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DEPARTMENT OF CIVIL ENGINEERING

Design of Powerhouse and Assessment of Underground Structure

(Case Study of Super Madi Hydroelectric Project (SMHEP))

A Final Year Project

Submitted in Partial Fulfillment of the Requirements for the Degree of

Bachelor of Engineering in

Civil Engineering (with Specialization in Hydropower Engineering)

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September, 2020

Dedications

CERTIFICATION

FINAL YEAR PROJECT REPORT

ON

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DECLARATION

We, **Prajwol Acharya, Dinesh Dhakal, Abhishek Dongol, Ajay Ghimire, Prawal Keshar Khatri, Raghav Poudel** and **Utsav Shrestha** hereby declare that this project work titled **"Design of Powerhouse and Assessment of Underground Structure"** submitted in partial fulfillment of bachelor's degree in Civil Engineering (Specialization in Hydropower) to Department of Civil Engineering, during the academic year 2020, is a genuine work done originally by us under the supervision of **Asst. Prof. Dr. Shyam Sundar Khadka**. Any help from other people has been duly acknowledged.

The report or any part of it has not been published or submitted for the academic award or any other Universities or Institution. Any literature, data, or works done by others and cited within this report has been given due acknowledgement and listed in the reference section.

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EXECUTIVE SUMMARY

The utilization of underground space in Nepal for development of infrastructures is a challenge as well as an opportunity. Underground space, in form of caverns and tunnels are already in use by hydropower projects for water conveyance systems and to house components such as surge shafts, settling basins and powerhouse. However, the young and fragile geology of Nepal brings forward with it a host of different issues like squeezing, water leakage, collapse etc. It is necessary to understand these issues to optimize the support systems and stability of underground excavations.

Our project deals with the empirical, analytical and 2D numerical investigations into the tunnel and cavern of Super Madi Hydroelectric Project (SMHEP).Finite element Modeling results highlight that the shape and size of the tunnel, existing geological structures and rock-mass parameters have significant influence to the induced stress field and rock deformation, which directly controls the stability of the underground excavation design.

Specific Studies carried out in our project are:

- Empirical and analytical study of the ventilation tunnel (Bagalethar Adit)
- 2D FEM analysis of ventilation tunnel and settling basin caverns

A section of tunnel in very weak rock has also been studied in this project and numerical modeling of the forepole support has been carried out.

The design of surface powerhouse has also been carried out. Components specific to a powerhouse such as gantry girders, corbels and machine foundation have also been done. The structural detailing of the powerhouse was done on the basis of architectural drawings provided by SMHEP. Alongside, the control building for the powerhouse has also been designed. Dynamic analysis wasn't carried out but equivalent static method has been applied to account for earthquake forces as per IS 1893. Ductile detailing of the structure according to IS 13920 has also been carried out.

Finally the estimation of the powerhouse has also been done.

The computer software RS2 by rocscience has been used for 2D FEM of underground excavation and SAP2000 by CSI has been used for analyzing the members and components of powerhouse.

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Chapter 1 Introduction

1.1 Background Information

Nepal is situated within the southern slope of the Himalaya. The Himalayan region in Asia covers an area of about 594,400 square kilometres, of which Nepal occupies a total land of 147,181 square kilometres. For rapid development of infrastructures such as hydropower schemes, irrigation systems, road networks, drinking water systems etc. demands the need for the utilization of underground space like tunnels and underground caverns. Till now, tunnels and underground structures have only been in use for hydropower projects. In topography like that of Nepal, underground works for hydropower are essential and challenging. In recent times, there have been several plans to include tunneling for transportation purposes as well. In topography such as Nepal’s, there is great scope for implementing tunnels for development of infrastructure.

The construction and development of tunnel has been a major challenge in Nepal. The presence of major faults like Main Central Thrust (MCT), Main Boundary Thrust (MBT), and Himalayan Frontal Thrust (HFT), or Main Frontal Thrust (MFT) has added to the problem in tunneling and underground construction. Major problems encountered during tunnelling in fault zones are excessive overbreaks and collapses, large, heterogeneous and unsymmetrical displacements, long term creep, and excessive water inflows(Schubert et al., 2006). Also, the Himalayan region is located at the two-plate boundary: Indian and Eurasian Tectonic plates, therefore Himalayan region is considered seismically active zone(Karki et al., 2018). Fragile geological conditions and tectonic stresses have rendered the rock mass in the Nepal Himalayas as highly deformed, weathered, folded and faulted. Proper support system must be installed during underground construction. Under estimation of support system leads to failure of the structure while over estimation leads to unnecessary increasing cost of the project.

