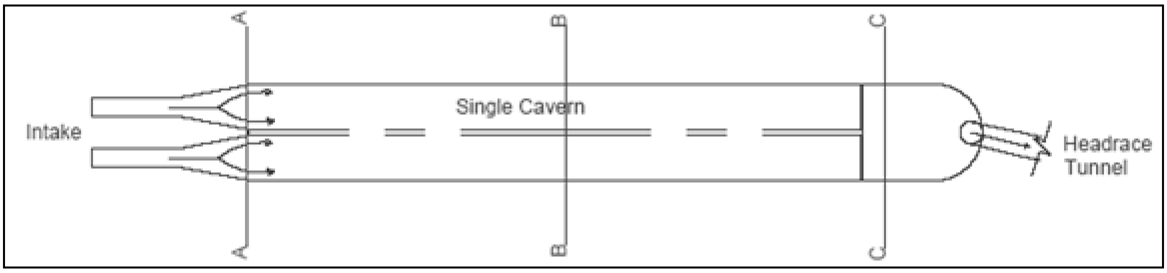

Figure 7-4 : Plan Layout of settling basin cavern for alternative II with distinct sections

7.2.2.2 Estimation of virgin/In-situ stresses

Valley model is generated to estimate the magnitude and direction of virgin stresses (stresses prior to excavation) and have discussed below.

<u>Approach of Analysis</u>

Elastic-plastic analysis of confined zone needs in-situ principle stresses at centroid (Geometrical centre) of cross-section for each cavern. Since the actual location of cavern is near the valley side of the river, consideration of anisotropy in stresses due to topography is must. To evaluate the magnitude and direction of actual principle stresses (both in plane and out plane) at particular depth of each section, we need to generate the valley model in $PHASE^2$.After generation of valley model for each section, simulation has done with input parameter-field stresses (as gravity stress). Ratio of total horizontal and vertical stresses in both plane (In plane and Out plane) is taken as same without considering the effect pore water pressure.

Virgin stresses are determined at three critical points (Centroid) of each section beneath the ground and are represented by sections A-A, B-B, and C-C. Calculated virgin stresses are useful for both alternatives.

Input parameters for the analysis:

Material Input Parameters

Material input parameter describes the properties of rock mass. They are needed for stability analysis of any type of underground structure. Some analytical and empirical approaches are used to determine the material input parameters (Refer Chapter 5). Parameters, which are calculated from those empirical and analytical approaches, would be input parameters for the GSI calculator or for Roc data software. GSI calculator and Roc data software are the tools that have direct linked to the ($PHASE^2$) program. It is also possible for manual input of all parameters based on suitable criterion. Since the rock mass in settling basin area of SMHEP is elastic-brittle-plastic, Generalized Hoek-Brown criterion is better than other criterion and have used in all numerical analysis in this report. Besides few parameters, all the parameters needed for both approaches (elastic and plastic) through ``Generalized Hoek-Brown criterion`` are similar and presented in Table 7.1.

Table 7-1 : Input parameters for material (Rock Mass) used in the analysis.

Sections	A-A	B-B	C-C
Rock Type	Banded Gneiss	Banded Gneiss	Banded Gneiss
Uni-axial compressive strength (MPa)	78	78	78
Poisson's ratio	0.2	0.2	0.2
Young's Modulus of Elasticity (GPa)	27	27	27
Modulus of Elasticity of Rock Mass(MPa)	7050	7050	7050
Unit Weight ($\mathrm{MN/m^3}$)	0.027	0.027	0.027
Hoek & Brown Constant ($\mathrm{M_i}$)	28	28	28
Geological Strength Index(GSI)	60	60	60
Disturbance Factor(D)	0.5	0.5	0.5

Field stress parameters:

For isotropic rock mass where we do not consider the tectonic stresses, ratio of total horizontal stress and total vertical stress for both in plane and out plane is taken as same because of distressing of stresses at shallow dept (In gravity stress analysis) .In that case, Vertical stress is completely linearly varying with the depth. In shallow depth, Initial element loading would be `` field stress and body force`` whereas the material type is assumed as elastic. Total horizontal stress is the summation of stresses caused by tectonic and gravity effect. For this calculation stress ration for both in plane and out plane is kept equal, which is calculated based on value of poison ration.

Valley Model`s Result, Discussion

Computed values from the valley models are useful to analyze the confine model in next stage while doing stability analysis along with rock support estimation. Results from the valley model for different cross-section are discussed below.

Section A-A (Chainage 0+057)

After generation and computation of valley model, output comes through interpret module. Stresses values prior to excavation are computed. Analysis gives the following results.

Major in plane principle stress (σ_1) = 2.8 MPa

Minor In plane principle stress (σ_3) = 1.35 MPa

Major out plane principle stress (σ_z) = 6.56 MPa

Orientation of in plane major principle stress (σ_1) : 109° (CCW from horizontal axis))

Kirsch`s gives a relationship between the principle virgin stresses and the maximum/minimum tangential stresses that may occur in the periphery of the circular opening. It gives the tentative idea of failure mode of tunnel and extent of rock support requirement prior to further extensive analysis. Linear relationship suggested by Kirsch`s are as follows:

$\sigma_{tmax} = 3\sigma_1 - \sigma_3$

$\sigma_{tmin} = 3\sigma_3 - \sigma_1$

By using his approach, Computed maximum and minimum tangential stresses are 7.05 MPa and 1.55MPa respectively. Rock mass strength suggested by Panthi (2006) gives 11.5MPa.This means any circular opening in that particular location is simply safe but in case of SMHEP, the opening is not exactly circular. Therefore, we cannot say the opening is safe without doing further analysis. This limitation of Kirsch`s equation will solve by doing numerical analysis with exact simulation for particular shape and size of the opening. Valley models that are generated to evaluate the principle stresses and their direction are presented in Figure 7.5 to 7.7

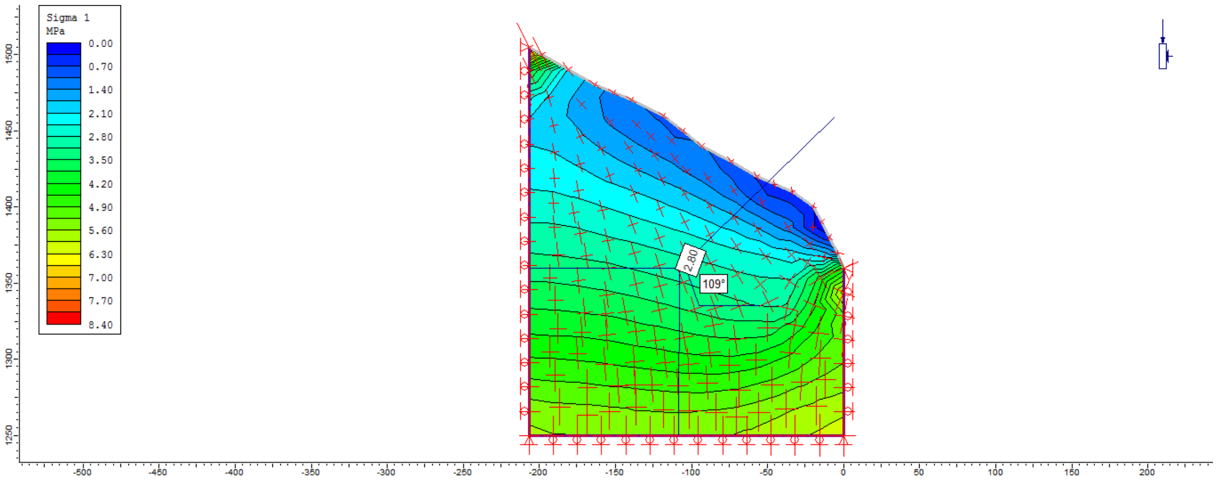

Figure 7-5 : Valley model showing σ_1 and its direction-CCW (Section (A-A)

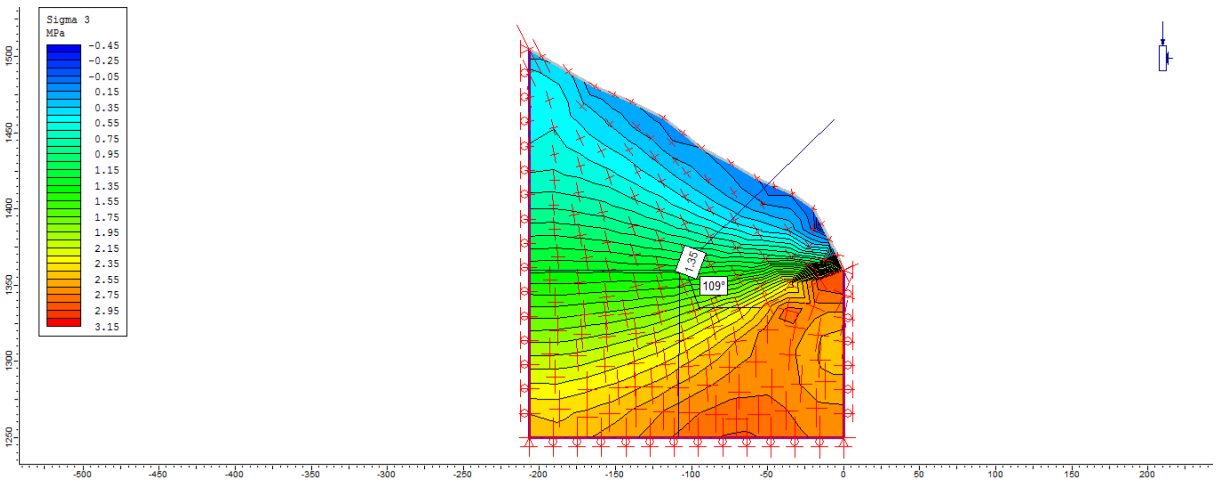

Figure 7-6 : Valley model showing the minor in plane principle stress -σ_3, (Section (A-A)

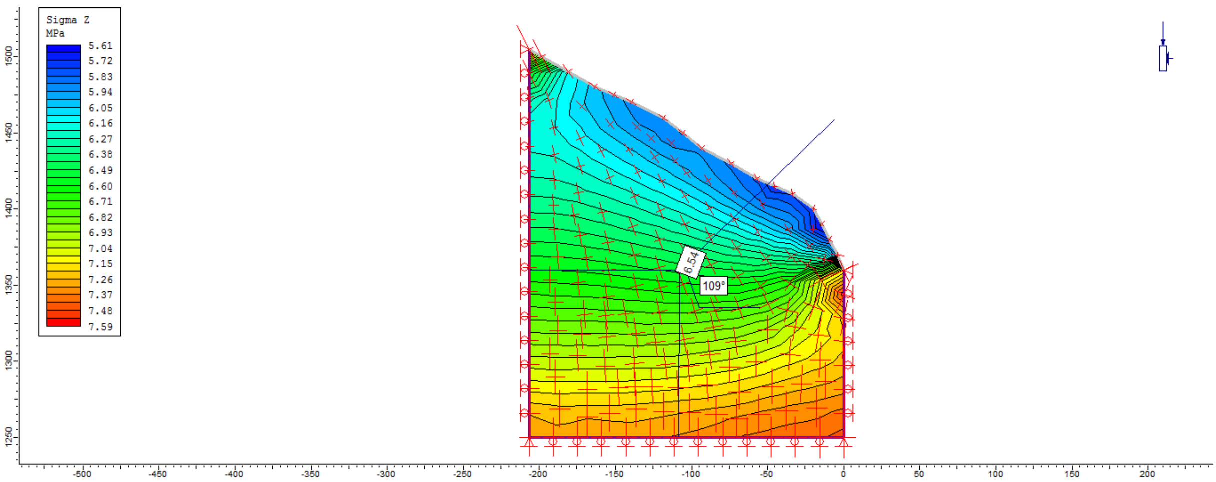

Figure 7-7 : Valley model showing the Major out plane principle stress-σ_z, (Section (A-A)

Section B-B (Chainage 0+142m)

Valley slope model at chainage 0+142 m chainage is shown in figure below with value of major principle stress and its orientation. These computed values are the in-situ stresses, where topographic effect is also considered. Analysis shows the followings results

Major in plane principle stress (σ_1) = 6.0 MPa

Minor In plane principle stress (σ_3) = 1.75 MPa

Major out plane principle stress (σ_z) =7.2 MPa

Orientation of in plane major principle stress (σ_1): 100° (CCW)

Maximum and minimum tangential stresses calculated using the Kirsch equation are 16.25 MPa and -0.75 MPa. Negative sign indicates that there is tensile stress as well and may have tensile failure, if this value exceed the tensile strength of rock masses. Valley models that are generated to evaluate the principle stresses and their direction are presented in Figure 7.8 to 7.10.

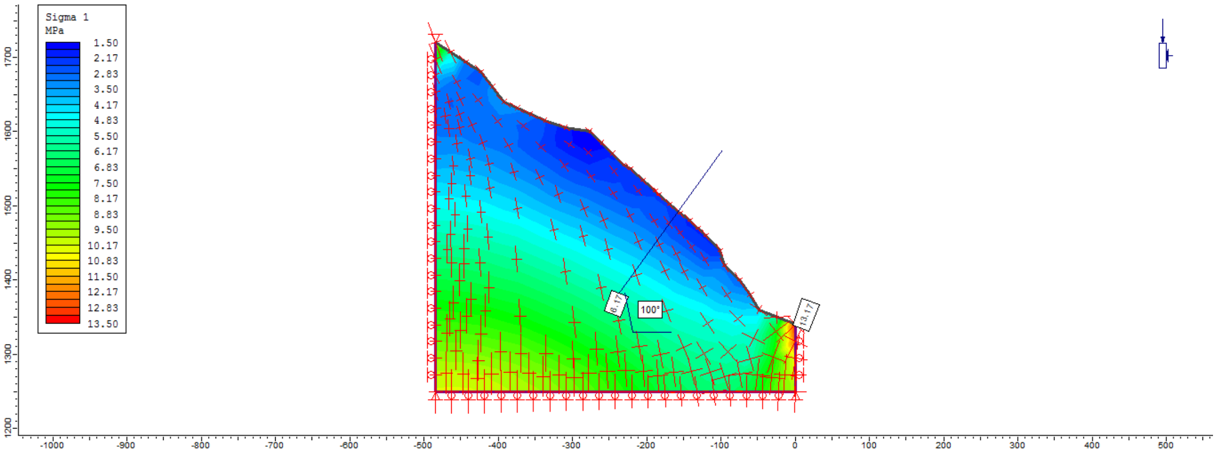

Figure 7-8 : Valley model showing,σ_1, and its direction-CCW (Section (B-B)

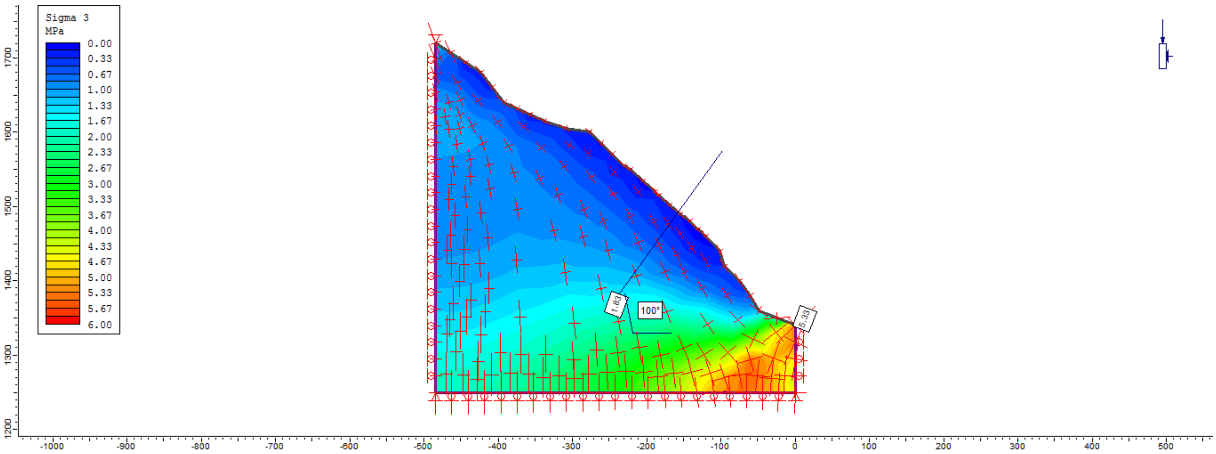

Figure 7-9 : Valley model showing the,σ_3, (Section (B-B))

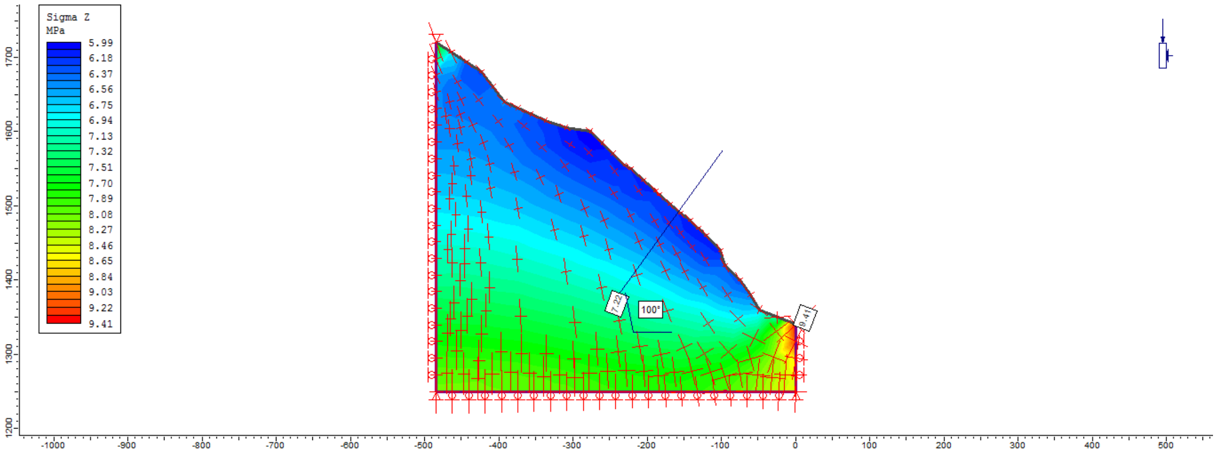

Figure 7-10Valley model showing,σ_z, (Section (B-B))

Section C-C (Chainage 0+218m)

Valley slope models at chainage of 0+218m are shown in Figure below with value of principle stresses and its orientation. These computed values of stresses are virgin stress where topographic effect is also considered. Analysis shows the followings results

Major in plane principle stresses (σ_1) = 8.0 MPa

Minor In plane principle stress (σ_3) = 2.4 MPa

Major out plane principle stress (σ_z) =7.8 MPa

Orientation of (σ_1): 96° (CCW)

Maximum and minimum values of tangential stresses from Kirsch equation are 21.6 MPa and -0.8 MPa. Negative sign indicates that there is also tensile stress. Tensile failure may also occur in this section, if these values exceed the tensile strength of rock masses. For highly jointed rock mass, tensile strength is very less. Maximum value computed by Kirsch`s equation is very high compare to rock mass strength (Calculated by Panthi`s approach). Chances of failure are high and requirement of rock support is also higher than the section at A-A and B-B. Valley models that are generated to evaluate the principle stresses and their direction are presented in Figure 7.11 to 7.13.

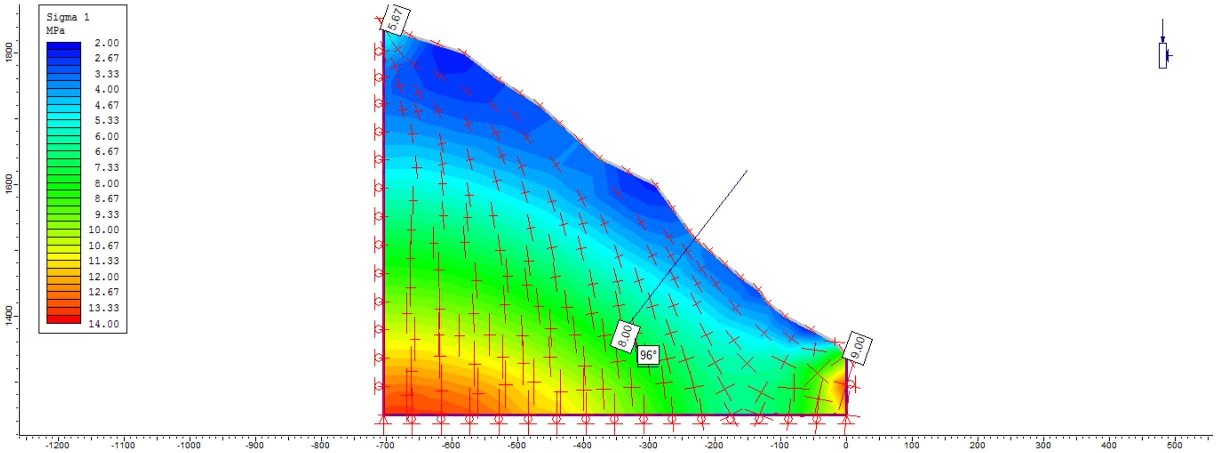

Figure 7-11 : Valley model showing (σ_1) and its direction (CCW) (Section C-C)

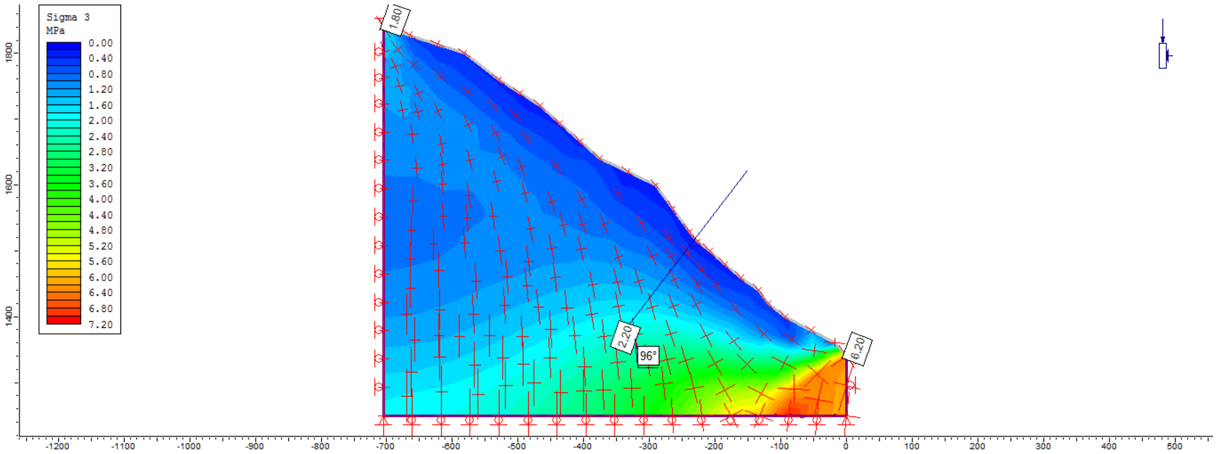

Figure 7-12 : Valley model showing the minor in plane principle stress (σ_3) (Section C-C)

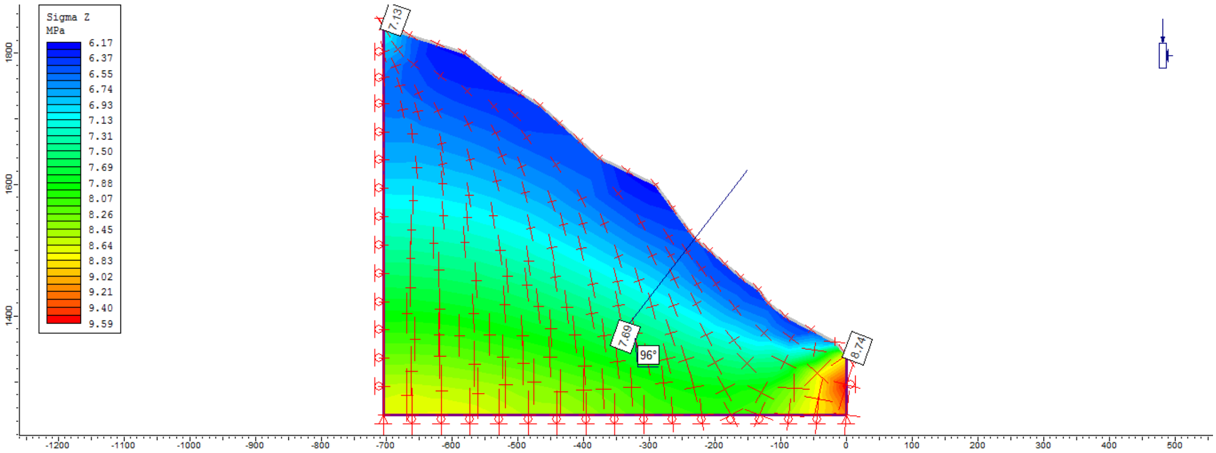

Figure 7-13 : Valley model showing the major out plane principle stress (σ_z) (Section C-C)

7.2.2.3 Evaluation of alternatives

7.2.2.3.1 Alternative I

This is the originally proposed alternative by the Himal hydro 2009 during feasibility study. Rock mass in this area is ``elastic-brittle-plastic`` - banded gneiss. Topography and geology are varying along and across the proposed alignment of cavern. Average overburden above the cavern is ranging from 92 m to 272 m. Alternative I has been analyzed in three different critical sections as described in section 7.2.2.1.

7.2.2.3.1.1 Input parameters

Parameters are the variables, which reflect the different properties of material, support systems, or external variable like field stress. Accurate calculation/estimation of input parameter determines the accuracy of result after the analysis. Therefore, precise determination of input parameters is a challenging task to all the engineer/designer. Input parameters, which are needed in numerical analysis through PHASE^{2}program, are discussed here. Material input parameter, field stress input parameter, and input parameters for rock support are the major input parameters and are discussed below.

Material input parameters

Material parameters for the banded gneiss in all the section are taken as same, which is described in table 7.1.Some other parameters that are needed for plastic analysis are discussed below.

Dilation parameter: It varies in elastic-plastic, brittle plastic, and perfectly plastic rock masses. Maximum practical value of dilation parameter for Hoek and Brown failure criterion suggested by (Hoek- Brown 2005) is as follows:

$$K_d^{assoc} = 1 + \frac{m}{2\sqrt{\frac{m\sigma_r}{\sigma_c}+s}}$$

Where,

K_d^{assoc} = 1, for non-dilating rock mass (fully elastic)

For elastic-brittle-plastic material, its value will be higher than one. In case of banded gneiss, Rock mass is assumed as elastic-brittle-plastic. Therefore, the value of dilation parameter is taken as half of the m_b parameter, which is almost equal to two.

If this assumption, in selecting the dilation parameter is wrong then obviously there will be error in calculating the displacement after simulation in the model. But the maximum error in displacement is only up to 5 % (Hoek- Brown 2005)

Residual Parameters: Since the rock mass is elastic-brittle-plastic, residual values are taken same as peak parameters (m_b, s and a).

Field stress input parameters

Result parameters from the valley models (Refer section 7.2.2.2.3) are the input stresses parameters for the confined model analysis (during stability analysis and rock support estimation). These values are generated by considering the topographic effect on stresses. Stresses are corresponding to the centroid of the cavern in each cross-section. To make calculation easier, these virgin stresses values at centroid are assumed as equal in all points around the centroid of cross-section of cavern (But in actual case, stresses are different in each point even within the cavern itself). These field stress parameters (σ_1,σ_3,σ_z, θ) are used for both cases of elastic and plastic analysis

Support input parameter

Rock supports estimated by Q-system are used as an initial input parameters for the estimation of more reliable rock support through numerical analysis for simulation. Rock support system estimation by Q-method has done in chapter six. In numerical analysis, various combination of rock support (steel fiber reinforced shotcret, fully bonded rock bolts, rein-forced concrete) are installed to reduce the maximum number of yielded finite elements. Simulation is done until the yielded finite elements are lowest, and is considered as optimum point and corresponding rock support is called optimum rock support. Mechanical and geometrical properties of rock support used for all section of alternative I is summarized in Table 7.3

Table 7-2 : Properties of Reinforced concrete used in the analysis

Beam Formulation	**Timoshenko**	**Liner Type**	**Reinforced concrete**
Reinforcement Type	Wire Mesh Canada(D=4mm)	Concrete Type	
Spacing (m)	2	Thickness(m)	0.1
Section Depth(m)	0.004	Young`s Modulus(MPa)	30000
Area (m^2)	$1.26e^{-005}$	Poisson Ration	0.15
Moment of Inertia(m^4)	$1.2566e^{-011}$	Compressive Strength(MPa)	40
Young`s Modulus(MPa)	200000	Tensile Strength(MPa)	3
Poisson Ratio	0.25		
Compressive strength(MPa)	400		
Tensile strength(MPa)	400		

Table 7-3 : Properties of different rock support system used in the analysis (Alternative I)

Properties of Shotcrete				Properties of bolt			
Sections	A-A	B-B	C-C	Sections	A-A	B-B	C-C
Liner Type	Standard beam	Standard beam	Standard beam	Type	Fully Bonded	Fully Bonded	Fully Bonded
Beam Formulation	Timoshenk0	Timoshenko	Timoshenk	Length	3 m	4 m	4 m
Young`s Modulus	30,000 (MPa)	30,000 (MPa)	30,000 (MPa)	Diameter	20 mm	25mm	25 mm
Poison Ration	0.2	0.2	0.2	Spacing (In×Out)	2.2×2.2	2.2×2.2	1.5×1.5
Thickness	0.07 m	0.1 m	0.1 m	Bolt Modulus	200000 MPa	200000 MPa	200000 MPa
Peak UCS/Residual	35/5 MPa	35/5 MPa	35/5 MPa	Peak Tensile Capacity	0.1 MN	0.1 MN	0.1 MN
Peak Tensile/ Residual	5/0 Mpa	5/0 Mpa	5/0 Mpa	Residual Tensile Capacity	0.01 MN	0.01 MN	0.01 MN

7.2.2.3.1.2 Output parameters:

Interpret module of PHASE^2 gives the facility to visualize and analyze the result after the simulation. Some of them, which are mostly useful during this analysis, are discussed below.

<u>Principle stresses:</u>

Principle stresses are the stresses, which corresponds to the principle plane where shear stress is zero. Three principle stresses are acting at any point of interest, which are perpendicular to each other. They are major principle stress (σ_1), minor principle stress (σ_3) and intermediate principle stresses (σ_2).They are defined in such a way that the highest value of stresses is corresponding to major, intermediate value of stress is corresponding to intermediate and the lowest value is corresponding to the minor principle stress.

InPHASE^2, σ_1 is corresponds to the in plane major principle stress, σ_3 is the in plane minor principle stresses and σ_z is the out plane major principle stresses. In plane refers to the plane, which is perpendicular, the alignment of interest. Since PHASE^2 is two dimensional elastic-plastic problem solver software, It assumes that the third direction (perpendicular to the in plane area) is just an infinitely perpendicular projection from the in plane area.

Symbolically, the longer bar in the interpret module represent the relative magnitude and actual direction of major principle stress where as the shorter bar represents the relative magnitude and actual direction of minor principle stress. In 2-D analysis, the intermediate principle stress is the point at the centre of intersection of those two major and minor principle bars.

Value of these principle stresses determines how the rock mass behaves with the given opening is stresses. Comparative study of those principle stresses having different opening geometry may somehow help to predict the relative stability condition.

Strength Factor:

Generally, it is defined as the ration of strength of rock mass to the induced stress in a particular point of rock mass (element of mesh). In PHASE^2 program, rock mass overstressing is allowed in elastic assumption so the value of strength factor may be below one. In elastic analysis, strength factor less than one means the rock mass strength is less than induced stress and have chance of failure. On the other hand, in plastic analysis it never goes below one because rock mass in actual case is elastic-plastic and if the program developer allows to go strength factor below one, the result after the analysis of non-elastic rock mass would lead to error. Rock mass may fail, if the strength factor in plastic analysis becomes one.

Displacement:

It is distance covered by the element mesh due to unbalance stresses and strength of rock mass. PHASE^2 has altogether five options to show which type of displacement we need to look. They are horizontal displacement and its absolute value, vertical displacement and its absolute value and the total displacement. Total displacement is the square root of absolute horizontal and vertical displacement.

Total displacement= $\sqrt{\mathrm{H}^2 + \mathrm{V}^2}$

Yielded Elements:

PHASE^2 Discretizes the rock mass and create thousands of elements, each elements are considered as the smallest unit of consideration during computation. Beside those finite elements, liners, bolts, and joints are also considered as elements. Elements, which fail in shear and tension, are known as yielded elements. If we consider all the elements like bolt, joints, liners, and finite elements are elastic then they will not fail before reaching the elastic limit (which is too high in elastic assumption). Furthermore, in PHASE^2 , if we assume all those elements are elastic then they will never cross the yielding point. However, in plastic analysis, yielding can occur.

Symbolically, in interpretation module of software, elements which fails in shear is indicated by symbol × whereas yielded elements in tension is indicated by symbol o .If both occurs then the symbols will overlap to each other.

7.2.2.3.1.3 Modeling Result, Comparison and Discussion

Section A-A (Chainage 0+057)

Elastic analysis

From the elastic analysis of section A-A, It shows that the cavern may fail after stage III of excavation because the strength factors at stage III is less than one. Maximum total displacement is very low in all the stages even though it increases in each further stages of excavation. Rock mass is assumed as elastic-brittle-plastic in nature. Maximum displacement is also within the elastic range of highly brittle rock mass. Generally, Brittle rock mass fails in approximate strain of 0.02% (Panthi 2012).In this case, maximum total strain (After distributing the total maximum displacement around the periphery of the tunnel) is 0.012%.

Different output parameters after the simulation through PHASE^2 in different stages of excavation in elastic mode are shown in Table 7.3. Comparative analysis in different stages of excavation shows that most probable chance of failure is after the final excavation because of least value of strength factor (< 1) and higher value of displacement, However, there is not significant change in displacement in consecutive stages. Which shows that the rock mass is more or less stable. Maximum value of major principle stress is 10.8 MPa, which is lesser than the rock mass strength (computed through analytical and empirical approach-(Refer Chapter 5)).

Table 7-4 : Different values of output parameter from elastic analysis at section A-A

Stages	σ_1(MPa)	Strength Factor	Total Displacement(m)	Remarks
I	7.2	1	0.00084	Critical Condition
II	7.65	1	0.00104	Critical
III	10.8	0.98	0.00104	Chance of Failure

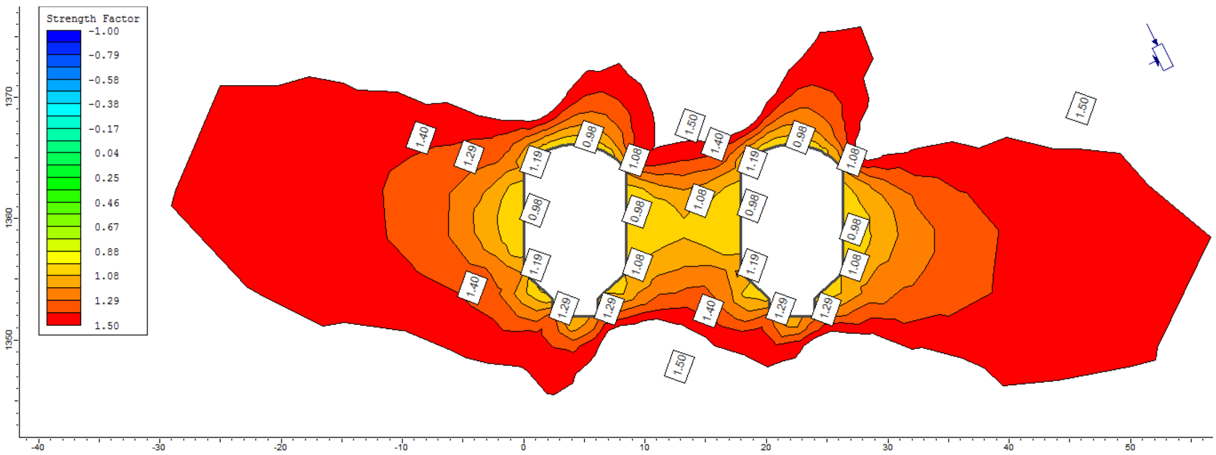

Figure 7-14 : Strength factor Diagram at section A-A (Elastic Analysis)

It is concluded from the above Table 7.4 and the figure 7.14, Maximum chance of failure is at crown and at centre of wall, if unsupported. Since the strength factor is less than one in some particular region, plastic analysis should be performed for rock support estimation to make the cavern stable.

Plastic analysis:

If we assume the rock mass is plastic materials, it also takes the load after the yielding. Results from the elastic and plastic analysis looks almost similar which has shown in table and figure below. While doing the plastic analysis some residual parameters are also needed to define and assign. Residual and peak values are taken as same because rock mass is assumed as elastic-plastic .Dilation parameter is taken as (M_b/2).It is selected after several trial values of dilation parameter in the analysis and consultation with previous reports and related articles

Plastic analysis without support:

Different output values from the analysis in different stages are shown in Table 7.5. While the figures are only presented for most critical stage (After final stage) to show exact region of distressing and failure (if occur) .Maximum total displacement is reduced from stage II to stage III. Yielded elements are almost constant as the excavations proceed in different stages. Strength factor is always greater than one except after final stages. Elements are not distressed a lot in all stages. Strength factor equal to one in plastic analysis suggest for further analysis to evaluate the extent of rock support. Rock support requirement would be very less but it is exactly finalized in next section. Figure 7.15 to 7.17 show the conditions of major principle stress, strength factor, and maximum total displacement after the final stage of excavation.

Table 7-5 : Different values of output parameter from plastic analysis at section A-A

Stages	σ_1(MPa)	Strength Factor (Strength/Stress)	Maximum Total Displacement(m)	Yielded elements	Remarks
I	9.9	1.04	0.0033	48	No support need
II	10.35	1.04	0.0037	48	No support need
III	10.4	1	0.00354	46	May need support

Figure below shows the value of major principle stresses, strength factor, and maximum total displacement at different elements/points in periphery of cavern. These figures are taken after finishing the final stage of excavation.

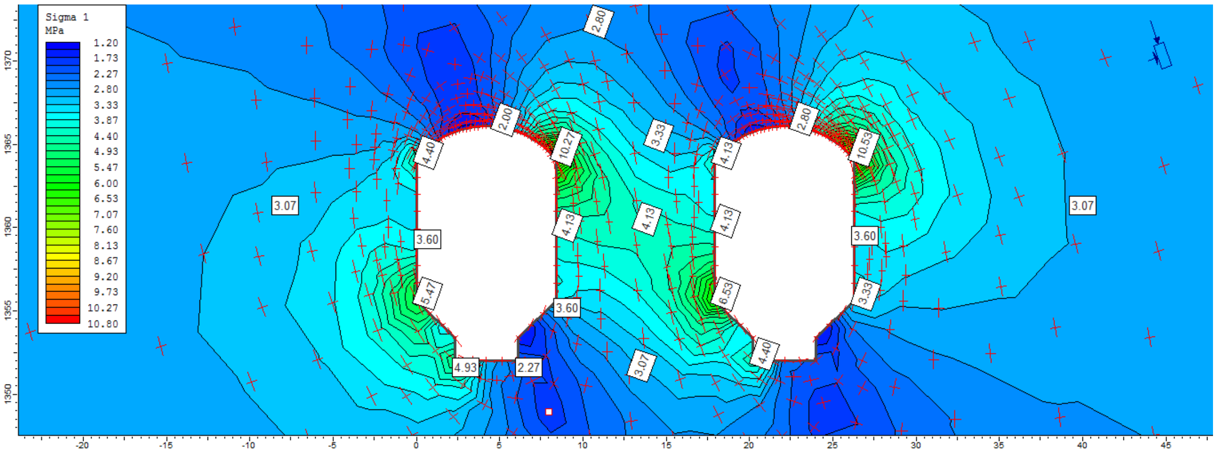

Figure 7-15 . Major Principle stress diagram at section A-A (Plastic analysis)

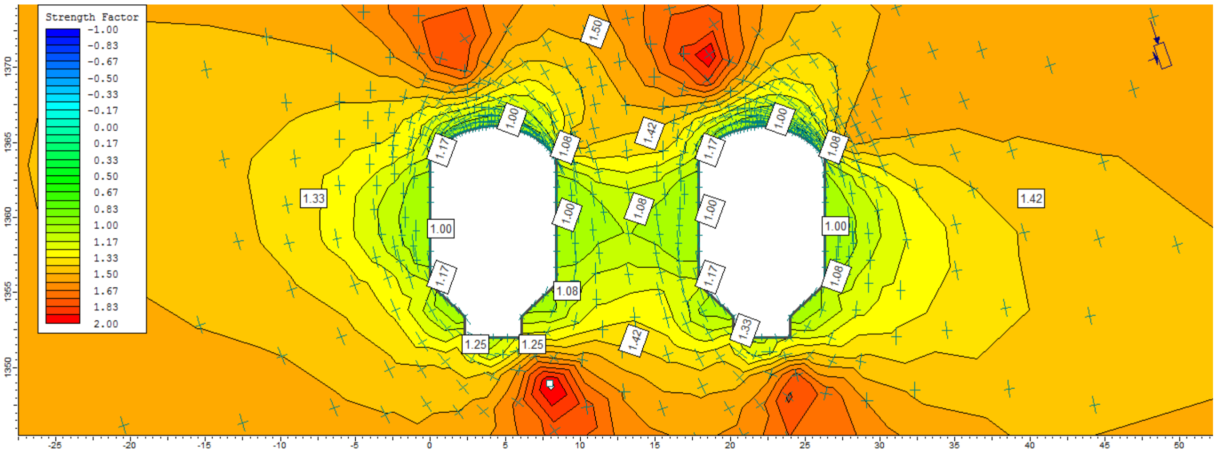

Figure 7-16 : Strength factor diagram at section A-A (Plastic analysis)

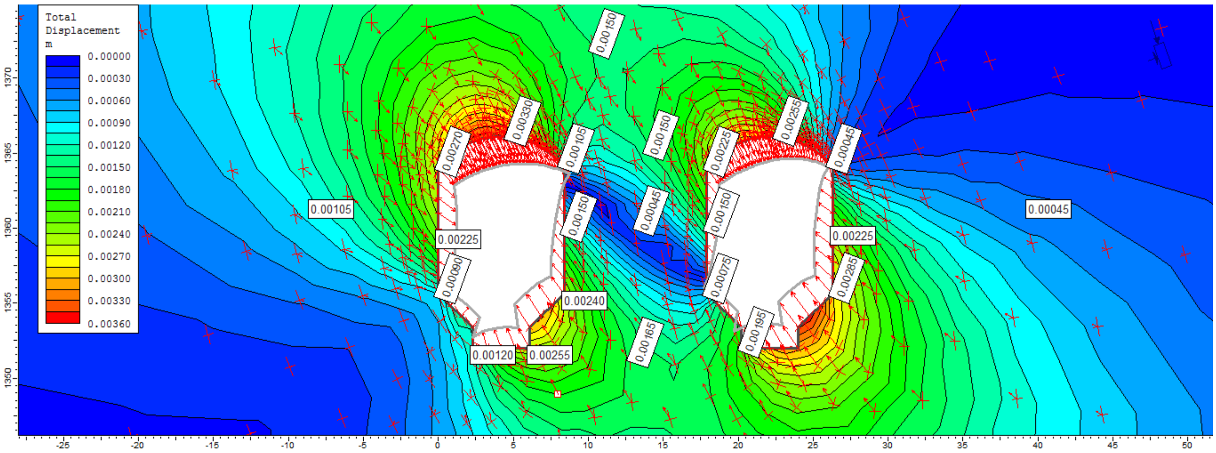

Figure 7-17 : Total displacement diagram at section A-A (Plastic analysis)

<u>Plastic analysis with rock support:</u>

Various combinations of rock support systems have been tried to increase the strength factor, to reduce deformation and to get the least number of yield finite elements, but it is hard to improve the quality of rock mass because of support systems. Excavation and installation of different rock supports are carried out in different stages. Properties of rock supports system that are used in the analysis are presented in Table 7.3.

Suggested Support systems from Q method are taken as initial input parameters for the simulation. Total numbers of yielded elements are presented in Table 7.6

Table 7-6 : Simulated values of different parameters in different stages. (Plastic analysis)

Stages	σ_1(MPa)	σ_3(MPa)	Strength Factor	Maximum Total Displacement (m)	Yielded elements	Yielded Shotcret/Bolts
I	9.9	0	1.04	0.0033	48	No
II	10.35	0.08	1.04	0.00373	47	4/No
III	10.4	0	1.04	0.00354	45	5/No

After comparing the Table 7.5 and Table 7.6, total numbers of yielded finite elements are reduced by one after installing support. Meanwhile, four shotcret elements are yielded. This means that support system are taking the load but not have major role to strengthen the rock mass system. Major principle stress and maximum total displacements are almost same in both cases (with support and without support) Slight improvements on strength factor is from one to 1.04.Results after the simulation with rock support installation as suggested by Q-system has shown in figure 7.18 to 7.20 and is recommended for implementation

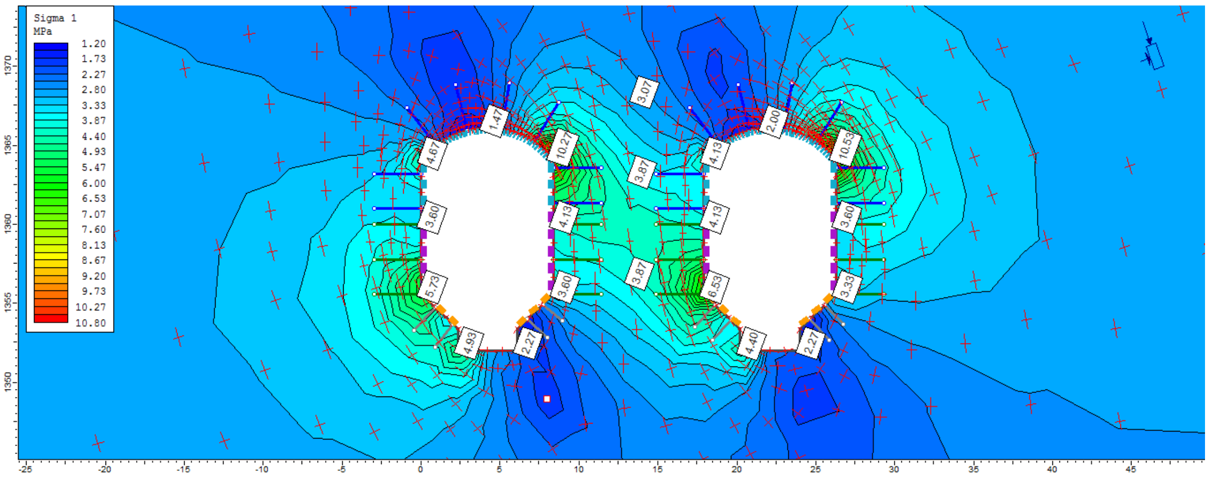

Figure 7-18 : Major principle stress diagram at section A-A (Plastic analysis)

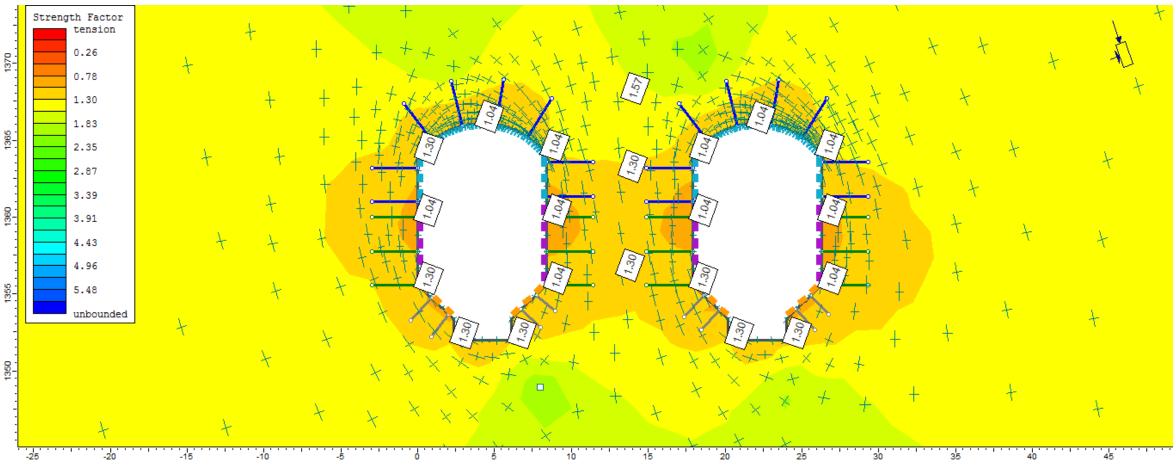

Figure 7-19 : Strength factor diagram at section A-A (Plastic analysis)

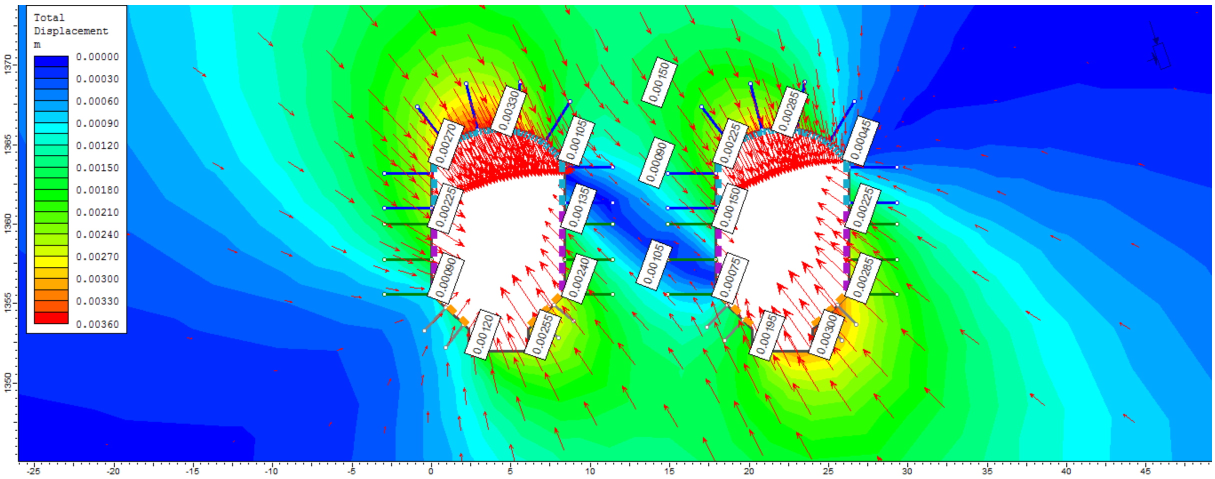

Figure 7-20 : Total displacement diagram at section A-A (Plastic analysis)

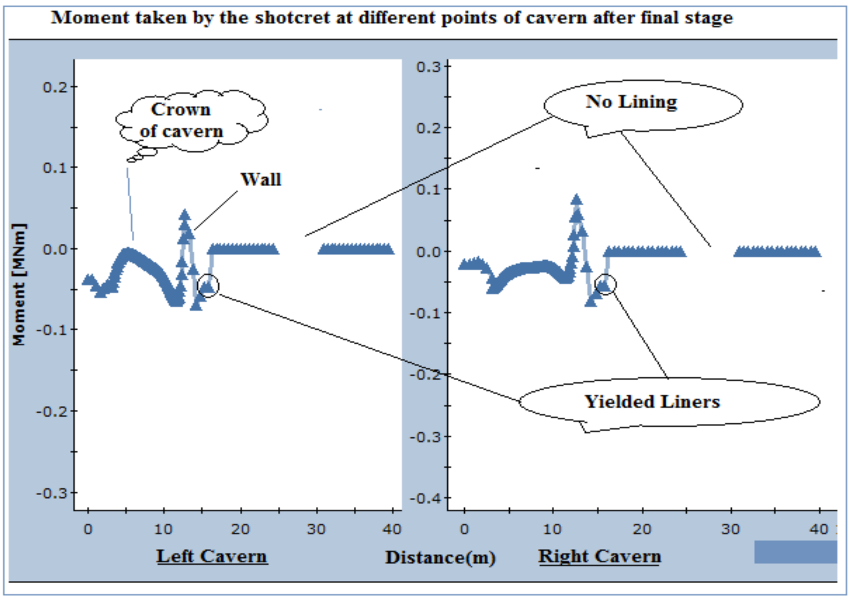

Figure 7-21 : Induced bending Moment at different Points of shotcrete around the cavern

Figure 7.21 shows that how the moment is taken by different part of shotcrete liner after complete excavation of the cavern. Yielded liner elements are the indication of functioning of shotcrete Negative bending moments at failure Shotcret element shows that there is tensile force at the wall of both caverns. It may be due to the vertical nature of major principle stresses as described by Selmer-Olsen and Broch, 1997(Refer Chapter 4).