Problems have been encountered in tunneling in Nepal Himalayas. There exists major squeezing problem in headrace tunnel of Chameliya Hydroelectric project and the support system provided is not able to control the squeezing. Examples of severe squeezing in the constructed tunnelling projects in Nepal are 144 MW Kaligandaki “A” Hydroelectric Project, 14 MW Modi Khola Hydroelectric Project and 60 MW Khimti I Hydropower Project(K K Panthi, 2004a).An example of the water leakage problem is the Khimti headrace tunnel. About 200 liters per second of water was leaking from only a single location of Adit 2 of this tunnel(K K Panthi, 2004a)

The project location, Super Madi Hydroelectric Project is located at Kaski district, Nepal. The major components of the hydropower – settling basin, surge tank both lie underground. Two underground settling basins are fed with the discharge by two inlet tunnels from the headpond. An outlet pond after the settling basins feed water into headrace tunnel. The headrace tunnel of length 5905m of finished diameter of 3.6~4.4m .

1.2 Objectives of the project:

Objectives of the project are as follows:

a. Primary Objective
    - Support assessment and analysis of underground structure in the Himalayan region
    - Analysis and structural design of subsurface powerhouse in the Himalayan region

b. Secondary Objectives
    - Numerical analysis of Rock Mass of Tunnels and Caverns of SMHEP
    - Study the interaction between rock mass and rock support.
    - Stress analysis in underground excavation.
    - Analysis of subsurface powerhouse by using commercial software
    - Structural design and detailing of powerhouse structural elements

1.3 Scope

The study mainly focuses on the analysis of underground structure present in the Himalayan Region of Nepal. The project location lies in the Higher Himalayan Sequence.

- Analysis of underground structure present in the Himalayan Region of Nepal using various approaches viz empirical, analytical and numerical method.
- Project limited to case study of Super Madi Hydroelectric Project
- Recommendation of Support system
- Application of numerical modeling using RS2
- Design of powerhouse is according to Department of Electricity Development (DOED) guideline and dynamic analysis will not be done

1.4 Methodology

1.4.1 Literature review

- Review of several underground caverns and powerhouses in Nepal.
- Review of factors affecting underground structures.
- Review of preexisting empirical and analytical approach to evaluate rock squeezing and its impact on underground structures.
- Review on numerical method of investigation and its effectiveness comparison to other approaches.
- Study of stability analysis and deformation calculation.

1.4.2 Data collection

- Data will be obtained from the project site.

1.4.3 Stability analysis of rock

- Based on available data, stability analysis of rock will be carried out. Rock supports are provided according to type of rock available in the area. After successful construction of tunnel.
- The empirical approaches; Convergence Confinement Method (Carranza-Torres and Fairhurst, 2000) and Numerical investigation will be used

1.4.4 Design of powerhouse building

- Architectural Drawing is obtained from SMHEP and will be analysed using SAP2000 program.
- Design will be done according to Department of Electricity Development (DOED) guidelines