<u>Section B-B (Chainage 0+142m)</u>

<u>Elastic analysis</u>

Elastic analysis is performed to plot the strength factor at different stages of excavation and to judge stability condition of cavern. Comparative analysis of Figure 7.22 and Figure 7.23 show that the cavern periphery is highly overstressed after first stages of excavation then final stage. It means, more support is required during intermediate stages of excavation than the final excavation. It may be due to better final shape and uniform stress distribution all along the periphery. Strength factor in stage I and stage II are less than one. Although the strength factor is slightly greater than one that is 1.04 after the final stage of excavation, it is recommended to do plastic analysis to estimate the required support system.

Table 7-7 ; Result parameters in different stages in elastic analysis (Section B-B)

Stages	σ_1(MPa)	Strength Factor (Strength/Stress)	Maximum Total Displacement(mm)	σ_3(MPa)	Remarks
I	23.05	0.69	0.0069	-1.75	Chance of failure(Tension crack)
II	18	0.69	0.008	0	Chance of failure
III	18	1.04	0.0075	0	Low chance of failure

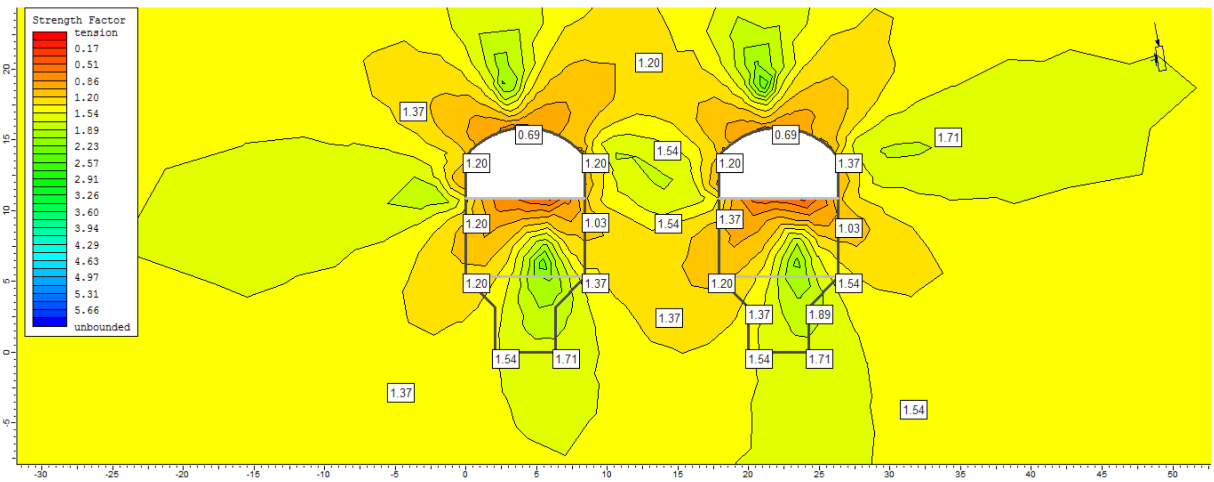

Figure 7-22 : Strength factor diagram of section B-B at stage I of excavation (Elastic analysis)

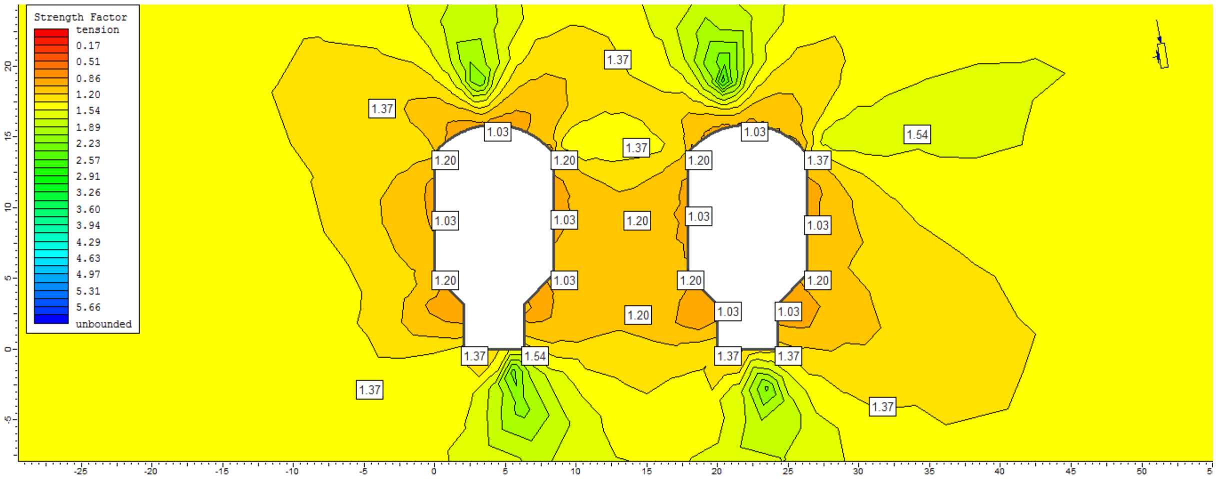

Figure 7-23 : Strength factor diagram of section B-B at final stages (Elastic analysis)

<u>Plastic analysis:</u>

Plastic analysis is done to estimate the rock support system by assuming the rock mass is plastic. Result from the cavern analysis shows that the cavern may fail, if unsupported because values of strength factor in each stages are one. Yielded elements are also increasing from stage I to stage II but it is reduced again in stage III. It may be due to more stable shape at the bottom part. Although the maximum total displacement is, only 0.77cm.Figure 7.24 to 7.26 shows the variation of major principle stresses, strength factor, and maximum total displacement around the cavern in different stages of excavation(without support condition).

Table 7.7: Result at different stages without support condition for section B-B (Plastic analysis)

Stages	σ_1(MPa)	Strength Factor (Strength/Stress)	Maximum Total Displacement(mm)	Yielded elements	Remarks
I	21.85	1	0.0079	299	Fail if unsupported
II	22.8	1	0.0085	328	Fail if unsupported
III	22.05	1	0.0077	325	Fail if unsupported

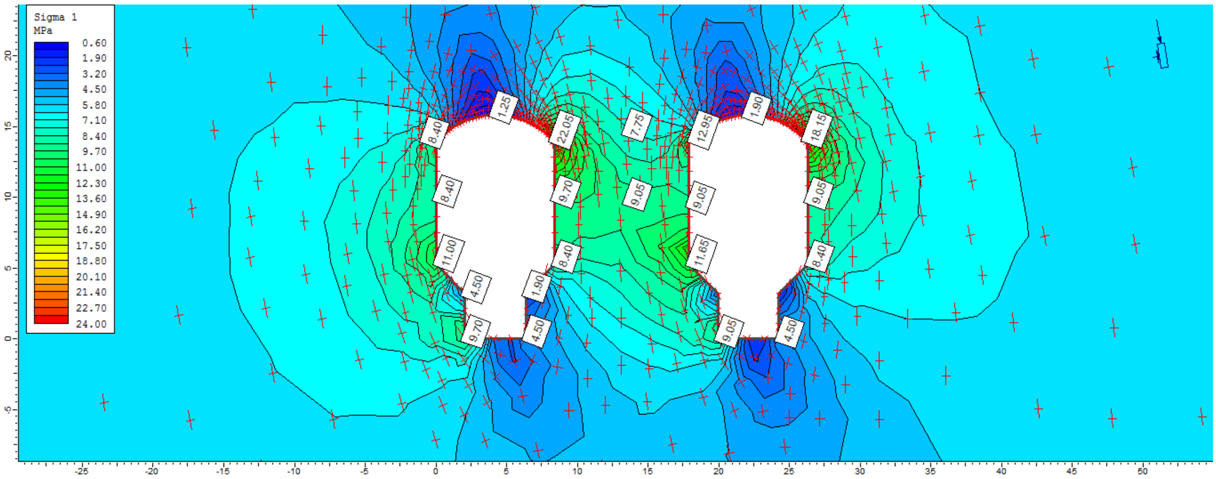

Figure 7-24 : Major principle stress diagram at section B-B (Plastic analysis)

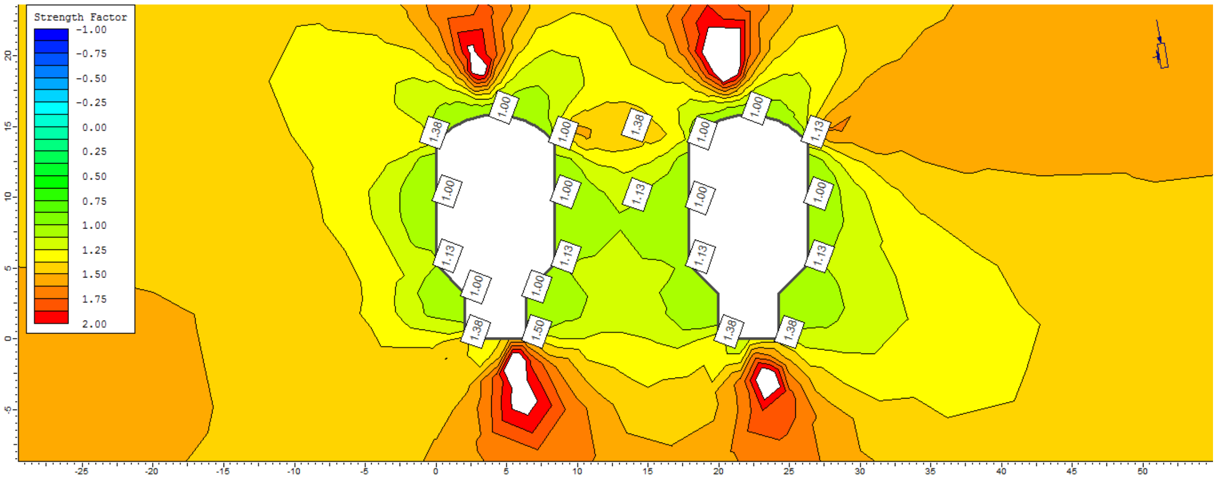

Figure 7-25 : Strength factor diagram at section B-B (Plastic analysis)

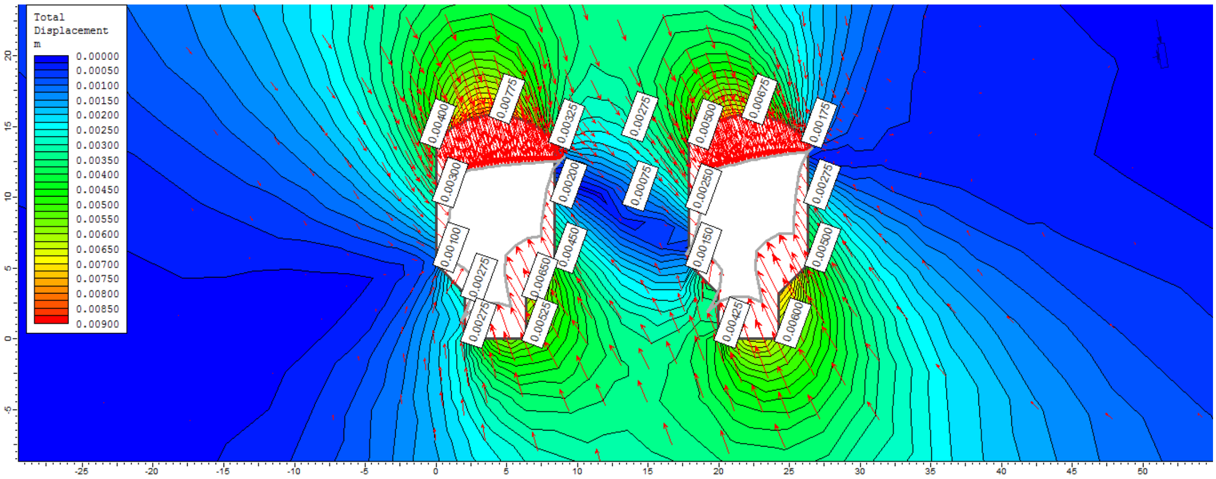

Figure 7-26 : Total displacement diagram at section B-B (Plastic analysis)

Plastic analysis with support:

Different combination of rock support system are installed to reduce the maximum number of yielded finite elements.Initially,6 cm thick shotcret with 3 m long bolt, keeping all other properties same as described in Table 7.2 are used. In this case, there are not significant changes in principle stresses and deformation compare to without support condition. Strength factor is increased from one to 1.04 whereas, reduction in yielded finite elements by one. It means, rock support system do not have much influence to strengthen the cavern. Reduction in number of yielded shotcrete from 13 to zero is possible after increasing the thickness of shotcret to 0.1m.In this second case, other parameters such as strength factor, stresses and deformation do not change significantly. Therefore, it is concluded that there is no need of support even if the strength factor is very close to one in both elastic and plastic analysis. For safety, Installation of minimum rock support suggested by Barton (Q-system) is recommended for this section as well. (Refer Chapter 6).

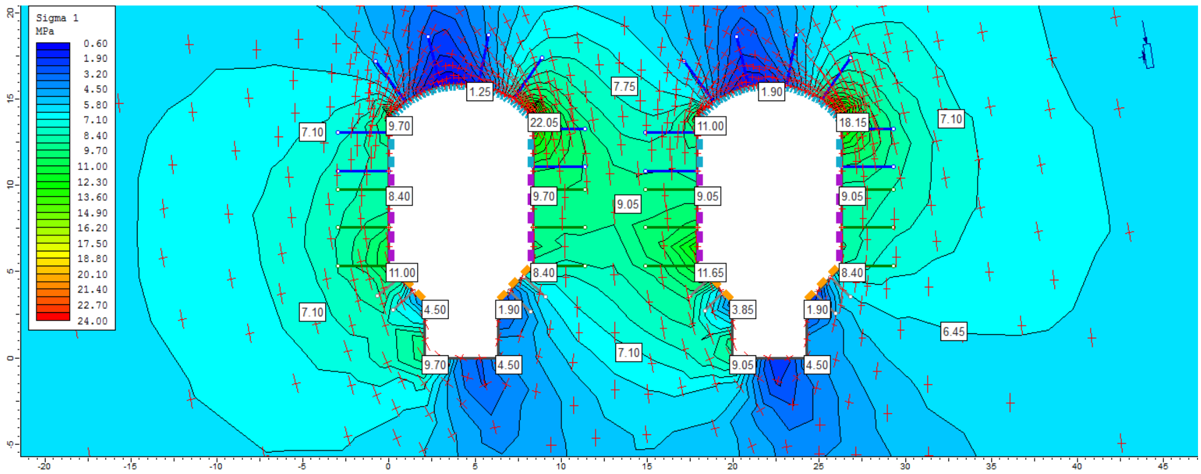

Figure 7-27 : Major principle stress diagram at section B-B after final stage (Plastic analysis)

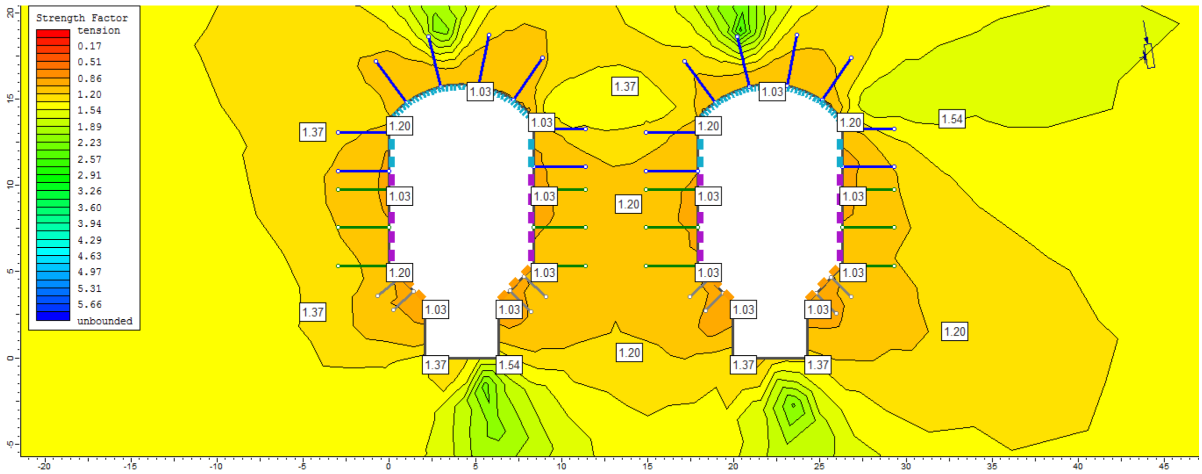

Figure 7-28 : Strength factor diagram at section B-B after final stage of (Plastic analysis)

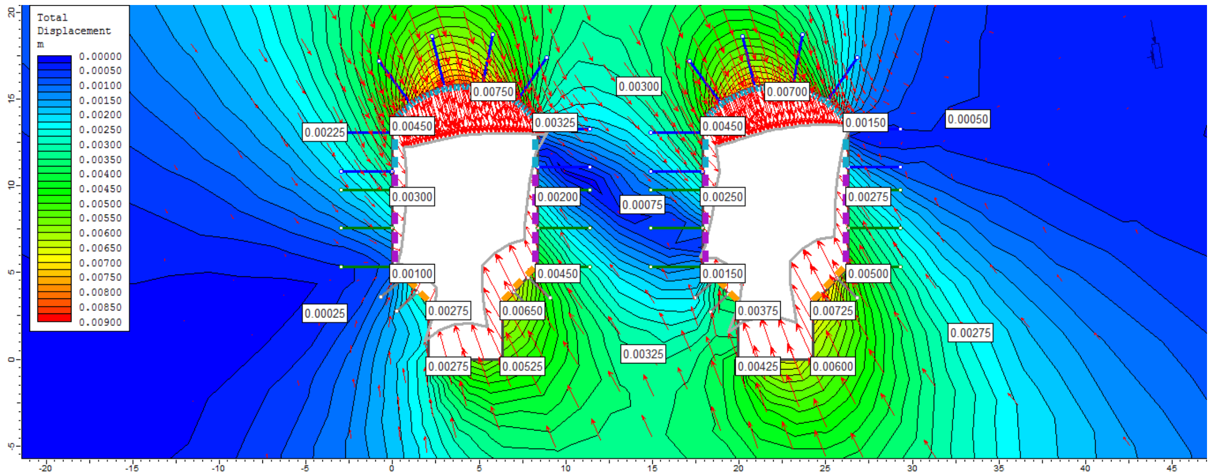

Figure 7-29 : Total displacement diagram at section B-B (Plastic analysis).

Section C-C (Chainage 0+218m)

Elastic analysis:

Elastic analysis at section C-C shows that the strength factor is less than one at crown and centre of wall of the cavern. Minimum value is 0.95 at centre of wall whereas the maximum is 1.81 at the invert level. In elastic analysis, strength factor gives the degree of overstressing of the rock mass. Strength factor less than one through elastic analysis suggest for plastic analysis for rock support estimation. Figure 7.30 shows the strength factor at various points in periphery of cavern.

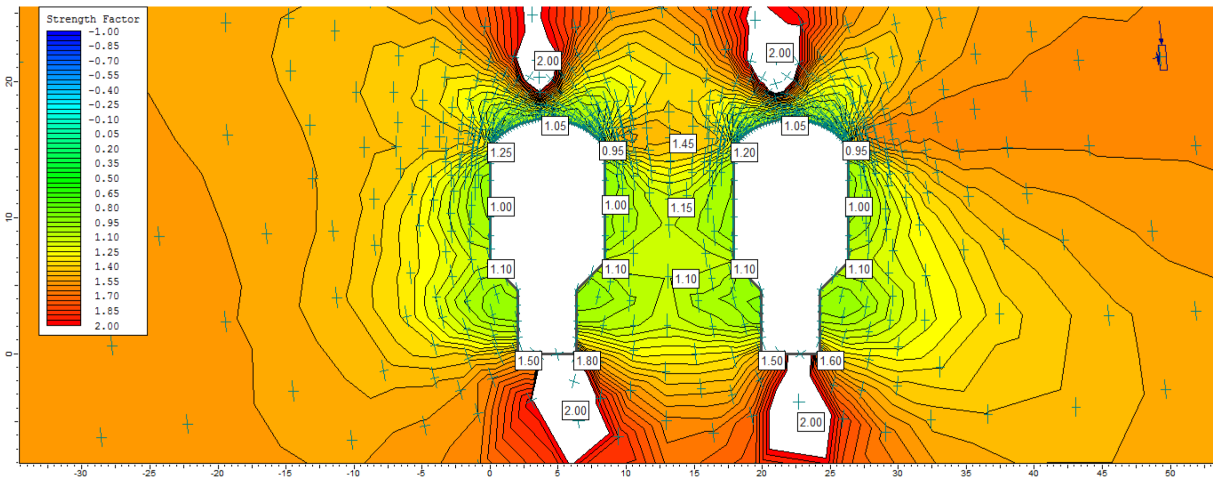

Figure 7-30 : Strength factor diagram at section C-C (Elastic analysis)

Plastic analysis

Results of plastic analysis are shown in Figure 7.31 to 7.33. Maximum value of major principle stresses is 26.6 MPa at the corner of the arch whereas Minimum value of minor principle of stresses is zero at centre part of wall and at crown. Strength factor is varying from one (at crown and centre of wall) to 1.88 (on the invert). This shows that the cavern may fail from crown or centre of wall or both, if unsupported. Maximum total displacement is about 1 cm at the crown. This value is quite high for the fully brittle rock, but it is assumed as okay for moderately brittle rock (banded Gneiss).Total numbers of yielded elements prior to installation of support are 372.

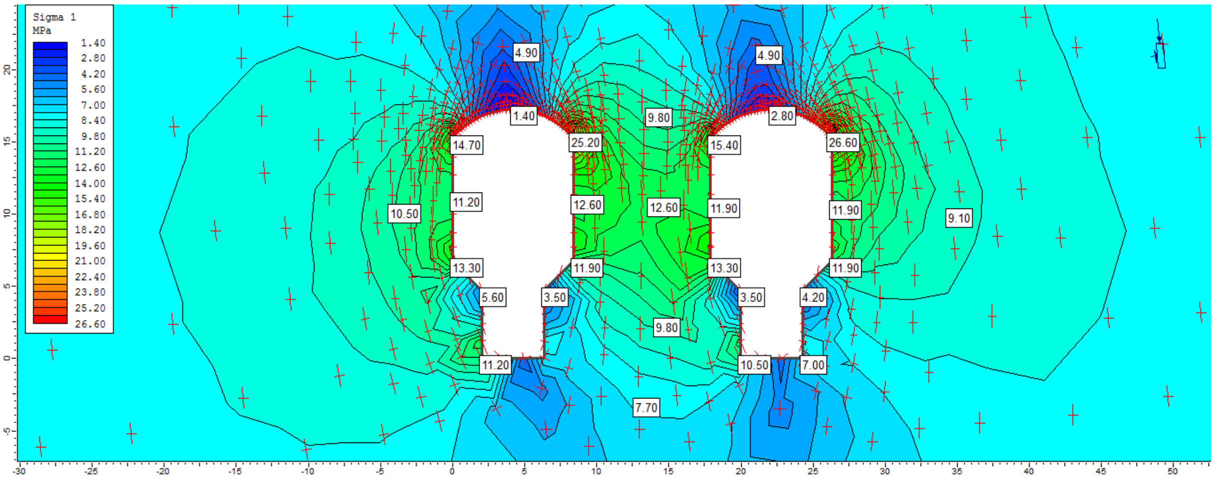

Figure 7-31 : Major principle stress diagram at section C-C without support condition (Plastic analysis)

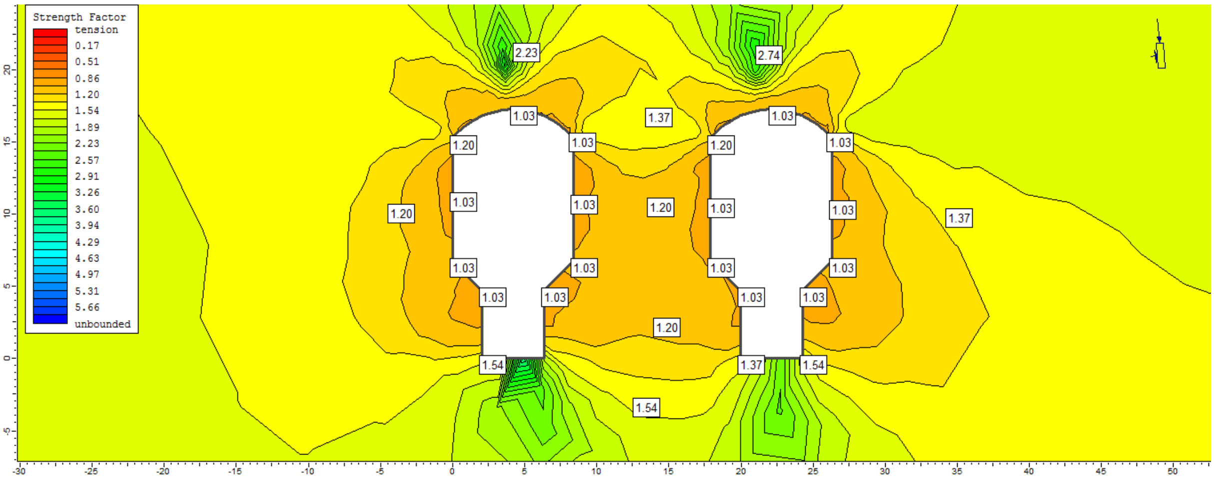

Figure 7-32 : Strength factor diagram at section C-C without support (Plastic analysis)