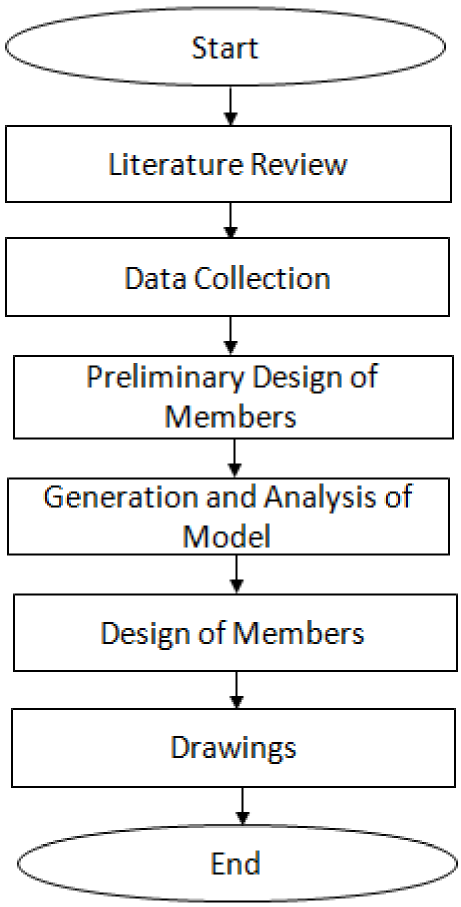


Figure 1-1 Methodology for Powerhouse Design

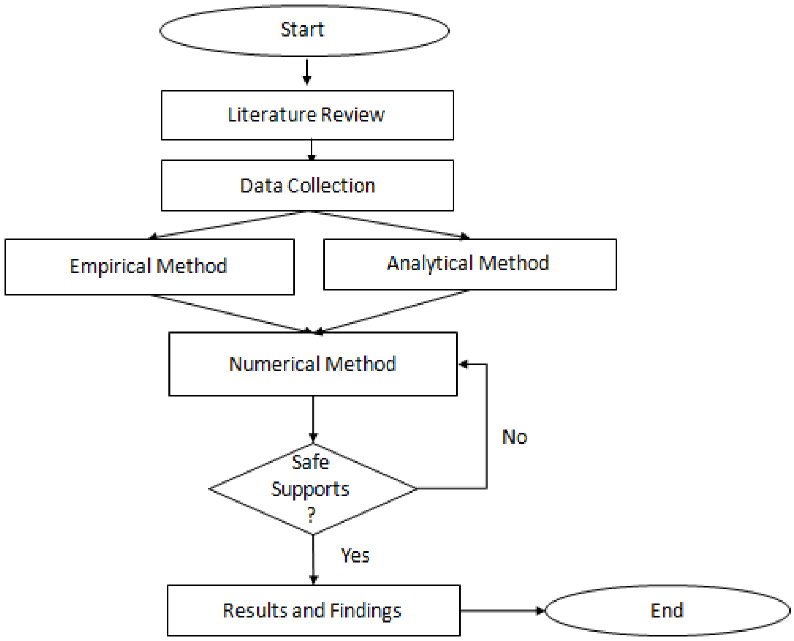


Figure 1-2 Methodology for Underground Structure

1.5 Limitations of the project

- Numerical modelling requires many rock parameters which needs to be performed in laboratory for precise measurement.
- Assumptions might increase errors in calculations the overburden calculated from profile map may not exactly match the existing overburden.

1.6 Organization of Chapters

The first chapter of the project is the introduction. In this chapter, we have discussed the preliminaries of our project i.e. the scope, methodology, limitations and objectives.

The second chapter is the literature review. This chapter contains mainly the background information on everything we have done in this project. We have discussed the geology of Nepal, tunneling in Nepal, and tunneling methods. Major focus is on the literature for various analysis and works done in this project.

The third chapter describes the description of the case study along with the empirical and analytical assessments applied for the ventilation tunnel (Bagalethar adit).

The fourth chapter is focused on the 2D numerical analysis. The analysis of the sections of tunnel and of the settling basin cavern has been described in this chapter. This section also deals with a case of tunnel in very weak and disintegrated rock mass.

The fifth chapter includes the structural design of the powerhouse and control building structure and the sixth chapter is the conclusion of this thesis

Chapter 2 Literature Review

2.1 Physiography of the Nepal Himalaya

Physiographic division of Nepal has been in practice since 1950s. It was 1969, Tony Hagen successively divided Nepal into eight well defined physiographic provinces from south to north. These provinces are E-W running and can also be incorporated in Indian Himalayan belt. The Hagen classification is the most appropriate classification and represents all characteristic physiographic zones of Nepal. Some geographer and geomorphologists also used fivefold classification in the general sense namely Terai, Chure, Middle Mountain, High Mountain and High Himalaya. Nevertheless, detail physiographical provinces of Nepal are given in **Table 2-1**and Figure 2-1and Figure 2-2illustrates the generalized physiographic profile of the Nepal Himalaya.

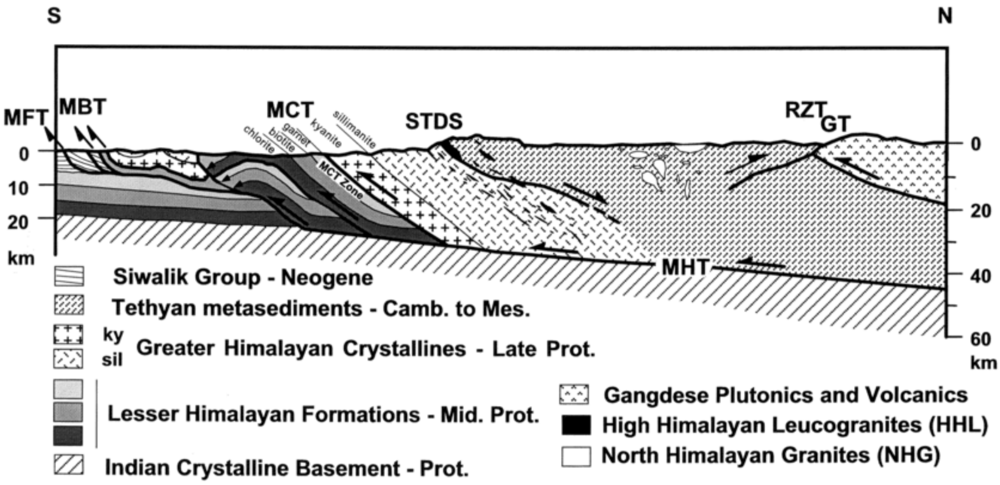


Figure 2-1 Generalized cross section of Himalaya(Harrison et al., 1999)

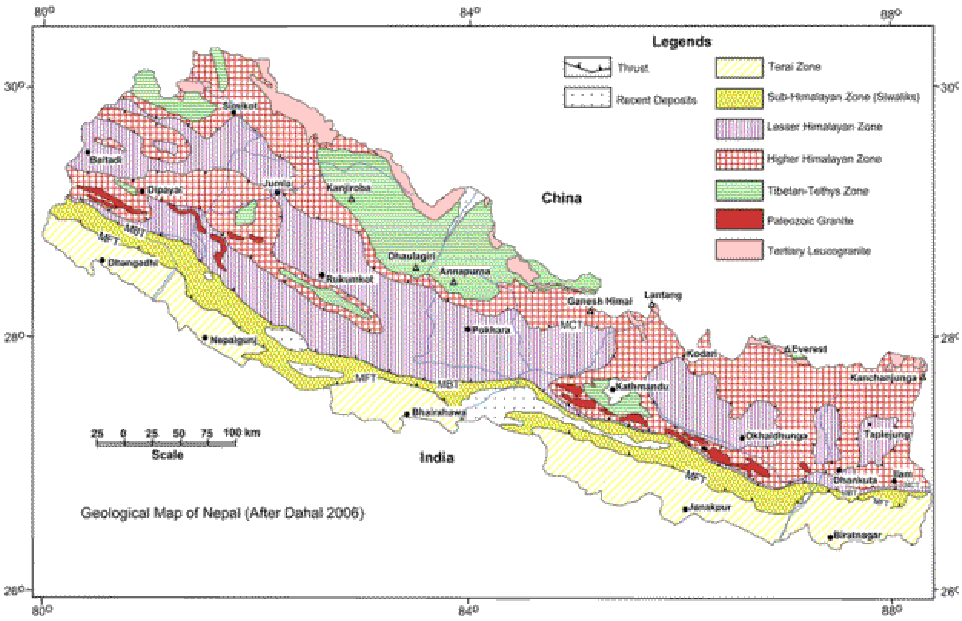


Figure 2-2 Geological map of Nepal (Dahal R.K, 2006)

Table 2-1 Physiographical division of the Nepal Himalaya (Upreti, 1999)

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

				northern edge of the Lesser Himalayan Zone	
7	Higher Himalaya	10-60	>5000	Gneisses, schists, migmatites and marbles belonging to the Higher Himalayan Zone	Tectonic upliftment, Weathering, erosion (rivers and glaciers), and slope failure
8	Inner and Trans Himalaya	5-50	2500-4500	Gneisses, schists and marbles of the Higher Himalayan Zone and Tethyan sediments (limestones, shale, sandstone etc.) belonging to the Tibetan-Tethys Zone	Tectonic upliftment, wind and glacial erosion, and slope degradation by rock disintegrations

2.2 Tunneling in Nepal

Nepal is a small landlocked country situated within the southern slope of the Himalaya between the Tibetan plateau and Gangetic plain. From north to south, Nepal has four distinct geological regions higher Himalaya, the mid-hills (lesser Himalaya),the Swaliks (Churia) and the Gangetic plane (Terai)(Panthi, 2004).

The geography of Nepal thus demands the proper utilization of underground spaces like tunnels and caverns to develop its infrastructures and to thrive on the economic development of the country. There are principally four areas where tunnels and underground caverns are needed in Nepal. They include: (Panthi 2004).

- water conveying tunnels,
- transport tunnels,
- mining and
- food storage facilities

For the time being most of the tunneling is focused on hydropower, and to some extent in mining and irrigation. In this respect, in recent past, the tunneling activities have increased considerably in the country with the development of many medium scale hydropower projects. This increased activity in tunneling has enabled to gain more knowledge in tunneling through the Himalayan rock mass. However, there are still many challenges to be faced and solved in future tunneling, since past tunneling experience has gave somewhat mixed feeling with respect to their successful completion. The majority of the tunneling projects developed in past have had suffered severe stability problems that made delay in completion and cost overruns. Most of these tunnel instabilities are related to the complex geological set up of the Himalaya that poses major challenge in solving the difficulties. In addition, the compressional tectonic stress regime in the Himalaya has resulted in intense deformation of the rock mass, making it highly folded, faulted, sheared, fractured and deeply weathered.(Panthi, 2008)The presence of major faults like Main Central Thrust (MCT), Main Boundary Thrust (MBT), and Himalayan Frontal Thrust (HFT), or Main Frontal Thrust (MFT) has added the problem in tunneling and underground construction geological setting has caused considerable stability problems (uncertainties) and is a great challenge for successful tunneling in Nepal.