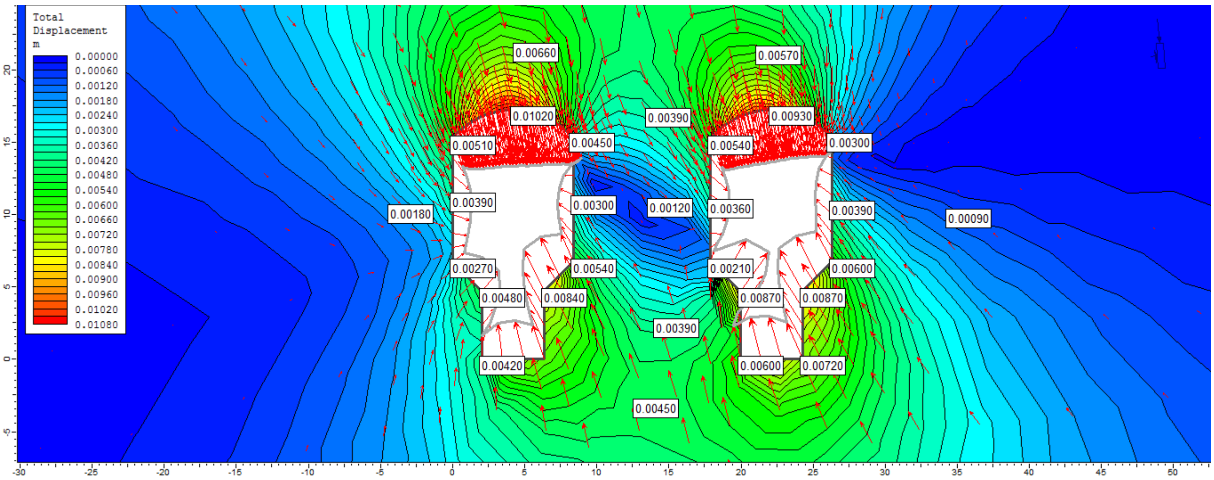

Figure 7-33 : Total displacement diagram at section C-C without support condition (Plastic analysis)

Plastic analysis with support

Plastic analysis is carried out with various combination of rock support system. When analyzing with rock support, stress behavior does not change a lot. Figure 7.34 to 7.36 show the distribution of the major principle stress, strength factor, and total displacement around the excavated cavern after the final stage with normal shotcrete and bolt as described in Table 7.3. In this case, Maximum value of major principle stress, Minimum value of strength factor (at crown and centre of wall), and maximum total displacements are 25.8 MPa, 1.03 and 1 cm respectively. Total numbers of yielded finite elements are reduced from 372 to 370 and five shotcret elements are yielded. Reduction in yielded elements shows that the support system is but not very active to increase the strength of rock mass. Support system is taking the load but it does not improve the strength of rock mass significantly

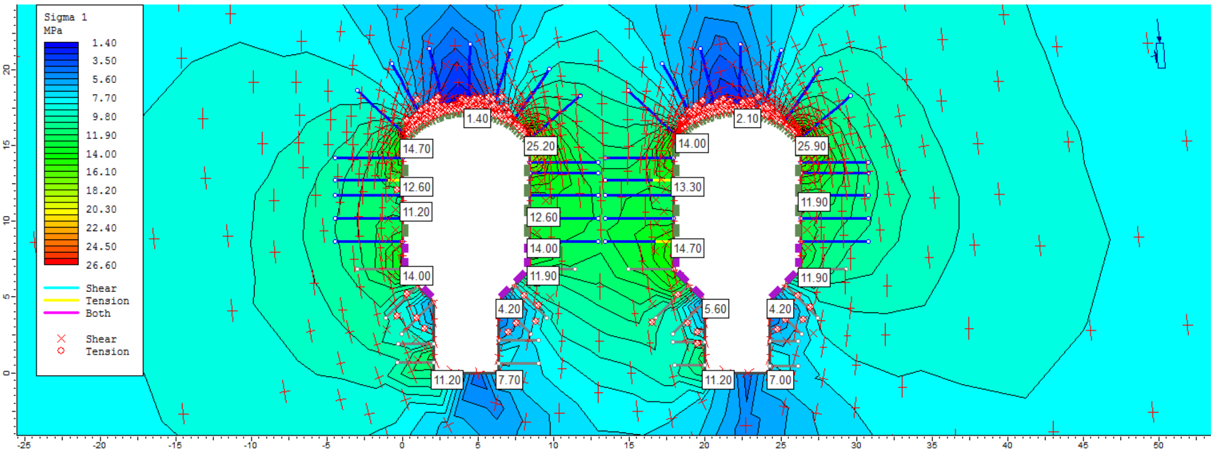

Figure 7-34 : Major principle stress diagram at section C-C with support condition (Plastic analysis)

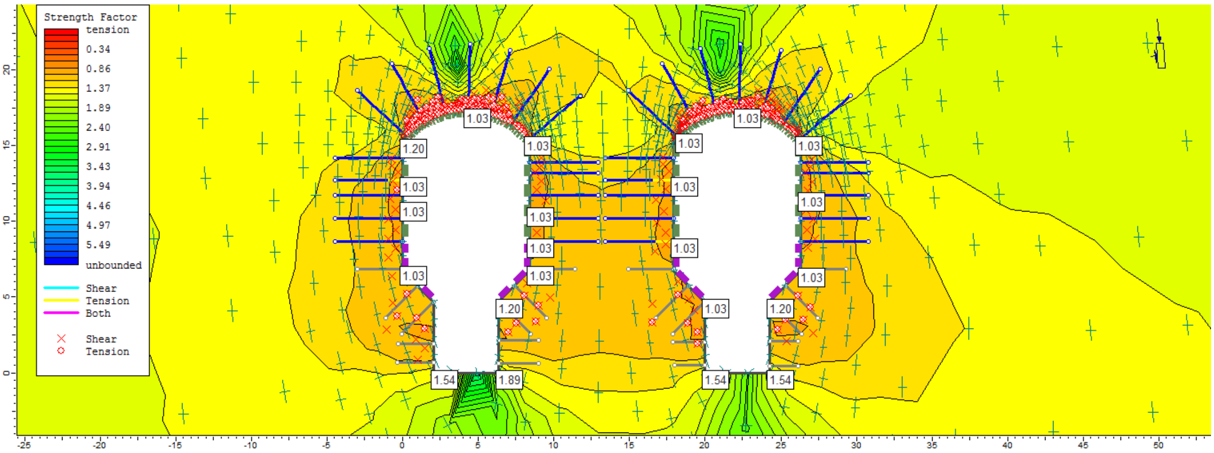

Figure 7-35 : Strength factor diagram at section C-C with support condition (Plastic analysis

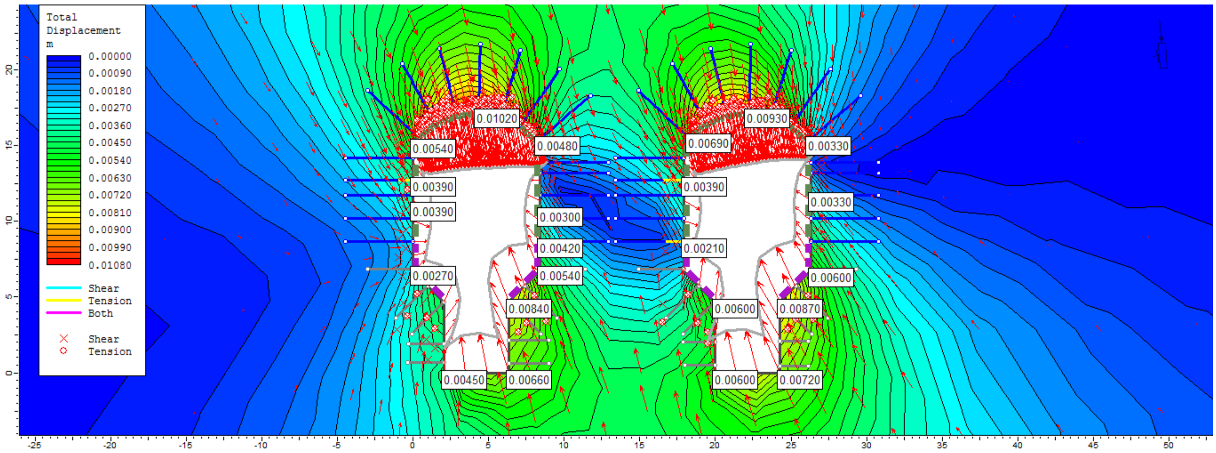

Figure 7-36 : Total Displacement diagram at section C-C with support condition (Plastic analysis)

Figure 7.38 shows, how and which shotcret elements are taking the load in two different stages. Results from the various combinations of lining and rock bolt shows that high tensile force (About 0.7MN) comes at wall of the cavern. High stresses on shotcret (at wall) results to yield four shotcret elements after next consecutive stage of excavation. Figure 7.37, shows that the values of strength factor in various location of cavern while using extensive rock support system (RCC with bolting). New support system is only able to reduce the total number of yielded finite elements from 372 to 369.Maximum total displacement with simple support system(As suggested by Barton Q system) and extensive RCC lining conditions are only differ by 1mm.There is not significant changes on stress condition and strength factor as well. Yielded bolt is reduced from two to zero, after the RCC lining.

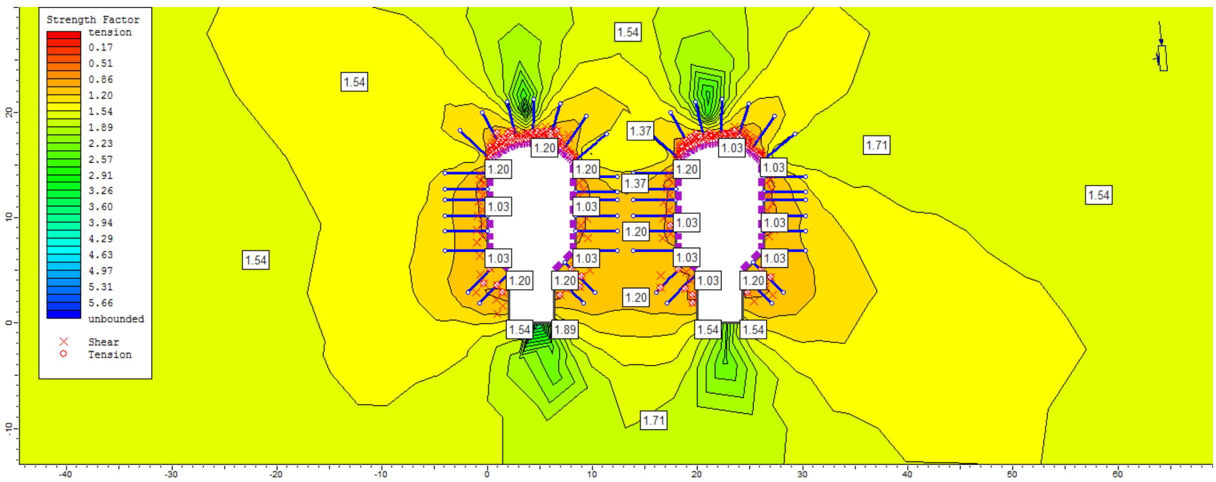

Figure 7-37 : Strength Factor diagram at section C-C with (Reinforced Cement Concrete) support condition at wall (Plastic analysis)

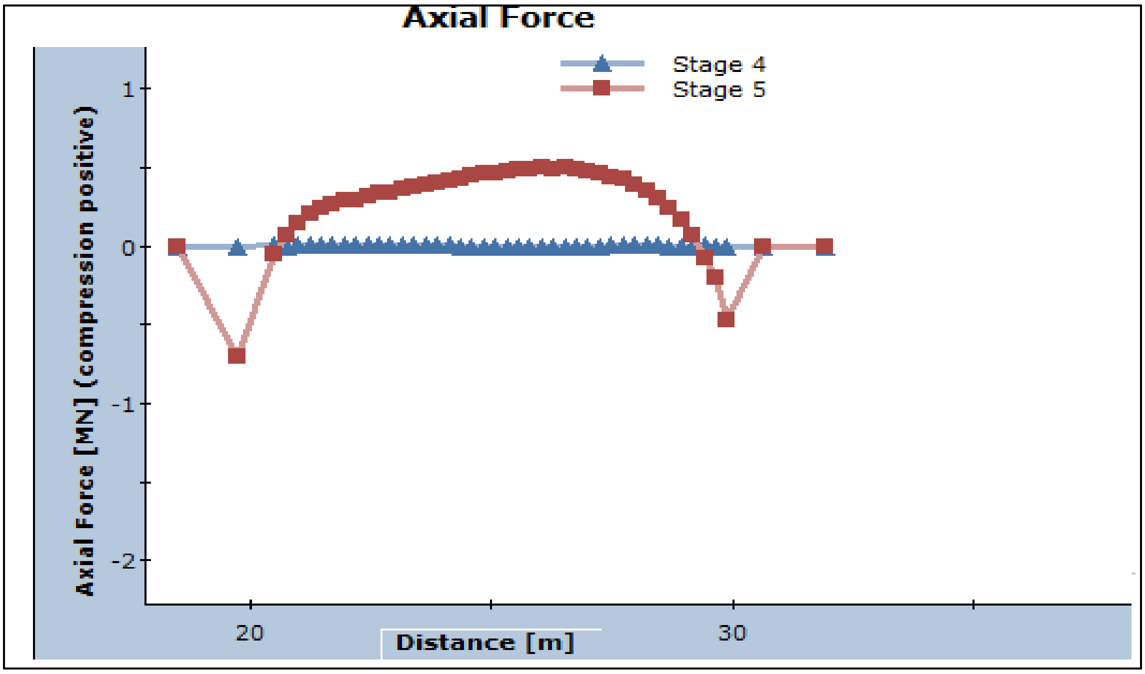

Figure 7-38 : Axial force taken by shotcret on wall in prior and post to excavation

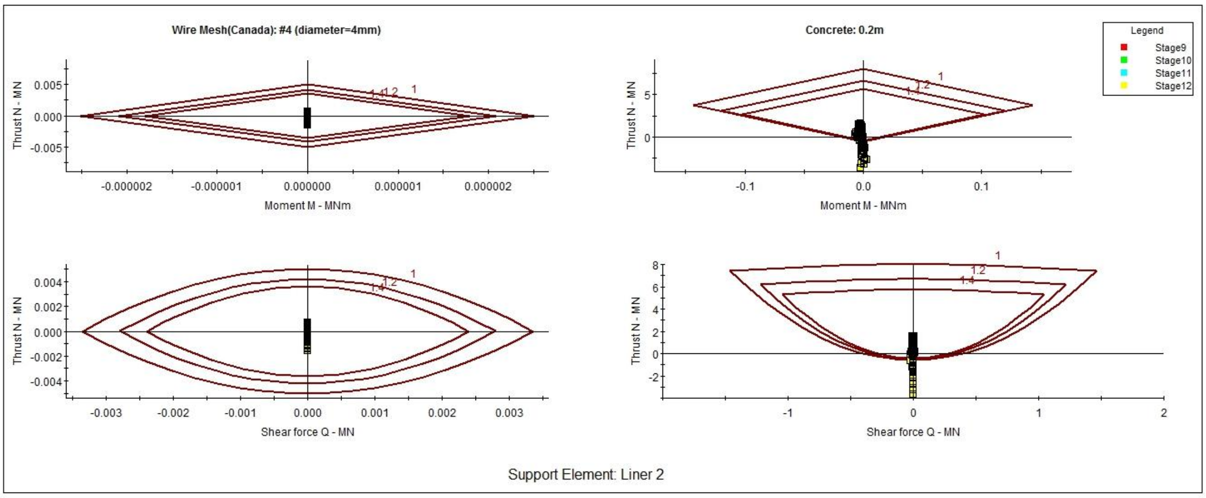

Figure 7-39 : Support capacity plot for RCC lining at section C-C

After installation of RCC support, total numbers of yielded bolt are reduced from two to zero. It is due to sharing load by RCC part. Support capacity plot for the RCC liner at both wall and roof shows that the steel part have still sufficient capacity to bear the more load. In case of wire mesh, all the elements are within factor safety of 1.4 but for concrete of thickness 0.2 m, many concrete elements are still failed. It is because of low tension bearing capacity of concrete. Concrete part is still crushed even after wire mesh.

If the shear strength of joints is less than the imposed shear stress, the crushed rock mass along with crushed concrete will move along the shear plane and block failure problem may arise. Block failure analysis is carried out in Chapter 8.

Since, there is no progressive nature of yielded elements and even RCC cannot reduce the existing values, it can say that the rock support is not required

7.2.2.3.2 Alternative II

In this alternative, Single cavern is excavated instead of two different caverns. Width of the cavern is 18.3 m and height is ranging from 15m to 20 m in different sections. Similar to alternative I, Three different sections A-A, B-B and C-Care taken for the analysis (Refer section 7.2.2.1).

7.2.2.3.2.1 Input Parameters

Parameters are the variables, which reflect the different properties of material, support systems, or external variable like field stress. Accurate calculation/estimation of input parameter determines the accuracy of result after the analysis. Therefore, precise determination of input parameters is a challenging task to all the engineer/designer. Input parameters, which are needed in numerical analysis through PHASE^{2}program, are discussed here. Material input parameter, field stress input parameter, and input parameters for rock support are the major input parameters and are discussed below.

Material input parameter

Material parameters for the banded gneiss in all the section are taken as same, which is described in table 7.1.Some other parameters that are needed for plastic analysis are discussed in section 7.2.2.3.1.1.

Field stress input parameter

Location of each section that is considered for analysis in alternative II and I are same. Therefore, induced virgin principle stresses in both the alternative are also equal which are also already discussed in section 7.2.2.2.2.

Support input parameter

In numerical analysis, various combination of rock support (steel fiber reinforced shotcret, fully bonded rock bolts, rein-forced concrete) is installed to reduce the maximum number of yielded finite elements. Simulation is done until the yielded finite elements are lowest, and is considered as optimum point and corresponding rock support is called optimum rock support. Followings are the input parameters for different type of support system that are used in the analysis.

Table 7-8 : Support input parameter for alternative II

Shotcret				Bolt			
Sections	A-A	B-B	C-C	Sections	A-A	B-B	C-C
Liner Type	Standard beam	Standard beam	Standard beam	Type	Fully Bonded	Fully Bonded	Fully Bonded
Beam Formulation	Timoshenk	Timoshenko	Timoshenk	Length	5	5	5
Young`s Modulus	30,000 (MPa)	30,000 (MPa)	30,000 (MPa)	Diameter	25	25	25
Poison Ration	0.2	0.2	0.2	Spacing (In×Out)	2.2×2.2	2.2×2.2	2.2×2.2
Thickness	0.1 m	0.12 m	0.15 m	Bolt Modulus	200000 MPa	200000 MPa	200000 MPa
Peak UCS/Residual	35/5MPa	35/5MPa	35/5MPa	Peak Tensile Capacity	0.1 MN	0.1 MN	0.1 MN
Peak Tensile/ Residual	5/0Mpa	5/0Mpa	5/0Mpa	Residual Tensile Capacity	0.01 MN	0.01 MN	0.01 MN

7.2.2.3.2.2 Output parameters

Interpret module of PHASE^2 gives the facility to visualize and analyze the result after the simulation. Most useful output parameters for the analysis of alternatives II are principle stresses, strength factor displacements, and yielded elements. They are similar to the parameters that have already discussed in section 7.2.2.3.1.2.

7.2.2.3.2.3 Modeling Result, Comparison and Discussion

Section A-A

Elastic Analysis:

Elastic analysis shows the stresses are concentrated at the corners. Maximum value of major principle stress is nine Mpa at the right top corner. Strength factor is always equal to or greater than one in every point on periphery of the cavern. Maximum value is 1.31 at the invert level. Maximum total Displacement is about 5.8 mm after final stages of excavation. From this analysis, Rock support is not required to make safe cavern but Q-system for rock support estimation suggests to install rock support (Refer Chapter 6).Since the value of strength factor is very close to one, It feels to do plastic analysis and will try to improve the stability condition by putting support system. Figure 7.40 to 7.42 shows the values of major principle stress, strength factor, and total displacements at different point in periphery of cavern.

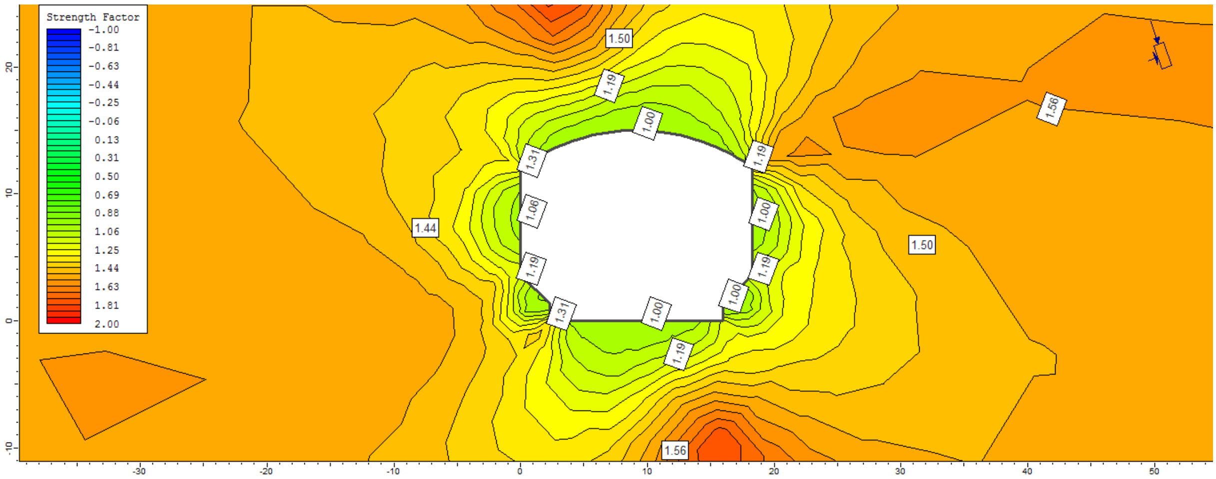

Figure 7-40 : Strength factor diagram at section A-A (Elastic analysis)

Plastic analysis:

Plastic analysis shows that strength factor is always greater than one in any points of contour. Minimum value is 1.03 (at crown, centre of wall and invert level) whereas the maximum value is 1.37. Stress conditions in elastic and plastic analysis are also almost similar. Maximum total displacement in plastic analysis is also same as elastic analysis after final stages of excavation and have value of 5.83 mm. yielding elements are 165 in stage I where as in stage III it decreases to 76 . This means cavern have capacity to stabilize itself after some time and have no chance of further losing of finite element as time progress. Figure 7.41 to 7.43 shows the results after final stage of excavation.

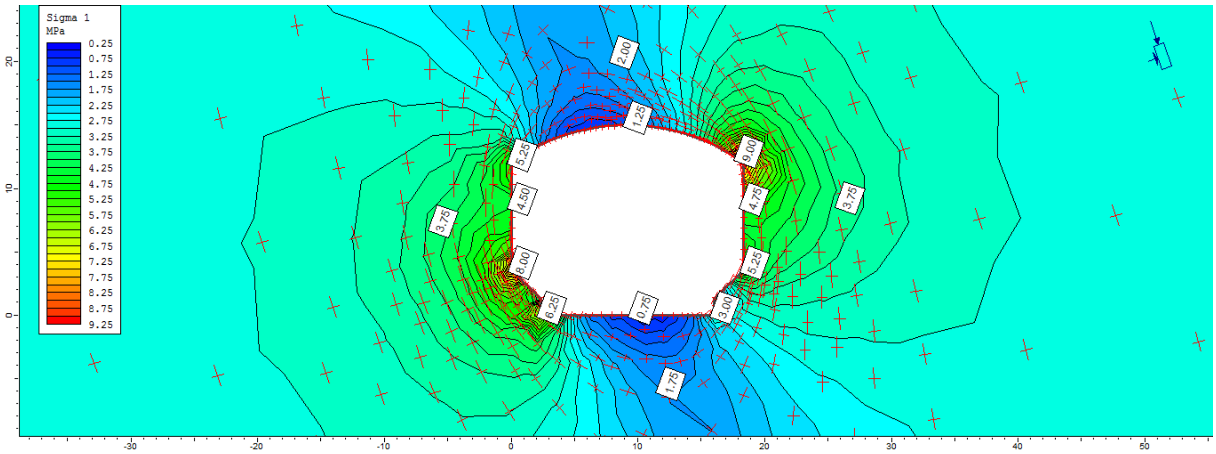

Figure 7-41 : Major principle stress diagram at section A-A Without support (Plastic analysis)

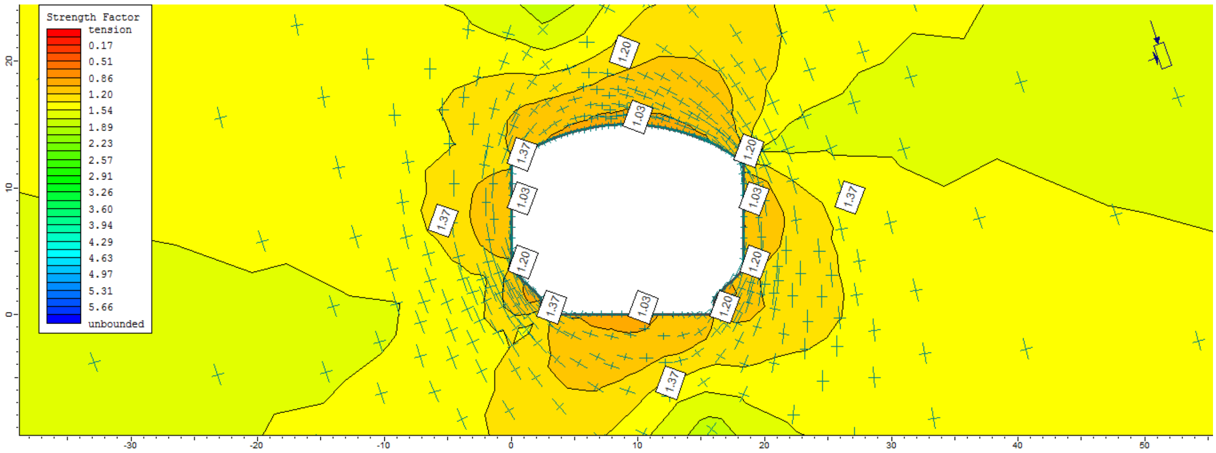

Figure 7-42 : Strength factor diagram at section A-A without support (Plastic analysis)

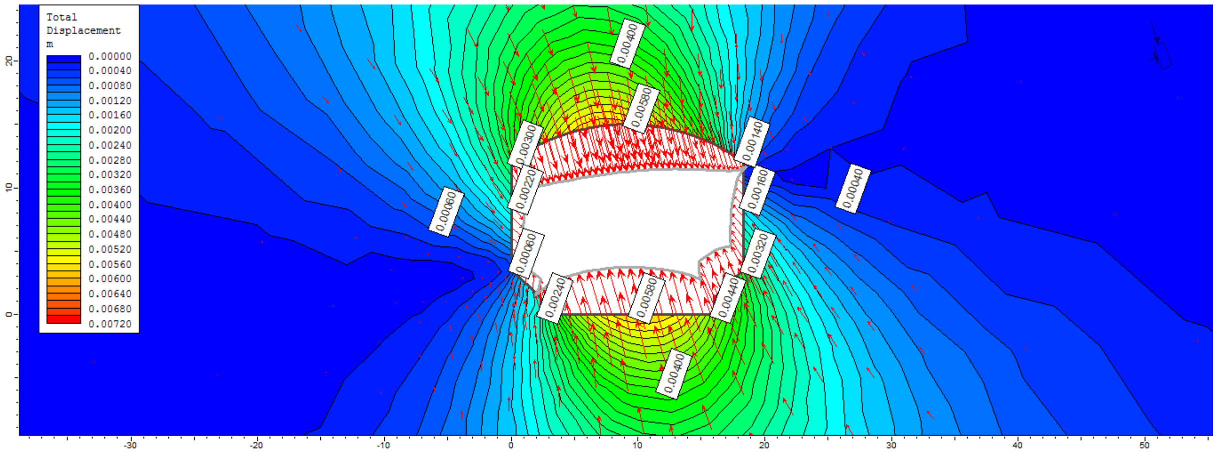

Figure 7-43 : Total displacement diagram at section A-A without support condition (Plastic analysis)

Rock support suggested by Q-system has applied but it does not able to reduce the yielded finite elements below 76, and total displacement below 5.8 mm. However it is recommended to put minimum rock support suggested by Barton (Refer Chapter 6) to prevent from time dependent failure (Rock mass (exposed cavern) may weathered and becomes weak as time progress).

<u>Section B-B:</u>

<u>Elastic analysis</u>

Here is a new analysis on how the shape of excavated cavern makes different in stability condition. First shape is named as B1-B1 and the modified shape is named as B2-B2.Both of cavern is excavated in four stages. Result from the each cavern after simulation in each stage is presented in table below.

Table 7-9 : Comparison and Selection of better shape of cavern through numerical analysis

Shape B1-B1				Shape B2-B2		
Stages	σ_1(MPa)	S.F	Displacement(m)	σ_1MPa	S.F	Displacement(m)
I	30	0	0.0137	24	0.26	0.01259
II	22.05	0.26	0.01446	18.5	0.26	0.01428
III	21.9	0.26	0.0149	16.55	0.26	0.014775
IV	21.85	0.52	0.0134	16.1	0.78	0.01338

After analyzing the above output data, it concludes that the shape B2-B2 is better than Shape B1-B2.. Although the excavation volume is slightly higher in second case, shape B2-B2 is taken for further analysis because of its more stable nature. Figure 7.44 and 7.45 shows the strength factor in two cases.

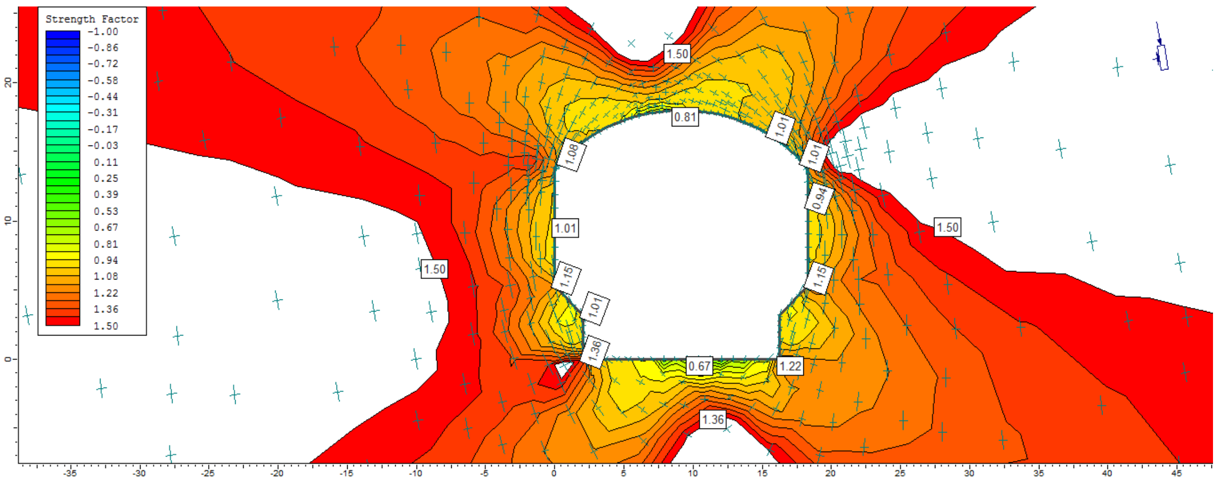

Figure 7-44 : Strength factor diagram for Type B1-B1 (Elastic Analysis)

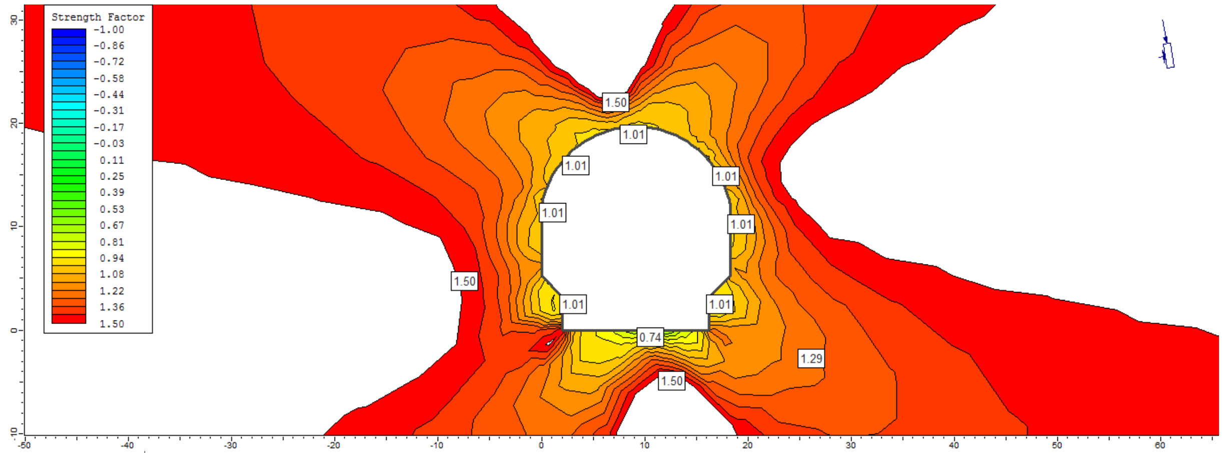

Figure 7-45 : Strength factor diagram for Type B2-B2 (Elastic Analysis)

Lowest value of strength factor is 0.74 at the invert. Whereas highest is 1.01 in all other location. Therefore it is suggested to do further plastic analysis to estimate the rock support. It shows that cavern may fail, if unsupported.

Plastic Analysis:

Minimum strength factor is found to be 1.03 in all points of contour except few places at invert. Total maximum displacement is 1.37 cm, which is quite high compare to other previous section (A-A).Total number of yielded elements are 196.Maximum value of principle stress is 16.1 MPa at the right top corner of wall. Figure 7.46 to 7.58 shows the different values of major principle stresses, strength factor, and total displacement around the cavern.

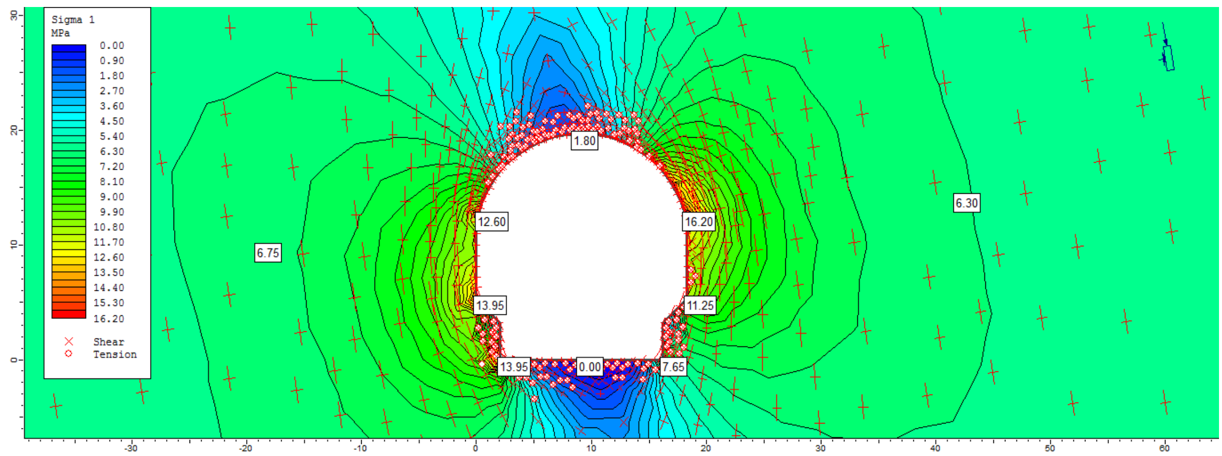

Figure 7-46 : Major principle stress diagram at section B-B Without support (Plastic analysis)

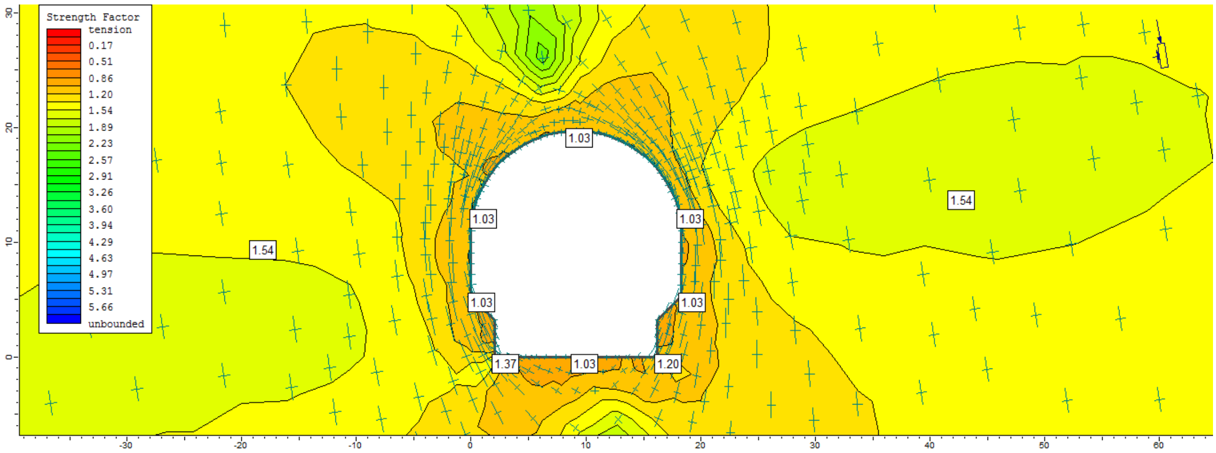

Figure 7-47 : Strength factor diagram at section B-B without support (Plastic analysis)

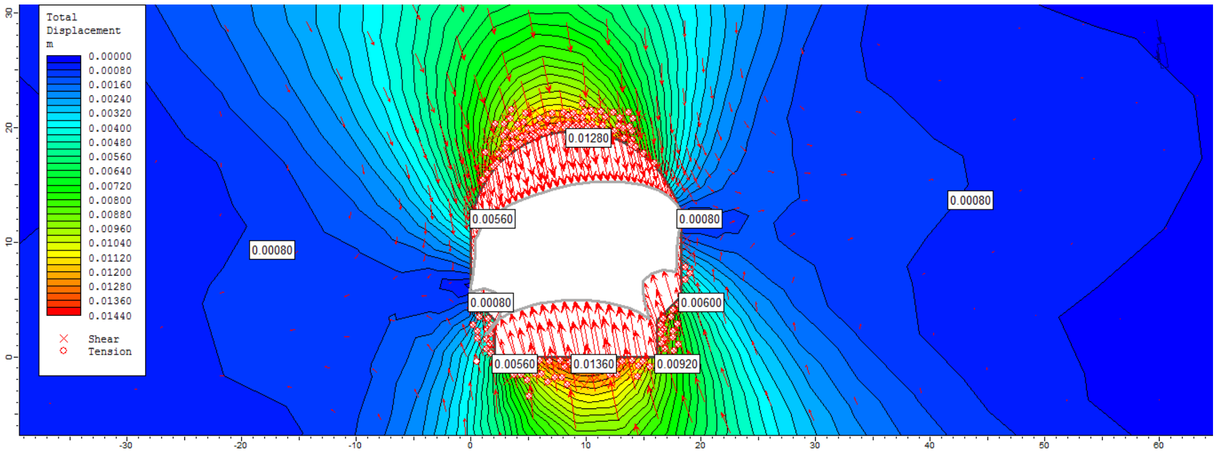

Figure 7-48 : Total displacement diagram at section B-B without support condition (Plastic analysis)

Plastic analysis with support:

In this analysis, supports are installed but it also does not have much significant like section A-A. Total numbers of yielded elements are reduced from 196 to 190. Maximum total displacement is reduced by 1 mm, minimum strength factor is same, but in some few locations, it increases slightly Reduction in Yielded finite elements after applying the support system signifies that the support system is working. In total there are 12 shotcret elements at wall are yielded. Figure 7.49 to 7.51 shows the different values of output parameters in periphery of contour. In this section, it is also recommended to put minimum rock support system suggested by Q-system (Refer Chapter 6) which is sufficient to make the cavern stable in future as well.

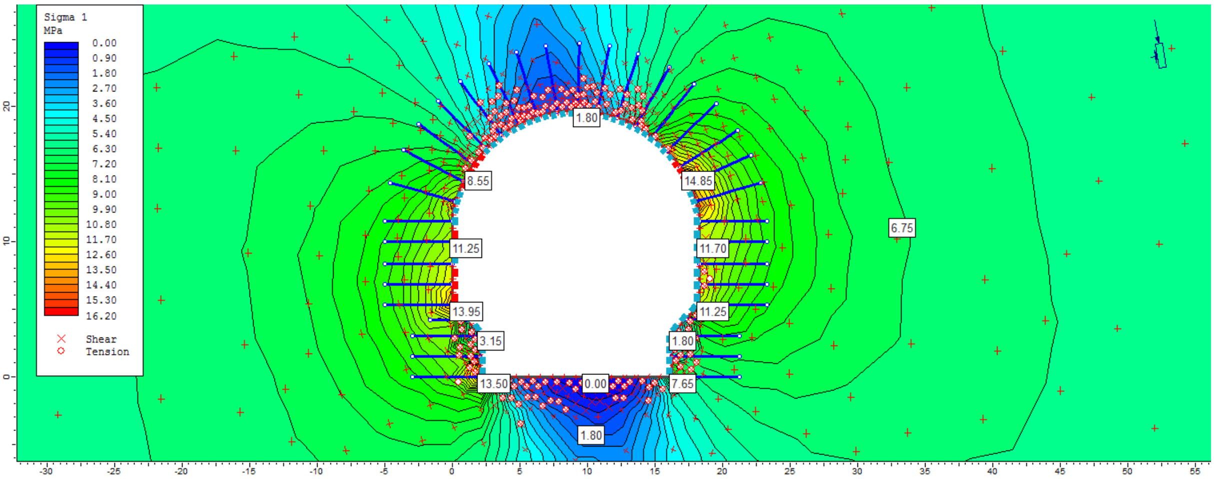

Figure 7-49 : Major principle stress diagram at section B-B With support (Plastic analysis)

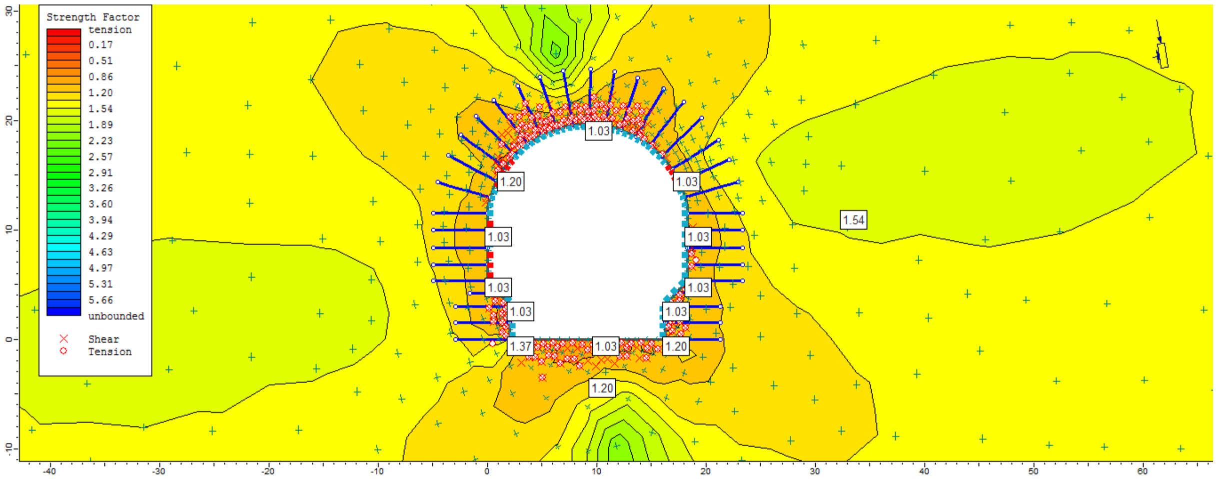

Figure 7-50 : Strength factor diagram at section B-B with support (Plastic analysis)

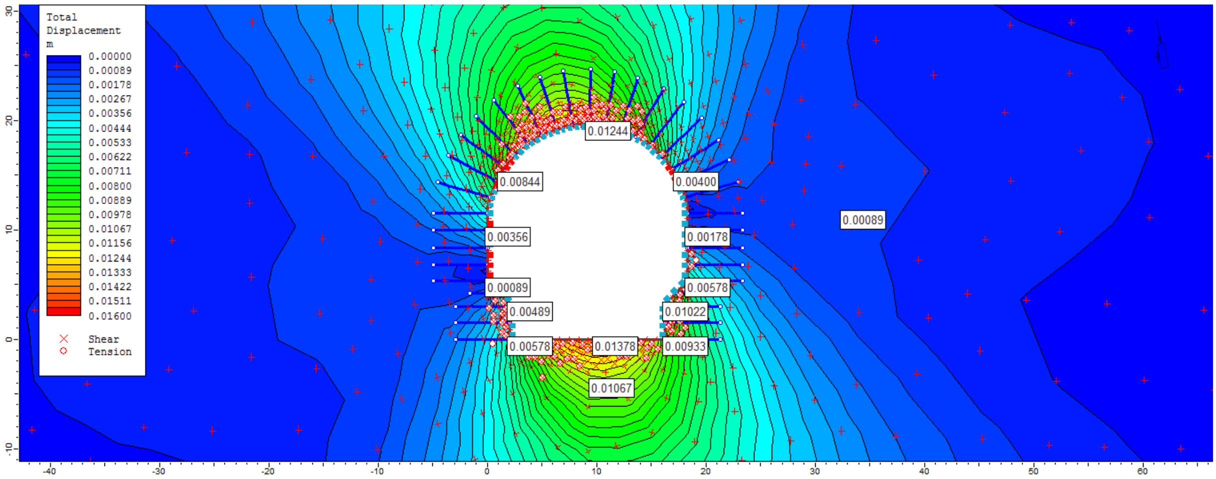

Figure 7-51 : Total displacement diagram at section B-B with support (Plastic analysis)

<u>**Section C-C**</u>

<u>Elastic analysis:</u>

This analysis shows the minimum value of strength factor at the invert level is 0.69 whereas the maximum value is up to 1.54. Strength factor less than one, through elastic analysis suggest for plastic analysis to estimate the extent of rock support to make the cavern stable. Maximum total displacement is 1.8 cm and is the highest among all sections. Cavern is highly stressed up to 26 MPa at the top right corner whereas least stressed at crown and invert that is zero MPa. Figure 7.52.shows the values of strength factor in different location around the cavern.

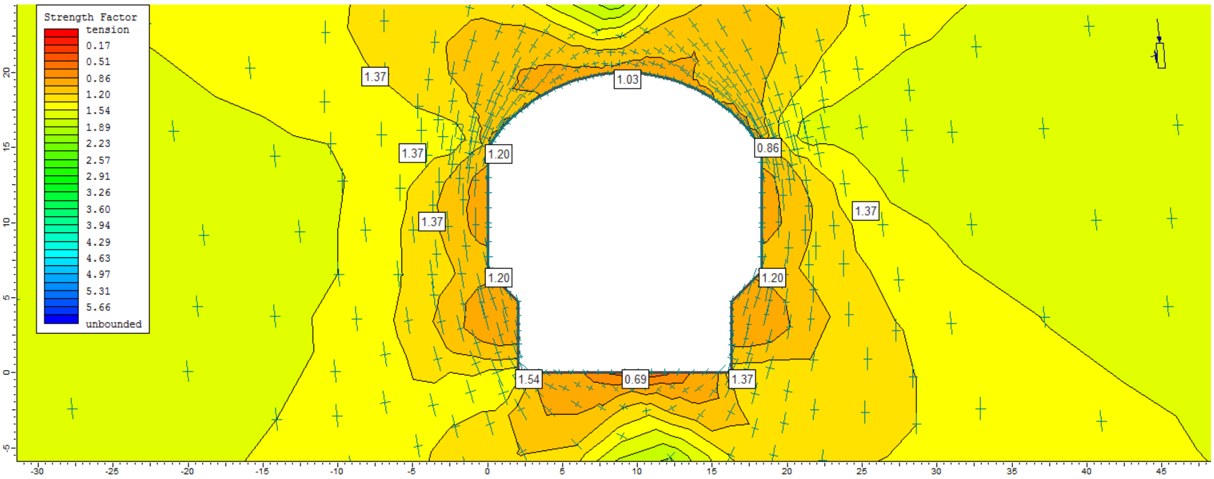

Figure 7-52 : Strength factor diagram at section C-C (Elastic analysis)

<u>Plastic analysis:</u>

Plastic analysis is performed to estimate the extent of rock support requirement in different part of the cavern. At first, analysis is done without support and then with support. Figure 7.53 to 7.55 shows the values of major principle stress, strength factor, and maximum total displacement at various points around the cavern prior to installation of support

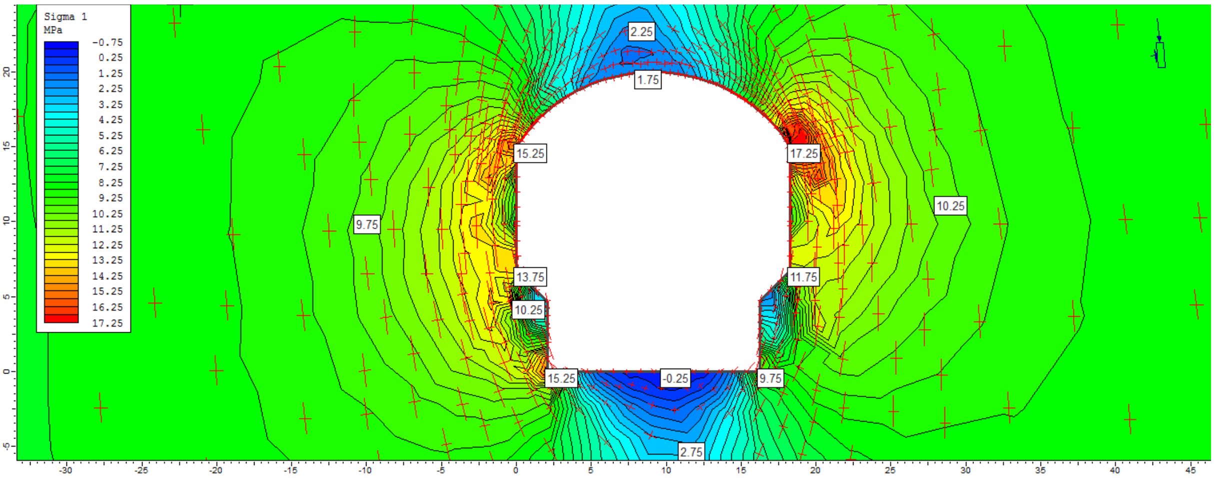

Figure 7-53 : Major principle stress diagram at section C-C without support (Plastic analysis)

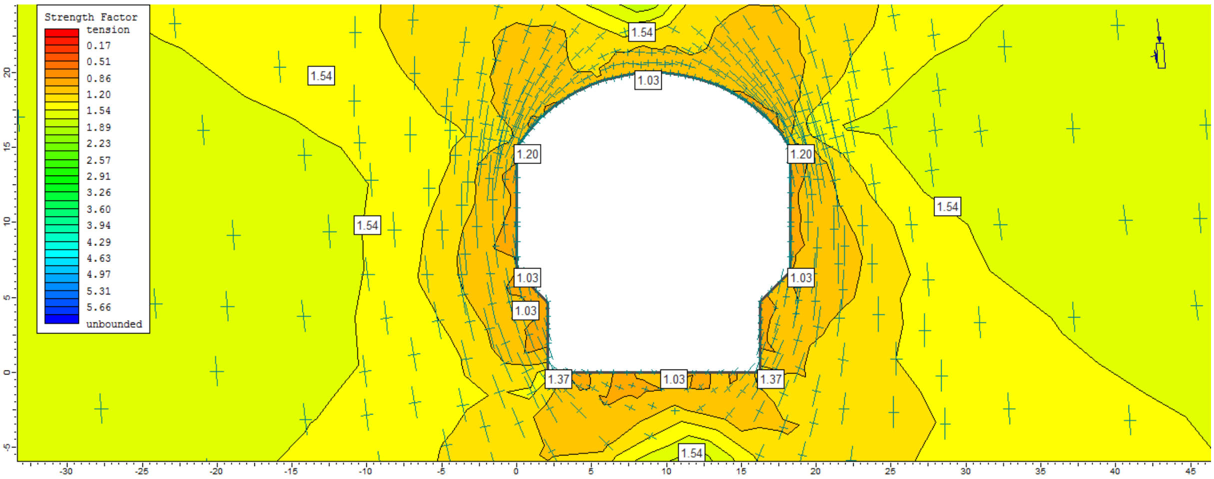

Figure 7-54 : Strength factor diagram at section C-C without support (Plastic analysis)

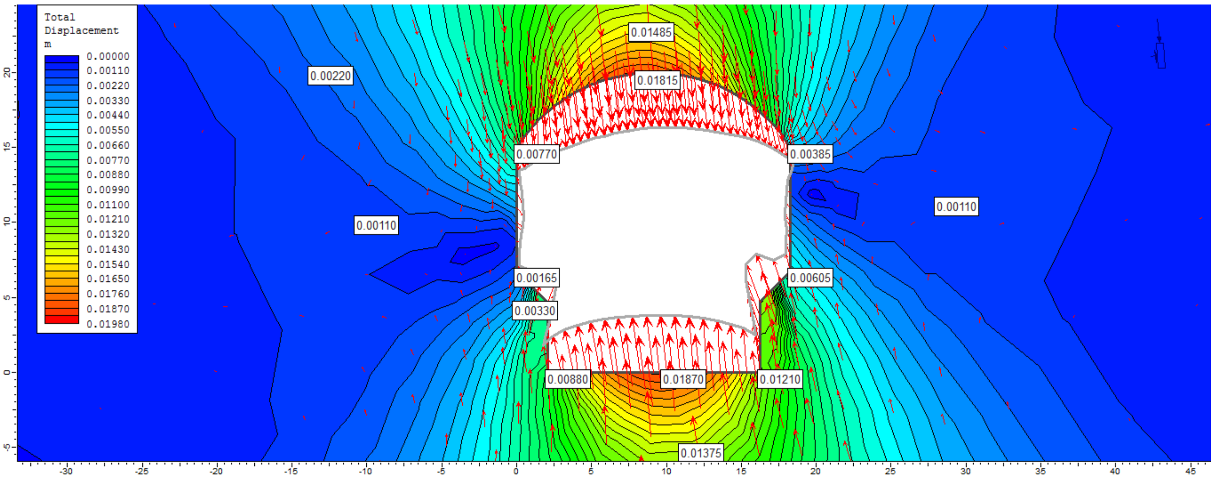

Figure 7-55 : Total displacement diagram at section C-C without support (Plastic analysis)

Major principle stress, strength factor maximum total displacement, and yielded elements are 17.25 Mpa, 1.03, 0.0188 m and 294 respectively after the final stage of excavation.