For tunneling, it is crucial to have a method characterized by cost effectiveness and flexibility to adapt in changing ground conditions, and by accuracy in the prediction of rock mass quality during planning. The design phase decision in selecting tunnel alignment and predicting the rock mass quality and rock support requirement has direct influence on the overall cost and time requirement of any tunneling project.(Krishna Kanta Panthi, 2008).

2.3 Status of Tunnels in Nepal

The very first tunnel constructed in Nepal is the Chure road tunnel connecting Makwanpur and Bara. It is 500 m in length and was constructed in 1917. It is currently not in use.

In hydropower sector, the first hydropower tunnel constructed was Tinau Hydroelectric Project situated at Dovan VDC-6 of Palpa District by his Majesty's Government and the United Mission to Nepal (UMN) in a joint initiative. 2400m tunnel was constructed with the diameter of 1.8 m and cross-sectional area 2.1 m2. The project commenced on 2022 B.S. (1966 A.D.), taking approximately 11 years for its completion. Similarly, other hydropower project that incorporated tunnel structure as the waterway is Kulekhani I hydropower project which was commissioned in 1982 A.D. with the length of headrace tunnel of 6233m followed by Kulekhani II HPP with the headrace tunnel length of 5847.8m. Adhikhola hydropower plant was commissioned on 1991 A.D. with the installed capacity of 5.1 MW. The headrace tunnel of the project is fully concrete lined having the length of 12847m with the cross-section area of 7.5 m2, drop shaft of 245m having the diameter of 4.5 m and at last long a tailrace tunnel with the 1080m having cross sectional area of 8.5 m^2.

Recently, the government has been motivated to adopt tunneling for transportation purposes as well. The following road tunnels are under various stages of planning and study(OnlineKhabar, 2019):

1. Kathmandu Terai Fastrack (Mahadev Danda, Dhedre and Len Danda)
2. Tokha-Chhahare-Gurjubhanjyang & Betrawati-Shyaprubesi (Ghattekhola)
3. Siddhababa
4. Thankot-Chitlang
5. Kulekhani-Bhimphedi
6. Koteshwor-Jadibuti
7. Godavari-Manechaur
8. Butwal-Narayanghat Daunne Ukalo
9. Kailali's Khutiya-BP Nagar-Dipayal
10. Hemja-Nayapul

11. Lama Bagar in Dolakha
12. Khurkot-Chiyabari

According to information obtained from the DOED website and from the annual report of NEA, the following hydropower projects are under various stages of study and obtaining licenses.

Table 2-2Hydropower projects under NEA

Project	Stage	Tunnel Length
Dudh Koshi HEP	Feasibilty Stage	13300 m
Upper Arun HEP	Engineering Design and EIA	
Upper Modi HEP	Feasibility conculed	
Andhikhola Storage HEP	Feasibility Stage	3112 m + 1227 m
Chainpur Seti HEP	Study ongoing	12492 m
Suligad ROR	Study ongoing	7400 m
Chera I Storage	Study ongoing	4250 m
Dadagaon Khalanga	Study ongoing	3500 m
Kaligandaki 2 storage	Study ongoing	
Raghuganga	Under construction	6270 mm

Table 2-3 Licensed projects in 2077 under DOED

Project	Length of Tunnel
Nyasin HEP	2362m
Budhi Gandaki	408 m + 438 m
Dudhkhola HEP	4336.61 m
Kunban Khola HEP	3400 m
Chino Khola	411 m
Landruk Modi	7273m + 3736 m
Sagu Khola	4275 m

The number of hydropower projects with tunnels currently under construction is 42 with a total tunnel length of 206 km. The number of projects currently under planning is 33 with a total tunnel length of 200.9 km. The data obtained from NTA report is tabulated as:

Table 2-4 Status of Hydropower Tunnels in Nepal (NTA, 2020)