Plastic analysis with support:

After installation of support that is suggested by Q-system (Refer Chapter 6), there is no change in the quantity of yielded finite elements. Displacement is quite high compare to section A-A and B-B but reduction in maximum total displacement prior and post installation of support is almost zero. Major principle stress is reduced from 17.25 MPa to 16.05Mpa, which is very less compare to quantity of invested rock support. Numbers of Bolt and shotcrete failure are 10 and 27 respectively. Most of them are from wall due to tensile failure. Therefore, it is suggested to install few reinforcements at wall, instead of huge concrete/shotcrete to withstand imposed tensile force that comes along the wall. Overall strength of excavated cavern is not improved even after installing the reinforcement. Mechanical and geometrical properties of Reinforcement, which are used is presented in

Table 7.3 Major principle stress, strength factor and total displacements after installation of support system are presented in Figure 7.56 to 7.58.

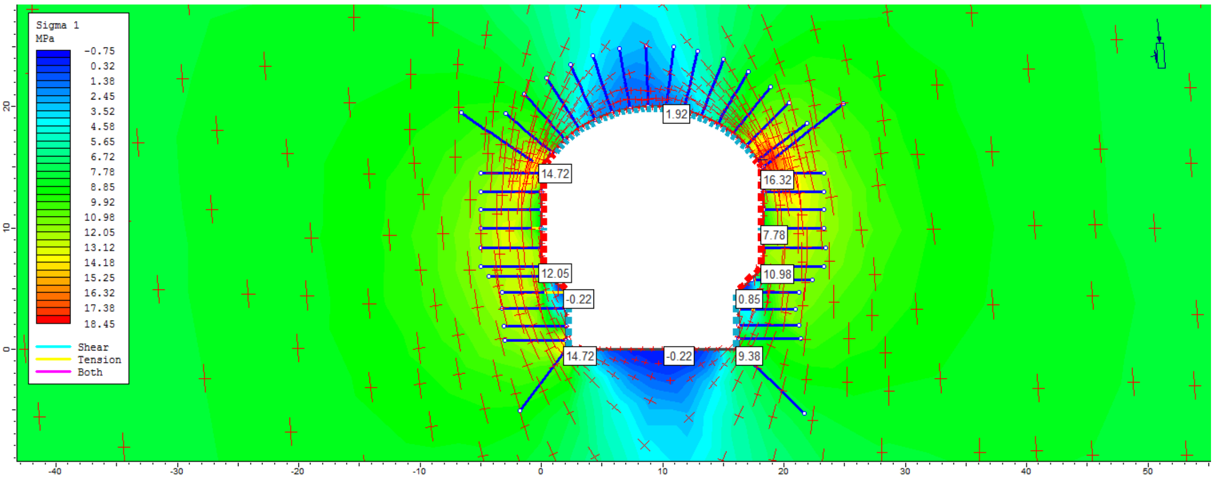

Figure 7-56 : Major principle stress diagram at section C-C with support (Plastic analysis)

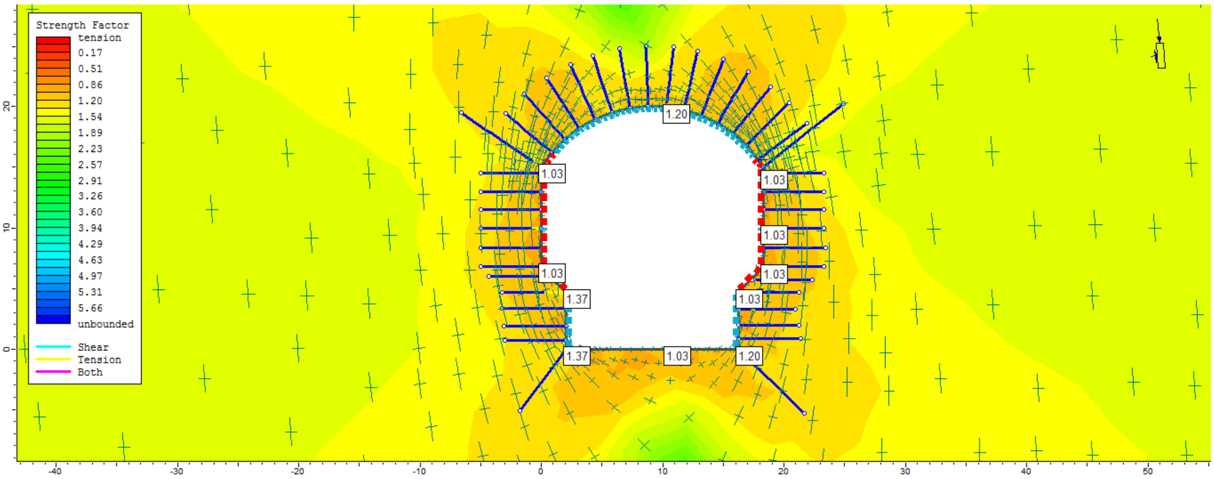

Figure 7-57 : Strength factor diagram at section C-C with support (Plastic analysis)

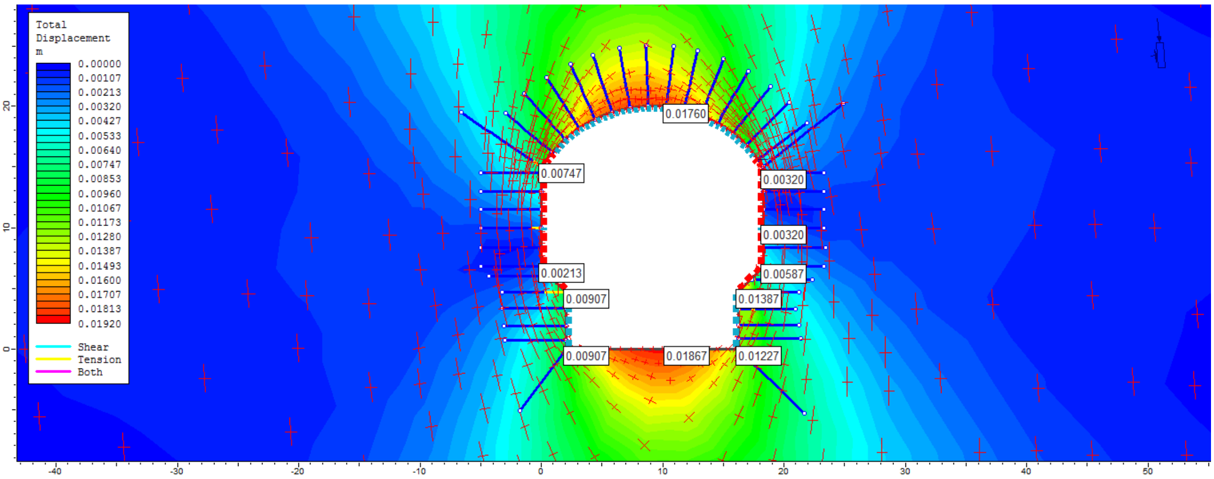

Figure 7-58 : Total displacement diagram at section C-C with support (Plastic analysis)

7.3 Discussion, conclusion and recommendation

In both elastic and plastic analysis, strength factor is always very close to one. Section A-A, B-B and C-C are relatively high, medium and low stable in both alternatives, which is due to the increasing rate of overburden pressure. In all the sections, alternative I is relatively more stable than alternative II. The result of the numerical analysis shows that the displacements, stresses, and strength factor are identical before and after the application of support system during plastic analysis. These may be due to the following reasons:

Rock mass Properties:

Properties of the rock mass in the project area shows that the rock is of good quality. It has UCS value of 78 MPa, which is derived from the world test result of UCS value of banded gneiss. Young`s modulus of elasticity of intact rock is 27 GPa. For such good quality of rock mass, post failure behavior of rock mass would be either elastic or elastic-brittle-plastic with sudden drop in the strength.

For both alternatives, walls have some instability problems due to vertical nature of major principle stresses. Because of longer span, there are also few stability problems at roof of alternative II. Yielded finite elements are of both shear and tensile natured. Relatively lesser increment of yielded finite elements in each stages of excavation show the cavern is stable. Since discontinuities are not accounted in these models and their effects are not considered, the displacements are relatively lesser.

Ground response on rock support:

Ground does not response immediately after the installation of support. The Figure below shows the relationship between Longitudinal Deformation profile (LDP), Ground Reaction Curve (GRC) and Support Characteristic Curve (SCC). Generally, support starts to take the load only after some deformation of the tunnel. In Figure 7-59, Top graph shows the longitudinal deformation of cavern in both sides from the face whereas Bottom figure shows how the ground response (radial deformation) with respect to time and load. Support characteristic curve shows that how and when the load is taken by the support system.

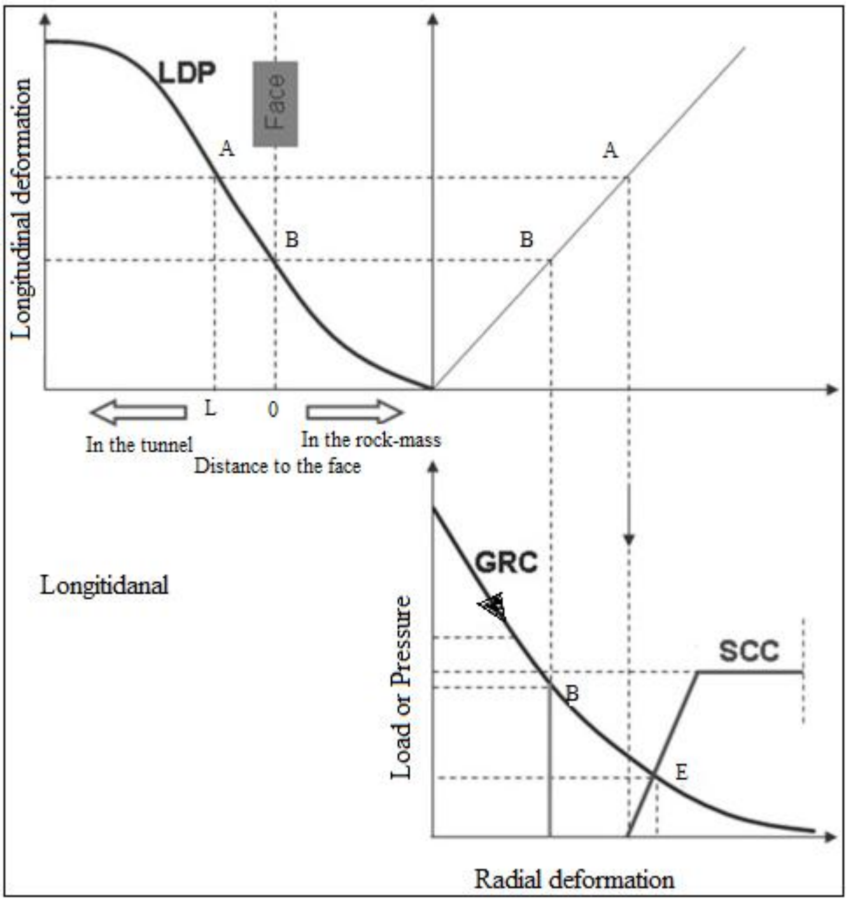

Figure 7-59 : Ground response with LDP, GRC, and SCC (Carranza-Torres and Fairhurst (2000)

In case of settling basin cavern of SMHEP, most of the support does not take the load immediately because of very low deformation at the initial phase of excavation (due to strong and brittle rock). Although the deformation is very less, it is highly recommended to install the minimum support suggested by Q-system in both the alternative to make the stable cavern. It is because of considering the time progressing deformation in and along the cavern as suggested by Carranza-Torres and Fairhurst (2000).

This type of time dependent failure analysis is vital while working with the elastic-brittle-plastic type of rock mass. Therefore it is highly recommended to do further analysis to the actual behavior of for the elastic-brittle-plastic rock mass, which is out of scope of this thesis work.

Many yielded finite elements in the analysis are failed due to sheared, therefore It is also recommended to do wedge failure analysis, to know whether the detach rock block will slide or not along the sheared plane. Wedge and Un-wedge analysis has been carried out in chapter 8.

CHAPTER 8

8 UN-WEDGE ANALYSIS

8.1 Introduction

Un-wedge is a tool for quick analysis of the geometry and stability of underground wedges.

Few important assumption and limitation of un-wedge program are as follows:

- Un-wedge is used to analyze wedge failure around excavation constructed in hard rock, where discontinuities are persistent; where stress induced, failure does not occur. It is assumed that displacement take place at the discontinuities and the wedges move as rigid bodies with no internal deformation or cracking
- By default, wedges are subjected to gravitational loading only. Stress field in the rock mass surrounding the excavation is not taken into account. While this assumption leads to some inaccuracy in the analysis, error is conservative and lead to lower factor of safety. But it can also include the effect of in-situ stress on the wedge.

8.2 Input parameter

Following input parameters that are used for the analysis:

General parameters

Tunnel Axis Orientation

Trend: N145E,

Plunge: one

Unit Weight:

Rock: 0.027 $^{MN}/_{m^3}$

Water: 0.00981 $^{MN}/_{m^3}$

Seismic Force

Trend: N47 E; (UTKHEP)

Plunge: Zero,

Seismic Co-efficient: 0.1 (Refer section 3.4)

Design factor of safety: 1.5 (Assumed)

In-situ field stress acts to clamp the wedge in its socket. The factor of safety can only increase because of field stress. Therefore, it is recommended to use long-term factor of safety for calculation, NOT include field stress, as any movement of the wedge, will negate the beneficial effect of stress. Field stress can only apply to perimeter wedges but cannot apply to end wedges.

Joint orientation parameters (Himal Hydro 2009)

Assumption: Same joint properties for all joint sets:

Table 8-1 : Joint Orientation (Himal hydro 2009)

joint	Dip	Dip Direction	Properties
1	48	190	Joint Properties 1
2	75	220	Joint Properties 1
3	68	100	Joint Properties 1

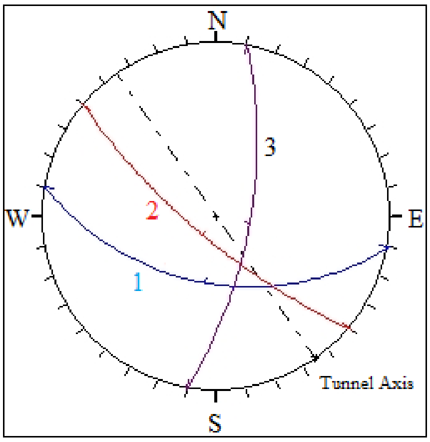

Figure 8-1 : Stereonet (Draw using Unwedge)

Joint Properties parameters

Assume water pressure and waviness are zero MPa .

Shear Strength

Model used: Barton-Bandit (Refer Chapter 5.5)

JRC: 10 (Barton and choubey, 1977)

JCS: 78 MPa

$\emptyset_r$ =30 (Himal Hydro 2009)

8.3 Result:

Result from the wedge analysis are shown in Figure 8.2 to 8.7.For a given factor of safety and input parameters, the results are support pressure, weight of blocks, their location and moods of failure are presented here.

Alternative I:

Few wedges are formed in all sections of the cavern alignment. Even though the required support pressure is zero in all location of each sections of cavern, available factor of safety is in decreasing order from section A-A to B-B and to C-C. It suggests installing relatively more support as the excavation go further inside from Section A-A to C-C. Factor of safety in each location is greater than one. If we need more factor of safety, it needs to install more support system to protect from sliding / falling of the wedges. Wedges from the roof have highest weight but with available factor of safety of more than 2.5.If we need the design factor of safety greater than this value, it is necessary to installed support. Here it assumed as safe.

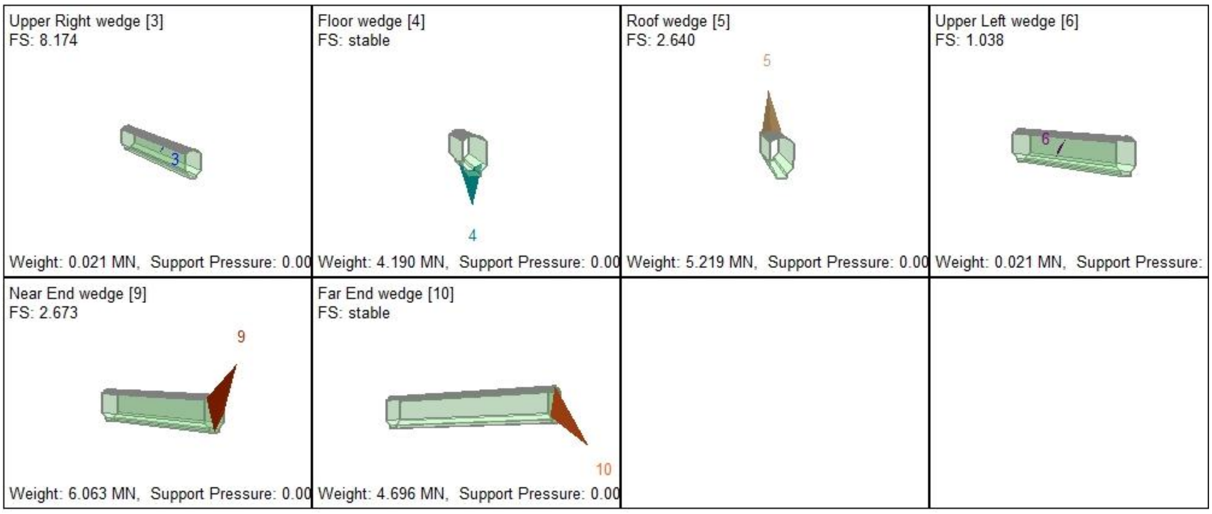

Figure 8-2 : Multi perspective view of wedge failure for alternative I (Section A-A)

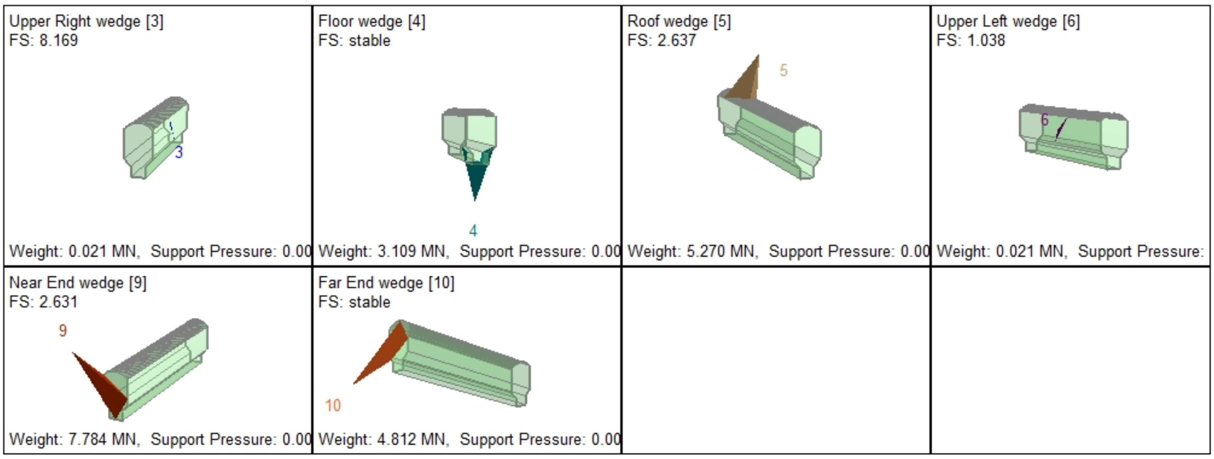

Figure 8-3 : Multi perspective view of wedge failure alternative I (Section B-B)

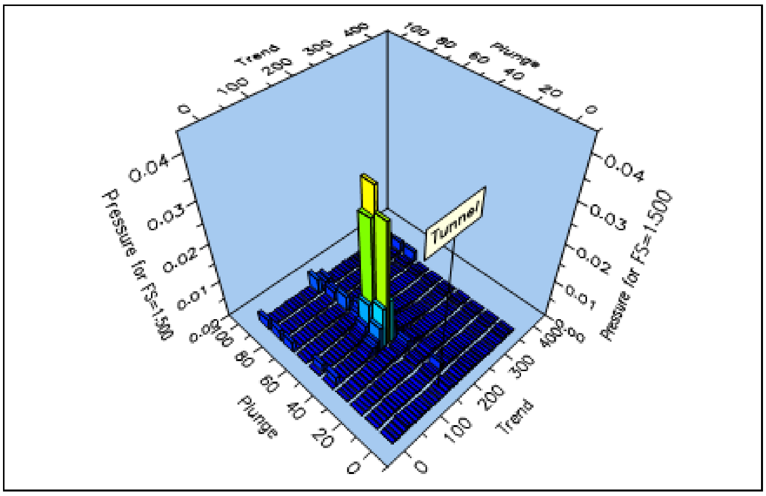

Figure 8-4 : Pressure plot for factor of safety 1.5, Alternative I (Section C-C)

Since the pressure for factor of safety (1.5) is very low at designed plunge and trend of settling basin, orientation of cavern at section C-C is perfect.

Alternative II:

Similar to alternative I, weights of wedges are increased as the excavation progress ahead from section A-A to B-B. Factor of safety is always greater than one. Roof is most vulnerable zone because of heaviest possible block failure. Since the factor of safety for the wedge of roof is about 2.3, which is greater than required factor of safety (1.5), it is not necessary to install the support system to prevent the block failure. Figure 8-5 to 8-7 shows the failure conditions of different blocks at different location of the caverns.