Province	Length (km)		Number	
	Under	Planned	Under	Planned
1	71.06	47	14	7
2	0	0	0	0
Bagmati	80.1	29.9	15	3
Gandaki	31.359	77.167	10	15
5	12	0	1	0
Karnali	0	22.6	0	4
Sudurpaschim	12.18	24.2	2	4
Total	206.7	200.27	42	33

This information is presented in the chart below

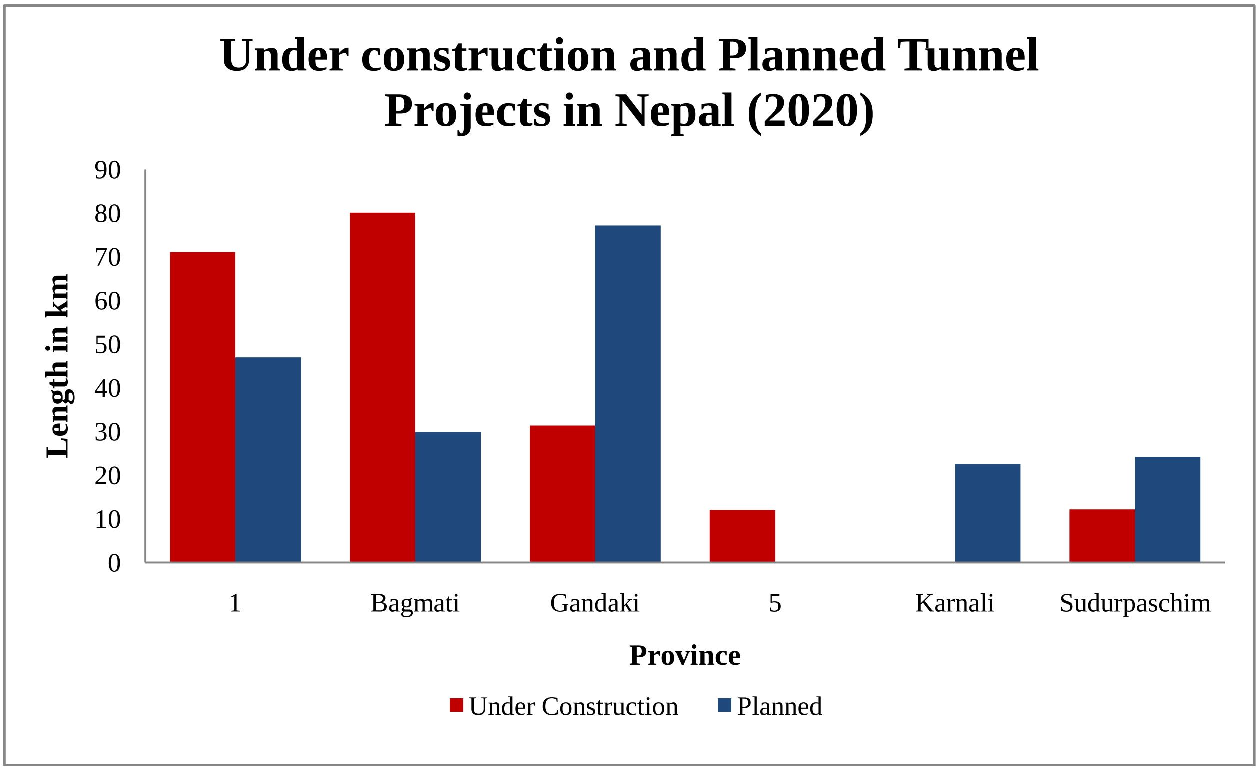


Figure 2-3 Status of Hydropower Tunnels in Nepal (NTA, 2020)

2.4 Tunnel Construction Practices in Nepal

2.4.1 New Austrian Tunnelling Method

With increase in the knowledge and the dynamics of the rock mass, new more economical yet safer support systems were searched for and introduced. New Austrian Tunneling Method (NATM) is the method that is continuously updating itself through the experience. It is notified by Singh et al.(2006) “NATM is a misnomer as it is not a method of tunneling but a strategy for tunneling which does have a considerable uniformity and sequence”. NATM defines its approach to be flexible with the philosophy of “Build as you go” where the optimum support condition is assessed making the support not too stiff or too flexible with carefully understanding the time factor being the important aspect of stability by installing the support neither too early nor too late(Karki et al., 2018).

The stabilization of underground opening depends upon the redistribution of the stress and controlling the stress release. These techniques will help to reduce the loosening effect of the rock mass around the boundary and which in-turn helps to minimize the hazards and risk. This is the specific strategy of NATM where this method helps surrounding to act as the self-supporting structure with the support installation like shotcrete and systematic rock bolts.
In rudimentary concept NATM defines its principle as follows:

- Taking Rock as engineering material by mobilization of its strength.
- Provision for initial shotcrete to localize the failure.