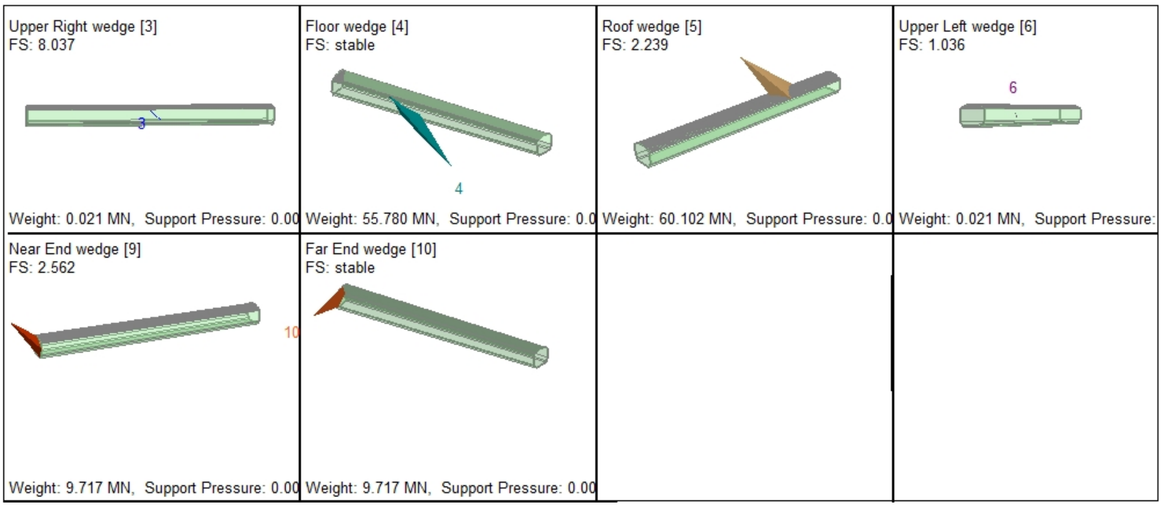

Figure 8-5 : Multi perspective view of wedge failure alternative II (Section A-A)

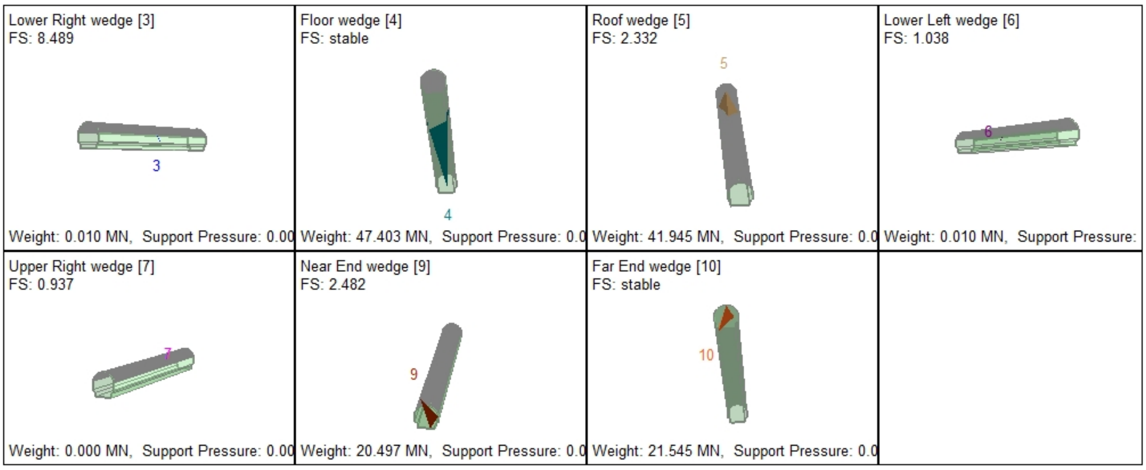

Figure 8-6 : Multi perspective view of wedge failure alternative II (Section B-B)

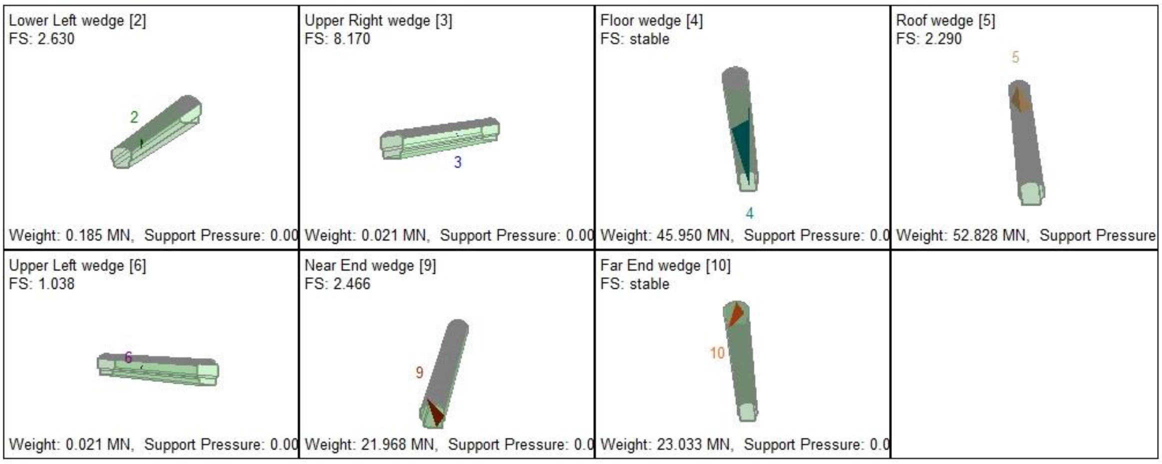

Figure 8-7 : Multi perspective view of wedge failure alternative II (Section C-C)

8.4 Conclusion and Recommendation

For both alternatives, available factor of safety is decreasing as the excavation progressed further from sections A-A to B-B and B-B to C-C. Roof and floor are more susceptible to failure with compare to other locations. Factor of safety is always greater than desired value (1.5). Therefore, it is concluded that relative orientation of the tunnel alignment (N145E) with respect to the joint sets and discontinuities characteristics is appropriate.

CHAPTER 9

9 CONCLUSION AND RECOMMENDATION

9.1 Conclusion

The main task of this thesis is to analyze the alignment for existing underground settling basin caverns. Another major task is to describe and evaluate the alternative layout, orientation, and shape of the caverns. Stability analyses along with designing of required support system for both alignments (Existing caverns and Alternative caverns) have also been performed. Empirical methods have been used for rock mass classification and predicting the rock support requirement. Empirical and semi analytical methods have been used to analyze the stability of the underground caverns. Finally, Numerical analysis has been performed to select the best shape of the caverns, to analyze the stability condition and to design optimum rock support system.

The underground caverns lies in the lower part of Higher Himalayas and mainly dominated by high-grade metamorphic rocks like gneiss. Both the existing and proposed settling basin caverns are located in steep hillside to the left bank of the Madi River and relatively close to the ground surface. There is an assumption of no major faults and weakness zone in the area of settling basin caverns. There are not sufficient field /laboratory tests to estimate the input parameters for the stability analysis of the underground structure. Therefore, several assumptions and references from previous reports / studies have been carried out and discussed in the respective chapters. Horizontal Tectonic stress is assumed as six MPa along south to north direction. Resolving components of tectonic stresses along and in perpendicular to the cavern have been used for the analysis. In the analysis, total horizontal stress is taken as the summation of tectonic and component of gravity stresses.

9.1.1 Settling basin cavern

- Alignment of existing settling basis cavern (N145E) is satisfactory but not the best one. It is because, the existing alignment is close to the Joint set (JS2) and Joint set (JS3),which are relatively stepper and have created the stability problems at the wall. While considering the orientation of all joint sets, the best alignment would be approximately N48E.Unfortunately this alignment is not feasible due to restriction imposed by locations of other underground structures and topography.
- There might have many errors in the analysis and in the results as well. It is due to lack of reliable input parameters such as UCS value, Joints characteristics parameters, stress measurement etc.
- Locations of caverns are at suitable depth, which ranges from 92m to 272m from the ground level. There is no space available to pull out the cavern near by the valley side, so that the stability problems could be reduced.
- Shapes of the caverns are optimized to some extent but still have possibility to make better shape to reduce the stability problems. Optimization of shape has been done through numerical analysis using $Phase^2$software.

- From the Empirical Analysis
    - Empirical and semi analytical analysis show, there are no chances of squeezing problems in any sections of both alternatives. Maximum total strain is only 0.082 % at section C-C of proposed alternative.
    - Rock spalling and rock bursting problems are created in sections B-B and C-C of both alternatives whereas the section A-A is almost stable.
    - RMR method suggests, the caverns shall have stand up time of less than one month for proposed alternative (Alternative II), whereas it is about six month for the existing alternative (Alternative I).This empirical method suggest to installed immediate rock support for the case of alternative II.
    - Q system suggests, both the alternatives need some support systems to make it stable. Proposed alternative needs relatively more support system, compare to existing alternative, which is due to larger span. Sections A-A, B-B, and C-C for both alternatives have relatively low, medium and high stability problems respectively. It is because of increasing rate of overburden depth and vertical nature of major principle stresses. Therefore the rock support requirements are also extensive, medium and low for section C-C,B-B and A-A respectively. Types, dimensions, and geometry of required rock support systems, which are based on Q system, have been discussed in chapter 6.
    - Empirical methods do not consider the effect on the rock mass between the outer walls of two caverns (In alternative I) and it does not have flexibility on consideration of shape of cavern. Therefore numerical analysis for stability assessment and rock support estimation has been carried out.
- Numerical modeling is the tool for analyzing the complex underground openings. It gives quite better results than the empirical and analytical approach. Results from the numerical analysis using $Phase^2$ gives almost the similar results with empirical and semi analytical approach.
    - For both alternatives, Major principle stresses are acting along the wall of the caverns. Due to high tangential stresses along the walls of the cavern, there are few stability problems like rock bursting and spalling. In case of alternative I, roof is almost stable but for alternative II, few finite elements are yielded due to longer span. Rate of increase of tangential stress along the wall is high while excavate from section A-A towards section C-C. Therefore, instability problems at wall are also in increasing from section A-A towards C-C.
    - The Total displacements and stress related problems are very less in all sections of both alternatives. It may be due to limitation of $Phase^2$ to account the effect of discontinuities. Therefore, the deformation may be higher in actual case than the computed values. Lack of consideration of all the minor joint sets that prevails in the rock mass may be one of major cause of error in the analysis. Therefore, the rock support suggested by empirical relation has been better option and suggested to install.

- In both type of analysis, Time progressive deformation along and in a radial direction of the cavern has not considered. Consideration of time dependent deformation in such a elastic-brittle-plastic rock mass is vital
- Effect of ground water is not considered as it may create problem during excavation. Therefore, it is suggested to make drain holes to pass out the possible water.
- Wedge failure analysis shows that the alignment of the cavern is good. All the detach wedges have factor of safety more than one, which means the cavern is stable without support system as well.

9.2 Recommendation:

Based on empirical and numerical analysis of settling basin cavern the following recommendations have been made:

While considering the stability problems, both the alternatives are feasible to construct. Rock support requirement for proposed alternative is slightly more than existing alternative. The cost benefit analysis is recommended to select the most economic alternatives that is beyond the objective of this thesis.

- Additional investigations should be performed to know the tectonic stresses, UCS value, discontinuity character of joints etc.
- Rock supports suggested by Q system have been recommended to install (Refer chapter 6).
- It is highly recommended to analyze in detail to find the best shape that gives the least stability problem.
- It is recommended to increase the outer spacing between the caverns from 9.5 m to at least 12 m, which reduces the interference problem during excavation of opening. It also makes suitable to install long rock bolt from both side of caverns.
- Since the dimensions of evaluated rock support system are based on stability analysis, it is recommended to fix the exact dimensions based on market availability during construction period. During fixing the exact dimensions, it is not permitted to reduce the calculated dimensions that are suggested in chapter 6.
- It is recommended to monitor during construction. Convergence measurements are needed to know the displacement. It is nice to perform the back analysis for all type of numerical analysis.
- Stability analysis of cavern is only performed in a static condition. Fluctuation of water (Emptying and filling) may create some stability problem. Therefore, it is recommended to perform dynamic analysis of the cavern before constructing.
- It is recommended to perform uncertainty analysis so that it would be possible to evaluate the probable error in the output parameters due to error in the input parameters.

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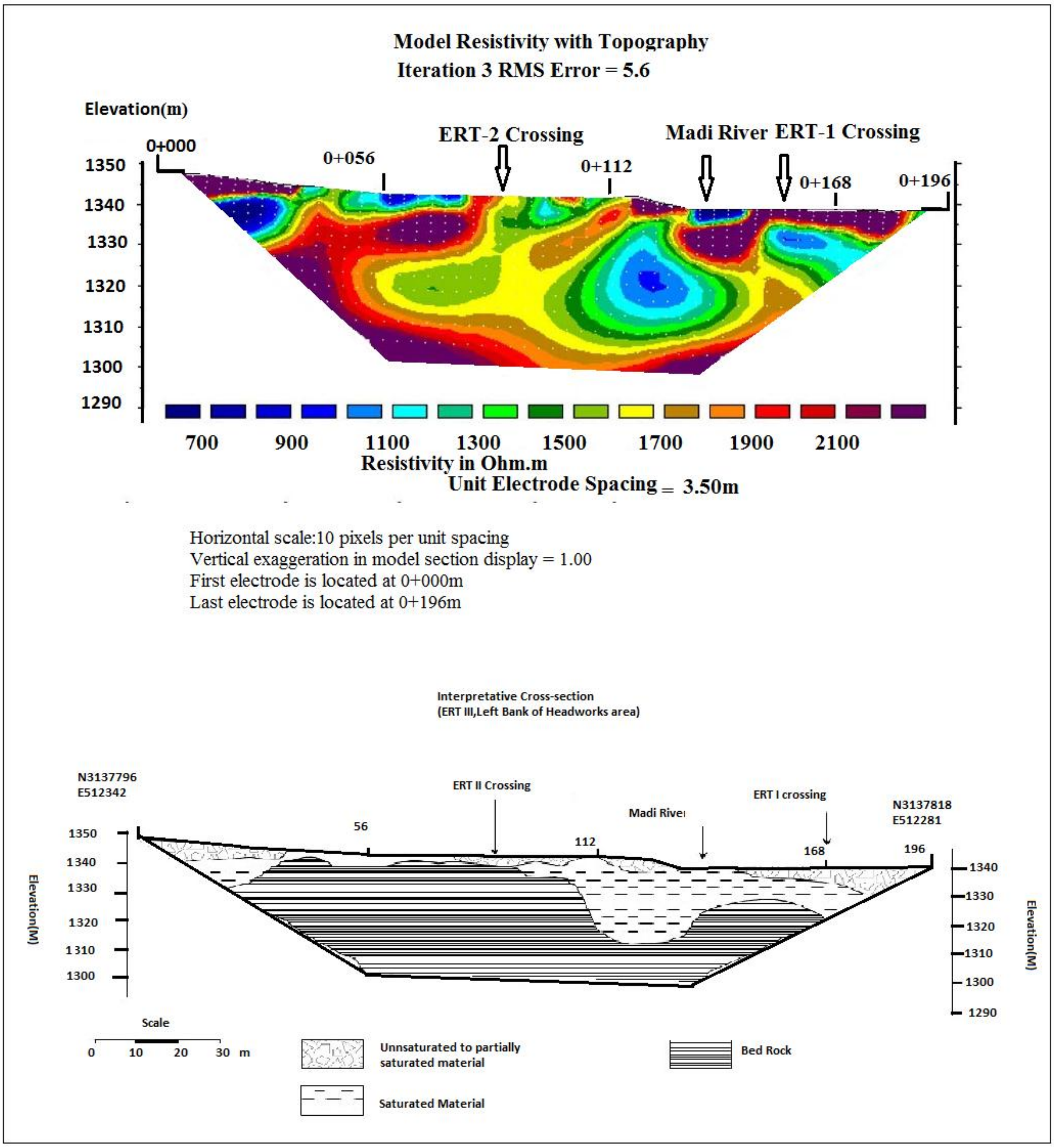
APPENDIX A
FIELD INVESTIGATION

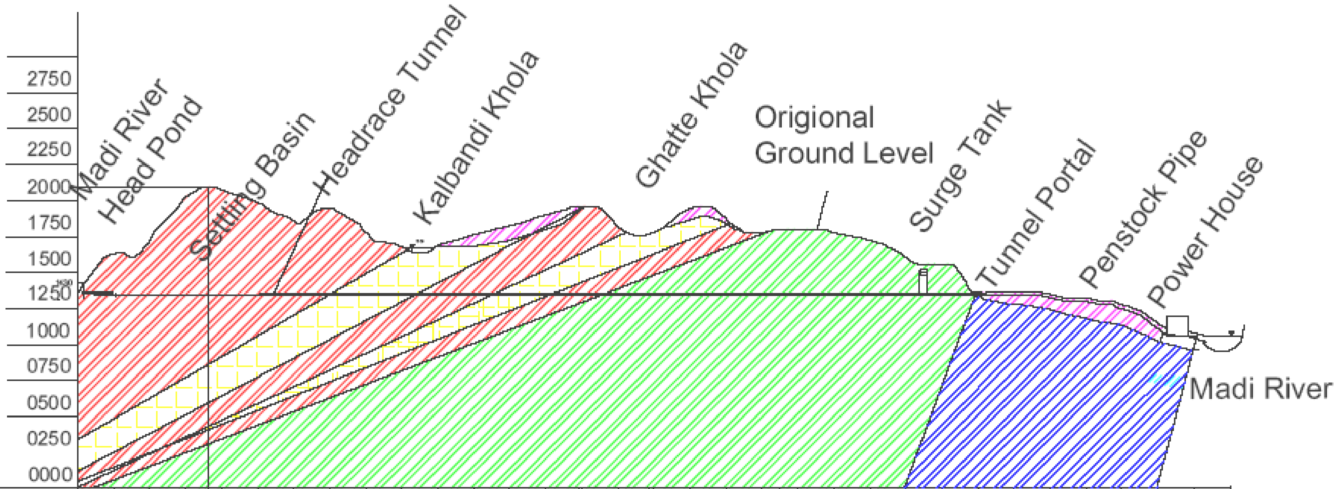
2-D ERT Profile with Corresponding Interpretive Cross-section at Headworks area (Himal Hydro 2009)



Rock showing the starting point for inlet tunnel at intake area(HH 2009)

Intake Location(HH 2009)

Longitudinal geological profile from intake to powerhouse



Chainage(M)	Ground Level(M)
0+0000	1367.5
0+0250	1613.2
0+0500	1615.0
0+0750	1858.0
0+1000	2055.0
0+1250	2073.5
0+1500	2008.0
0+1750	1904.8
0+2000	1884.5
0+2250	1928.3
0+2500	1781.3
0+2750	1698.5
0+3000	1640.0
0+3250	1717.5
0+3500	1776.0
0+3750	1827.5
0+4000	1888.0
0+4250	1943.5
0+4500	1958.5
0+4750	1784.8
0+5000	1780.8
0+5250	1919.8
0+5500	1959.2
0+5750	1800.04
0+6000	1788.0
0+6250	1800.0
0+6500	1797.5
0+6750	1747.0
0+7000	1689.5
0+7250	1573.5
0+7500	1560.0
0+7750	1369.5
0+8000	1340.0

RMR VALUE	75-80	70	75-80	9-12	64-72	15-20	60	55-60	55-60	60-65	55-60
Q-Value	10-14.1	4.8	10-14.1		6	0.24-0.32	2.0	4.1-8.3	1.1-3.6	5-9.1	2.29-4.1

Legends

Banded Gneiss
Weak Zone
Garnetiferous Schist
Crushed Rock
Colluvial Deposit
Alluvial Deposit

Scale
0 m
1250 m

NTNU	Master Thesis	PRPD BY:UMESH GAUTAM	TITLE: Geological Profile of Supermadi Hydroelectric Project,Nepal	Dwg No
Date: June 2012		DGND BY:Himal Hydro		3
		APPROVED BY:		

APPENDIX B

CALCULATIONS

APPENDIX B

Stress calculation for unit crosssectional area of Settling Basin Cavern of SMHEP

Rock Type	Banded Gneiss	Poission ration	0.2 For Gneiss
Tectonic stress(In Plane)	1.29 Mpa	Specific weight of water	0.01 MN/M3
Tectonic stress(Out Plane)	5.93 Mpa	Specific weight of rock	0.027 MN/M3
Rock mass class	Class II	UCS of Intact Rock	78 Mpa
Average Q-Value	12	Rock Mass Strength	11.5 MPa
RMR	65	Youngs Modulus of Elasticity of Intact Rock(Gpa)	27 Gpa
GSI	60	Deformation Modulus	3.97

	Slope Angle (Degree)	Avg over Burden (m)	Vertical Stress (Mpa)	Value of K	Total Horizontal Stress(In plane) (Mpa)	Total Horizontal Stress(Out plane) (Mpa)	Resolved principle stresses (Mpa)		Major Principles Stress (Mpa)	Minor Principle Stress (Mpa)
							Along ground surface	Perpendicular to ground surface		
Alternative I										
Section A-A	70	92	2.484	0.582348	2.736552	7.376552	3.27121826	3.421961122	3.421961122	2.736552
Section B-B	80	218	5.886	0.40644	3.682308	8.322308	6.437846319	4.652107946	6.437846319	3.682308
Section C-C	73	272	7.344	0.380941	4.087632	8.727632	8.219346341	6.059962815	8.219346341	4.087632
Alternative II										0
Section A-A	70	92	2.484	0.582348	2.736552	7.376552	3.666271808	3.604142273	3.666271808	2.736552
Section B-B	80	218	5.886	0.40644	3.682308	8.322308	6.524684124	6.905528866	6.905528866	3.682308
Section C-C	73	272	7.344	0.380941	4.087632	8.727632	7.622531054	8.34439182	8.34439182	4.087632

Assumption: Outplane horizontal stress is not considered in the calculation

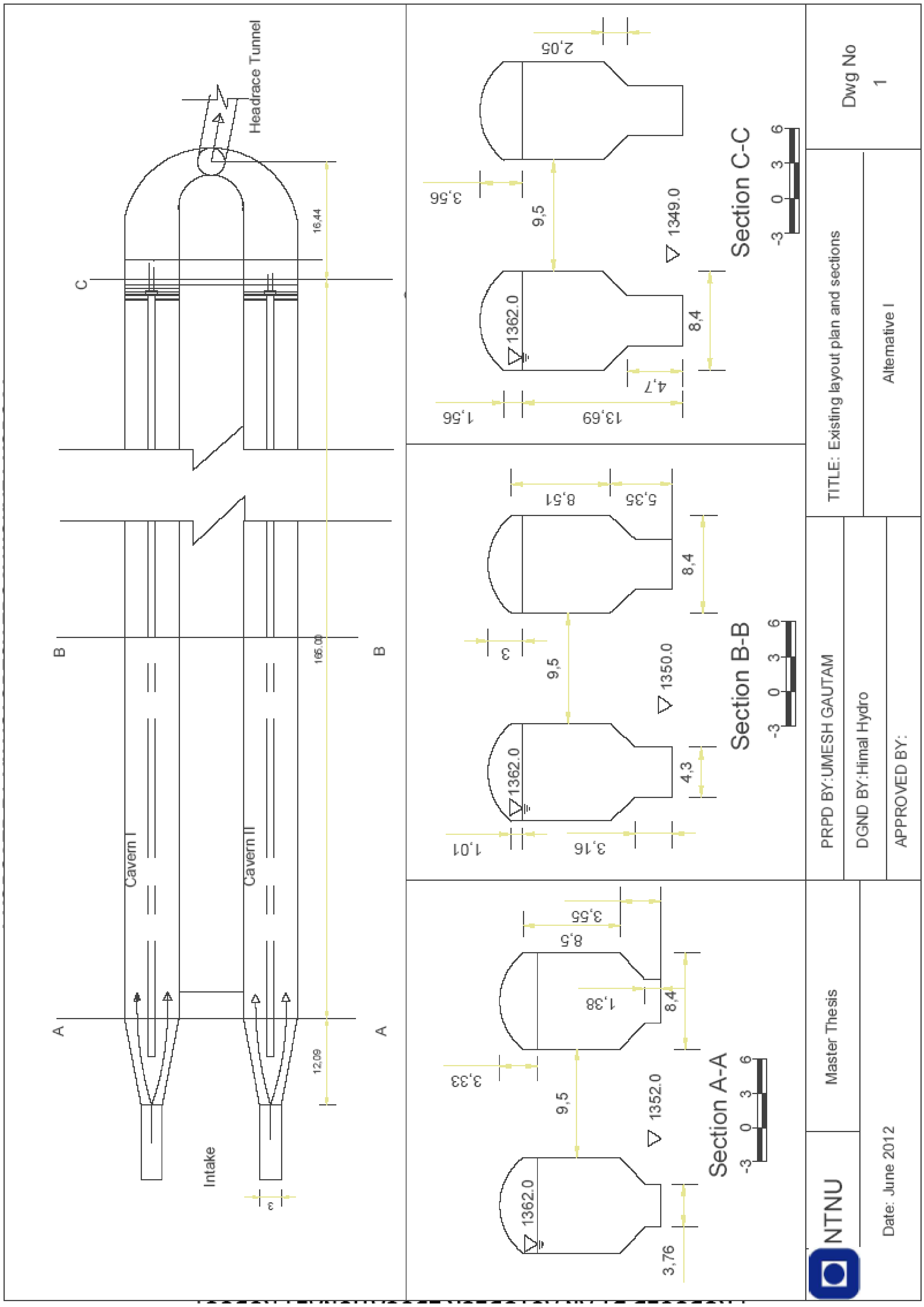
Note:To calculate the rock mass strength Panthi 2006 is used

Shorey Equation $K = 0.25 + 7\,E_h\,(0.001 + \frac{1}{z})$ Where, E_h is the average deformation modulus of upper and lower crust

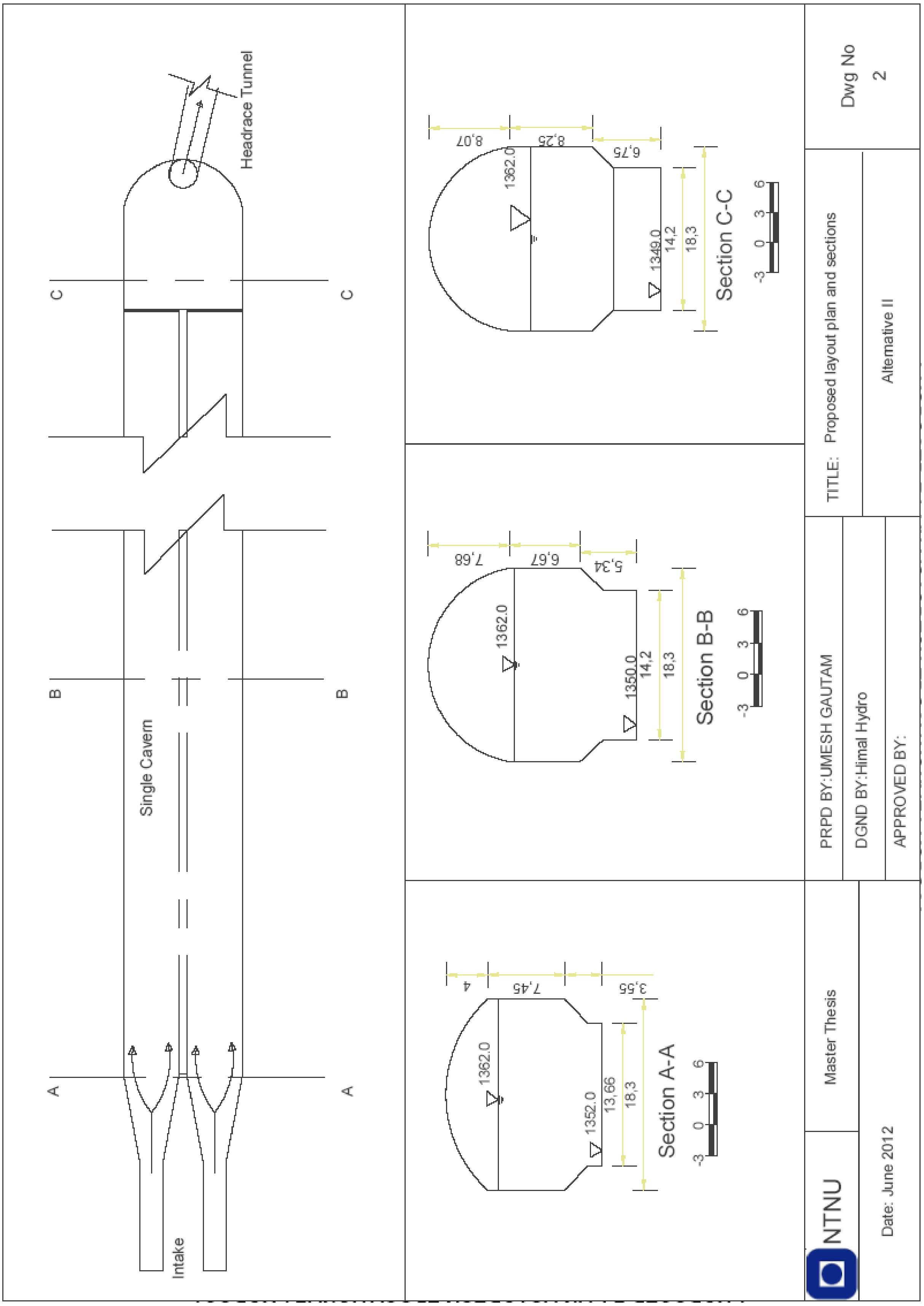
APPENDIX C

DRAWINGS

Existing Layout plan and sections at three different location of settling basin cavern of SMHEP (Himal Hydro 2009); (Alternative I)



<u>Proposed Layout plan and sections at three different location of settling basin cavern of SMHEP (Alternative II)</u>



APPENDIX D
STANDARD CHARTS

Rock Mass Classification System

Q-SYSTEM

Based on analysis and evaluation of approximately 200 tunnel cases, Barton et al (1974) of the Norwegian Geotechnical Institute (NGI) proposed the Q-system of rock mass classification. The Q-system gives useful correlation between Q-value and tunnel rock support. This method has got major updates in 1993 with the inclusion of data base from more than 1000 tunnel cases (Grimstad and Barton, 1993). Several papers have been published on the Q-system aiming to extend its applications in the estimation of rock mass properties (Barton, 2002 and Grimstad et al, 2003). In principle, this system is based on a numerical assessment of six different input parameters as defined by Equation A-1. The numerical estimation of these six input parameters according to Barton (2002) is presented in Table A-1.