- Monitoring the deformation of the opening.
- Providing adequate support according to the requirement.
- Providing the invert lining to form a complete continuous load bearing support system.

In a whole NATM is the method derived from the experience and is a favorable method for weak ground where smooth profile of the opening can be obtained using either perimeter blasting or smooth blasting. The timing and the extent of secondary support is decided significantly by monitoring the performance of the underground construction. The main idea is to use the geological stress of surrounding rock mass to stabilize the tunnel itself. The first use of NATM in soft ground tunnel was done in 1969 AD

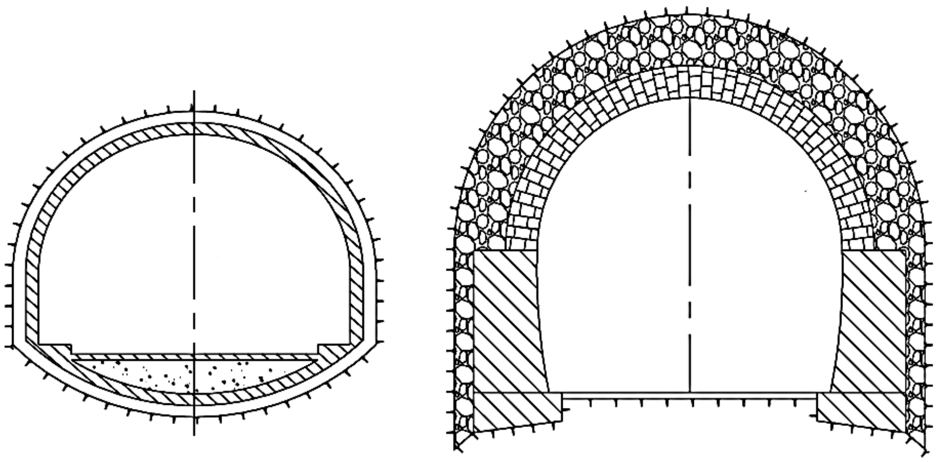


Figure 2-4 Showing the use of invert lining and comparative thickness of modern (Left) and Traditional (Right) (Karki et al., 2018)

2.4.2 Norwegian Method of Tunneling (NMT)

The NMT is a form of tunnelling system and process that outlines a complete set of techniques, for investigations, design, construction and rock support. It adopts a systematic approach to the different phases of tunnelling. The NMT follows the principles of the observational method which includes assessment of the variations in ground conditions, observations during construction and modification of design to suit the actual conditions. Norwegian tunnelling leverages on close cooperation between tunnellers, contractors, design engineers and engineering geologists and this is important for developments in tunnel design, excavation methods and rock supporting measures. The 'hard rock regime of NMT' is based on the self-standing capacity, impermeable nature and the stress-induced confinement of the tunnels. In general, the main features of Norwegian Method of Tunnelling, as outlined by Barton , encompass the following(Karki et al., 2018):

- Engineering geology report used as basis for cost estimates
- Unit prices for various rock conditions; client pays according to actual rock conditions;
- Preliminary design used for tendering
- Detailed design decided during excavation after tunnel mapping
- Close collaboration between geologists of contractor and client

- Forum for resolving differences on site
- Emergency power conferred to contractor in the event of adverse conditions

The utilization of high strength and highly ductile fibre-reinforced micro silica shotcrete is another speciality of NMT which removes the need for mesh reinforcement and has sufficient early strength to replace steel and cast concrete arches under a wide-range of tunneling conditions. The use of wet-mix shotcrete has been increasing over the years, both for temporary support and permanent support. The combination of rock bolting and shotcrete used for rock support is widespread. It should be further added that numerical modeling and monitoring during construction supplement the empirical design and should be considered a part of the NMT.

2.4.3 Tunnel Boring Machines

The TBM is usually fabricated to match the rock conditions. If the TBM is not powerful enough for the rock, the machine will be overstressed and be prone to breakdowns. Similar to an electric drill, the TBM operates by using thrust and torque. Like the drill, the motor rotates the bit (cutterhead), whereas the thrust for the hand electric drill is provided by the person operating the drill; the TBM receives its thrust by cylinders that push the cutterhead against the rock face. As with an electric drill, there is an optimum thrust and torque combination. The machine gets its forward thrust by pushing against the precast segmental lining