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

Where; *RQD* is the rock quality designation, J_n is the joint set number, J_r *is the* joint roughness number, J_a is the joint alteration number, J_w is the joint water reduction factor, and *SRF* is the stress reduction factor.

Table A-1. Description of ratings for input parameters of Q-system (based on Barton, 2002).

RQD (Rock quality designation, %)		***J_n (Joint set number)***	
Very poor	0 - 25	Massive, no or few joints	0.5 - 1
Poor	25 - 50	One joint set	2
Fair	50 – 75	One joint set + random joints	3
Good	75 – 90	Two joint sets	4
Excellent	90 - 100	Two joint sets + random	6
Notes:		Three joint sets	9
(i) where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q.		Three joint sets + random	12
		Four or more joint sets, heavily jointed, sugar cube etc	15
(ii) RQD intervals of 5 i.e. 100, 95, 90 etc., are successfully accurate.		Crushed rock, earthlike	20
		Note: For tunnel intersections, use (3 x J_n) and for portals use (2 x J_n)	

J_r (Joint roughness number)

(a) Rock wall contact		*(b) Rock wall contact before 10 cm shear*	
Discontinuous joints	4	Rough or irregular, undulating	1.5
Rough or irregular, undulating	3	Smooth, undulating	1
Smooth, undulating	2	Slickensided, undulating	0.5
Slickensided, undulating	1.5		

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

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

J_a (Joint alteration number)

(a) Rock wall contact (no mineral fillings, only coatings)	*ϕ_r (appr.)*	*J_a*
Tightly healed, hard, non-softening, impermeable filling i.e. quartz/epidote	-	0.75
Unaltered joint walls, surface staining only	25 - 35	1
Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay free disintegrated rock ,etc.	25 - 30	2
Silty or sandy clay coatings, small clay fractions (non-softening)	20 - 25	3
Softening or low friction clay mineral coatings, i.e., kaolinite or mica. Also chlorite, talk, gypsum, graphite etc., and small quantities of swelling clay	8 - 16	4
(b) Rock wall contact before 10 cm shear (thin mineral fillings)		
Sandy particles, clay free disintegrated rock etc.	25 - 30	4
Strongly over-consolidated non-softening clay mineral fillings (continuous, but < 5mm thickness)	16 - 24	6
Medium or low over-consolidated non-softening clay mineral fillings (continuous, but < 5mm thickness)	12 - 16	8
Swelling clay fillings, i.e., montmorillonite (continuous, but < 5mm thick)	6 - 12	8 - 12
(c) No rock wall contact when sheared (thick mineral fillings)		
Zones or bands of disintegrated or crushed rock and clay	6 - 24	6, 8 - 12
Zones or bands of silty or sandy clay, small clay fraction (non-softening)	-	5
Thick, continuous zones or bands of clay	6 - 24	13 - 20

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

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

SRF (Stress Reduction Factor)

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

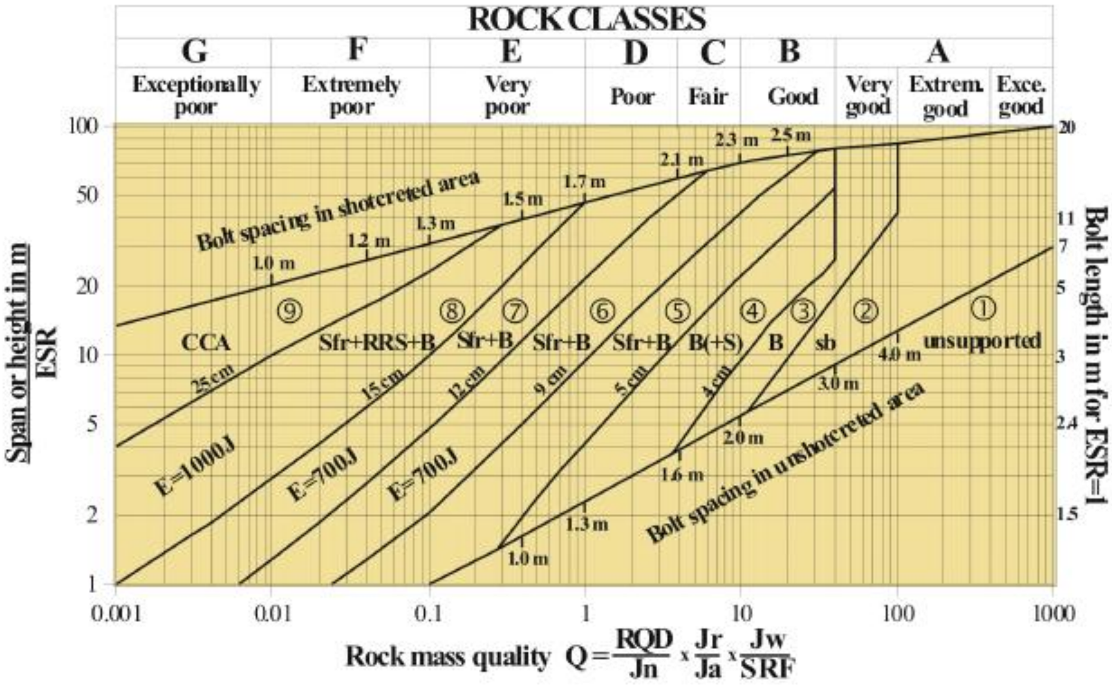
Note: Reduce these values of SRF by 25 – 50 % if the relevant shear zones only influence but do not intersect the excavation.

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

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

© Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure	σ_t / σ_c	*SRF*
Mild squeezing rock pressure	1 - 5	5 - 10
Heavy squeezing rock pressure	> 5	10 - 20
(d) Swelling rock: chemical swelling activity depending on pressure of water		*SRF*
Mild swelling rock pressure		5 - 10
Heavy swelling rock pressure		10 - 15

In the original version of Q-system, Barton et al (1974) defined rock mass quality in nine different classes ranging from 'exceptionally poor' to 'exceptionally good' and the Q-value was correlated with actually applied rock support measures in the tunnels. In 1993, Grimstad and Barton modified the Q-system, particularly its support chart and inclusion of squeezing conditions on the *SRF* rating. The most recent version of the support chart is shown in Figure A-1.



REINFORCEMENT CATEGORIES:
1) Unsupported
2) Spot bolting, **sb**
3) Systematic bolting, **B**
4) Systematic bolting, (and unreinforced shotcrete, 4-10 cm), **B(+S)**
5) Fibre reinforced shotcrete and bolting, 5-9 cm, **Sfr + B**
6) Fibre reinforced shotcrete and bolting, 9-12 cm, **Sfr+B**
7) Fibre reinforced shotcrete and bolting, 12-15 cm, **Sfr+B**
8) Fibre reinforced shotcrete > 15 cm, reinforced ribs of shotcrete and bolting, **Sfr+RRS+B**
9) Cast concrete lining, **CCA**

E) Energy absorption in fibre reinforced shotcrete at 25 mm bending during plate testing

Figure A-1. Updated rock support chart for tunnels and caverns (after Grimstad et al 2003).

As can bee seen in Figure A-1, the rock support chart incorporates the equivalent excavation dimension, which is the ratio between the span or height of an underground opening and an excavation support ratio (ESR). According to NGI (1997), the ESR reflects the degree of safety and support required for the underground opening. Its value varies from 5 to 0.5 depending upon the type of underground excavation. For instance, an ESR value of 1.6 is used for water tunnels, permanent mine openings, adits and drifts and an ESR value of 1 for underground power house, road tunnels, railway tunnels, and civil defense chambers. For more details, reference is made to NGI (1997).

In the author's opinion, the main strength of the Q-system is the well described support chart, which is useful particularly for quantitative estimation of tunnel rock support at the pre-construction phase of the project, when there is a great need for reliable and trustworthy economic evaluation.

According to Stille and Palmstrøm (2003), the major criticism that the Q-system has received is connected to the SRF. This is because, as shown in Table A-1, it represents four complicated factors; weakness zones, stress influence in brittle, blocky and massive ground, stress influence in deformable (ductile) rock mass and swelling rocks. Barton (2001) argues that since the Q-system is an empirical design tool based on applied rock support of already completed project cases, it is logical that water and stress parameters are components of the classification systems.

RMR-SYSTEM

Bieniawaski already in 1973 introduced an empirical rock mass classification system called rock mass rating (RMR) system (also known as Geomechanics Classification). Over the years, this classification system has been modified as more case histories have become available. The last modification was made in 1989 (Bieniawaski, 1989). Basically, the RMR-system uses the following six parameters to classify a rock mass:

1. Uniaxial compressive strength of the rock
2. Rock quality designation (RQD)
3. Spacing of discontinuities
4. Condition of discontinuities
5. Ground water conditions and
6. Orientation of discontinuities

These classification parameters are evaluated by field measurements as shown in Table A-2 giving a numerical rating value to each parameter. The rating of each of these parameters is summarized to give a value of RMR, which can be used to define rock mass quality and its class. The RMR is also related to stand-up time as shown in Figure A-2.

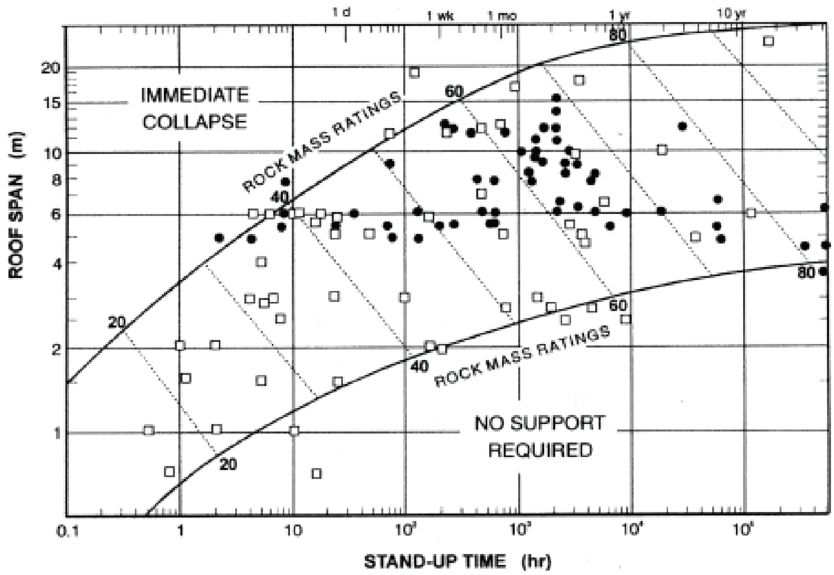


Figure A-2. Stand-up time of an underground opening as a function of roof span and RMR value (Bieniawaski, 1989).

Table A-2. RMR classification of rock mass (Bieniawaski, 1989).

A. Classification parameters and their ratings

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

B. Rating adjustment for discontinuity orientation					
Tunnel alignment	very favor-able	favor-able	fair	unfa-vorable	very unfavorable
Rating adjustment	**0**	**-2**	**-5**	**-10**	**-12**
C. Rock mass classes determined from total ratings					
Rating	100-80	80-61	60-41	40-21	< 20
Class No.	I	II	III	IV	V
Description	Very good	Good	Fair	Poor	Very poor
D. Meaning or rock mass classes					
Class No.	I	II	III	IV	V
Average stand-up time	Can be estimated from Figure 4-4				
Cohesion of the rock mass (kPa)	> 400	3-400	2-300	1-200	< 00
Friction angle of the rock mass (degrees)	< 45	35-45	25-35	15-25	< 15

The main strength of the RMR-system is its relationship with the standup time, see Figure A-2, and its correlation with rock mass properties and with Hoek-Brown failure criterion to estimate constants such as *m* and *s*. The major deficiency is that its lacks a good system for prediction of rock support measures (Bieniawaski suggested a support table that is suitable only for a single sized tunnel with 10 meters span).

When using the RMR system at the planning and design phases, the rock mass is divided into structural regions with uniform features within each region. During tunnel excavation, the rating is generally related to the length of the blasting round or the recently excavated tunnel section.

Field Estimate of Uni-axial compressive Strength (ISRM, 1978)

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

* Grade according to Brown [2]

** Point load tests on rocks with a uniaxial compressive strength below 25 MPa are likely to yield ambiguous results.

Rock mass class based on Q and RMR values

$RMR \approx 9 \times \ln Q + 44$ (Bieniawaski, 1989) $RMR = 15 \times \log Q + 50$ (Barton, 1995)

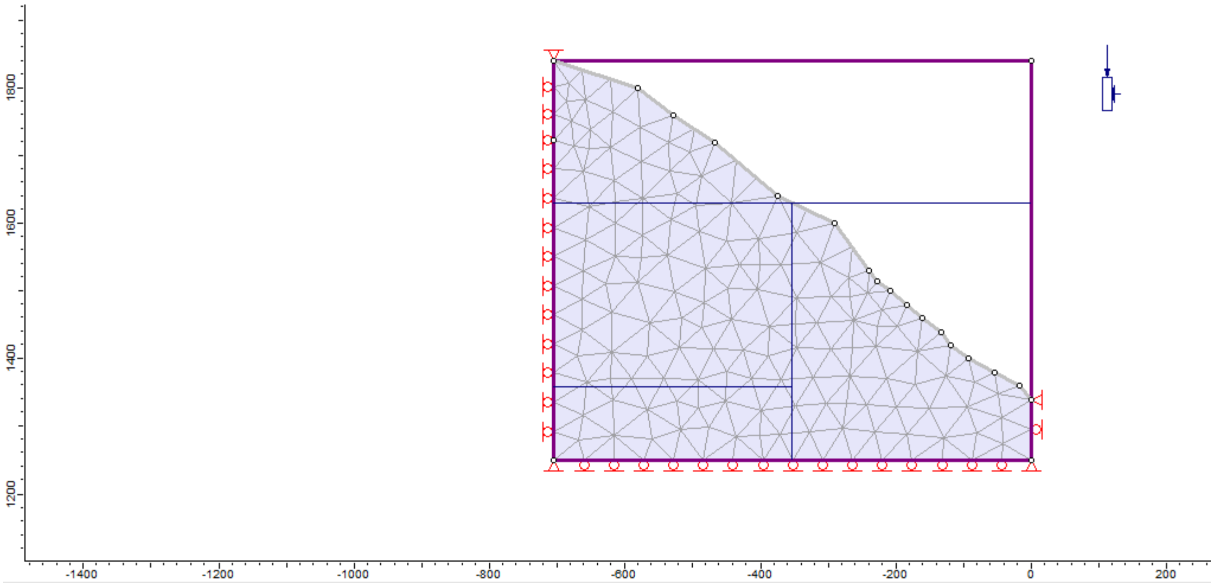
Descriptions		Range of Q-values		Range of RMR-values	
Rock Class	Quality descriptions	Minimum	Maximum	Minimum	Maximum
Class 1	Very good to excellent	100	1000	85	100
Class 2	Good	10	100	65	85
Class 3	Fair to good	4	10	56	65
Class 4	Poor	1	4	44	56
Class 5	Very poor	0.1	1	35	44
Class 6	Extremely poor	0.01	0.1	20	35
Class 7	Exceptionally poor	0.001	0.01	5	20

Weathering classification according to ISRM,1978

Term	Description of rock mass conditions	Weathering grade
Fresh rock	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surfaces.	I
Slightly weathered	Discolouration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering and may be some what weaker externally than in its fresh condition.	II
Moderately weathered	Less than half of the material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.	III
Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	IV
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	V
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI

APPENDIX E
PHASE ^{2}MODELS

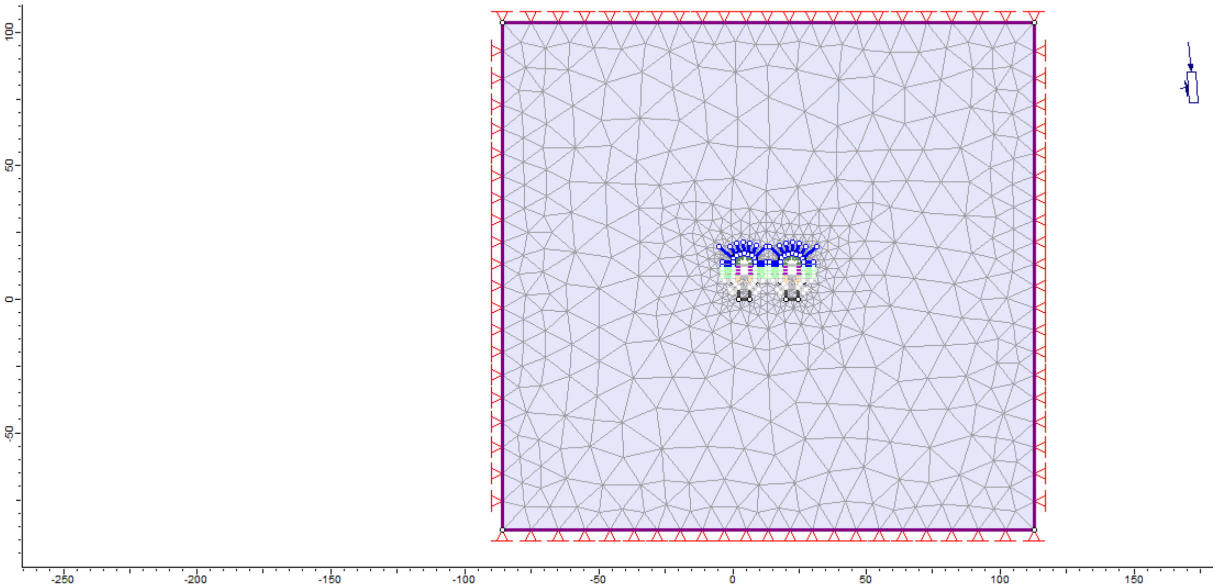
Valley Model



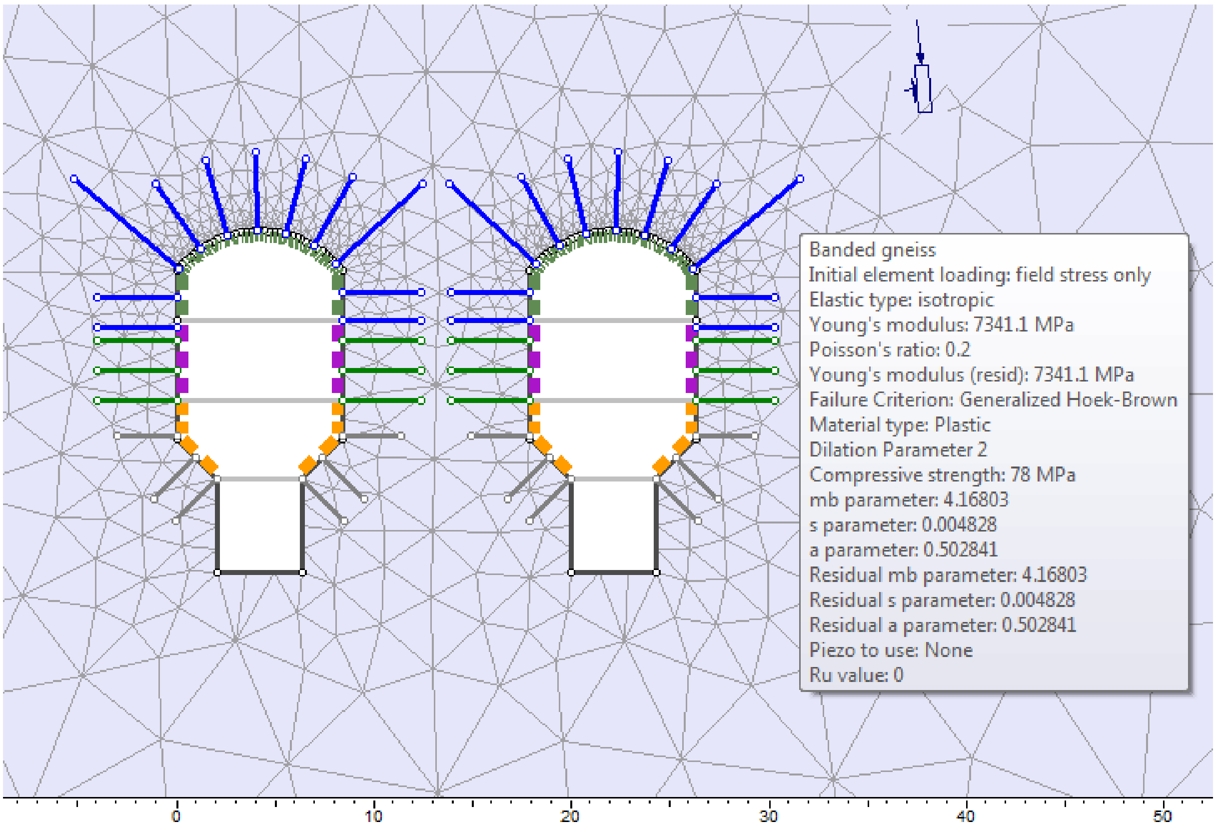
Valley model for section C-C

Confine Models

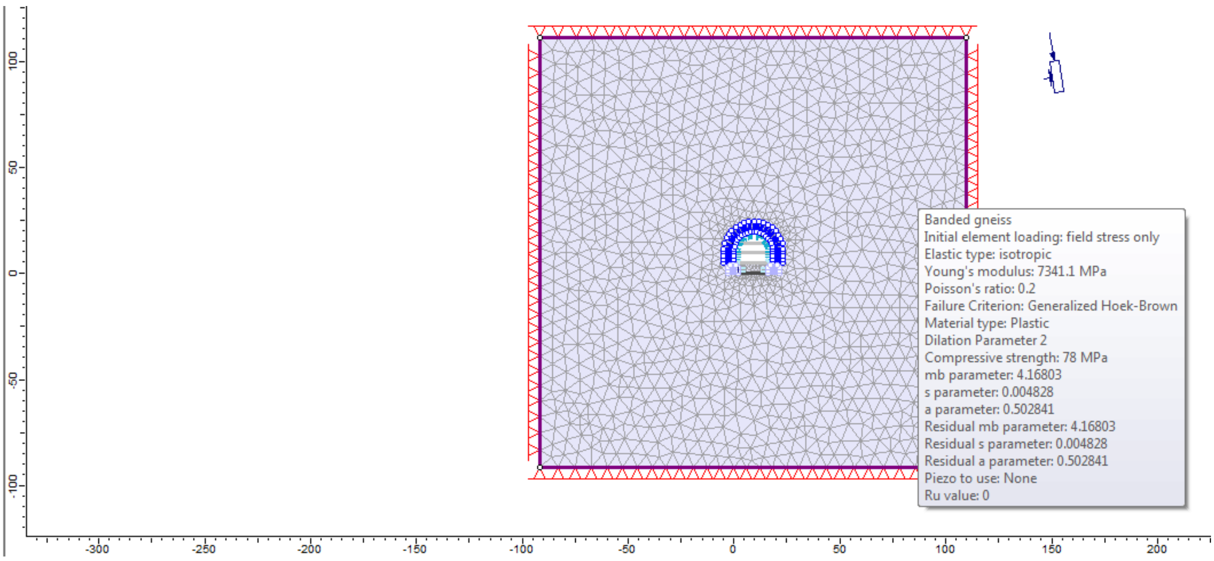
Stages of excavation for settling basin cavern



After second stage of excavation with support of Alternative I (Section C-C)



After final stage of excavation with support of Alternative I (Section C-C)



After third stage of excavation with support of Alternative II (Section C-C)