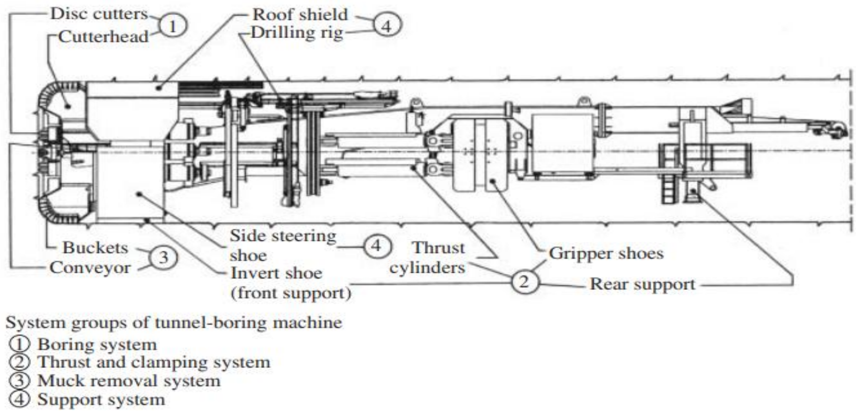


Figure 2-5 Schematic of a TBM

2.5 Estimation of Rock Supports

Study of rock mass is a significant aspect for the design and estimation of rock support in tunnel. When little detailed information is available on the rock mass of a particular area, its stress and hydrologic characteristics, the use of rock mass classification scheme can be of substantial benefit. This can involve the use of classification scheme as a check list to see that all relevant information has been put into consideration. One or more rock mass classification schemes can also be used to prepare a picture of the composition and characteristics of a rock mass. This can help in the initial estimation of support requirements and to prepare an estimation of the strength and deformation properties of the rock mass.

2.6 Geomechanics Classification or the Rock Mass Rating (RMR) system

The geomechanics classification or the rock mass rating (RMR) system was initially developed by Z. T. Bieniawski at the South African Council of Scientific and Industrial Research (CSIR). Bieniawski developed this model based on his experiences in shallow tunnels in sedimentary rocks. Over the years, this system has been successively refined with numerous case records examined. Bieniawski made notable changes in the ratings assigned to different parameters. RMR employs the following six parameters to classify a rock mass (Palmstrøm, n.d., 2014)

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

This classification requires dividing the rock mass into a number of structural regions. The regions then are classified separately. The boundaries of the structural regions though generally coincide with structural features as fault or with a change in rock type.

Table regarding guidelines for excavation and support of 10 m span rock tunnels in accordance with RMR system (after Bieniawski 1989) has been presented below in Table 2-5 and **Table 2-6**:

Table 2-5 Support system for RMR classification (Bieniawski, 1989)

RMR Range	Excavation	Rock Bolts	Shotcrete	Steel Set
81-100	Full face 3m advance	Generally no support except spot bolting		
61-80	Full face 1-1.5m advance. Complete support 20 m from face	Locally, bolts in crown, 3 m long spaced 2.5 m with occasional wire mesh	50 mm in crown where required	None
41-60	Top heading and bench, 1.5-3m advance in top heading. Commence support after each blast. Complete support 10m from face	Systematic bolts 4-5 m long spaced 1-1.5 m in crown and walls with wire mesh	100-150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required
21-40	Top heading and bench, 1.0-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting	Systematic bolts 4-5 m long spaced 1-1.5 m in crown with walls with wire mesh	100-150 mm in crown and 100 mm in sides	light to medium ribs spaced 1.5m where required
<20	Multiple drifts, 0.5-1.5m advance in top	Systematic bolts 5-6 m long spaced 1-	150-200 mm in crown, 150 mm in	Medium to heavy ribs

	heading Install support concurrently with excavation Shotcrete as soon as possible after blasting	1.5m in crown and wall with wire mesh. Bolt invert	walls and 50mm on face	spaced 0.75m with steel lagging and forepoling when required. Close invert

Table 2-6 RMR Classification for Rock Mass (Bieniawski, 1989)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
		Rating	15	12	7	4	2	1	0
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/(Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

Discontinuity length (persistence) Rating	< 1 m 6	1 - 3 m 4	3 - 10 m 2	10 - 20 m 1	> 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1 - 1.0 mm 4	1 - 5 mm 1	> 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard filling < 5 mm 4	Hard filling > 5 mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0
Weathering Ratings	Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
Fair	Unfavourable	Fair	

* Some conditions are mutually exclusive . For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.
** Modified after Wickham et al (1972).

2.7 Rock Tunnel Quality- Q system

The Rock Tunnel Quality- Q system was developed by Barton, Lien and Lunde. This system expresses the quality of rock mass in the Q- value. This system is basically recommended for underground excavations.

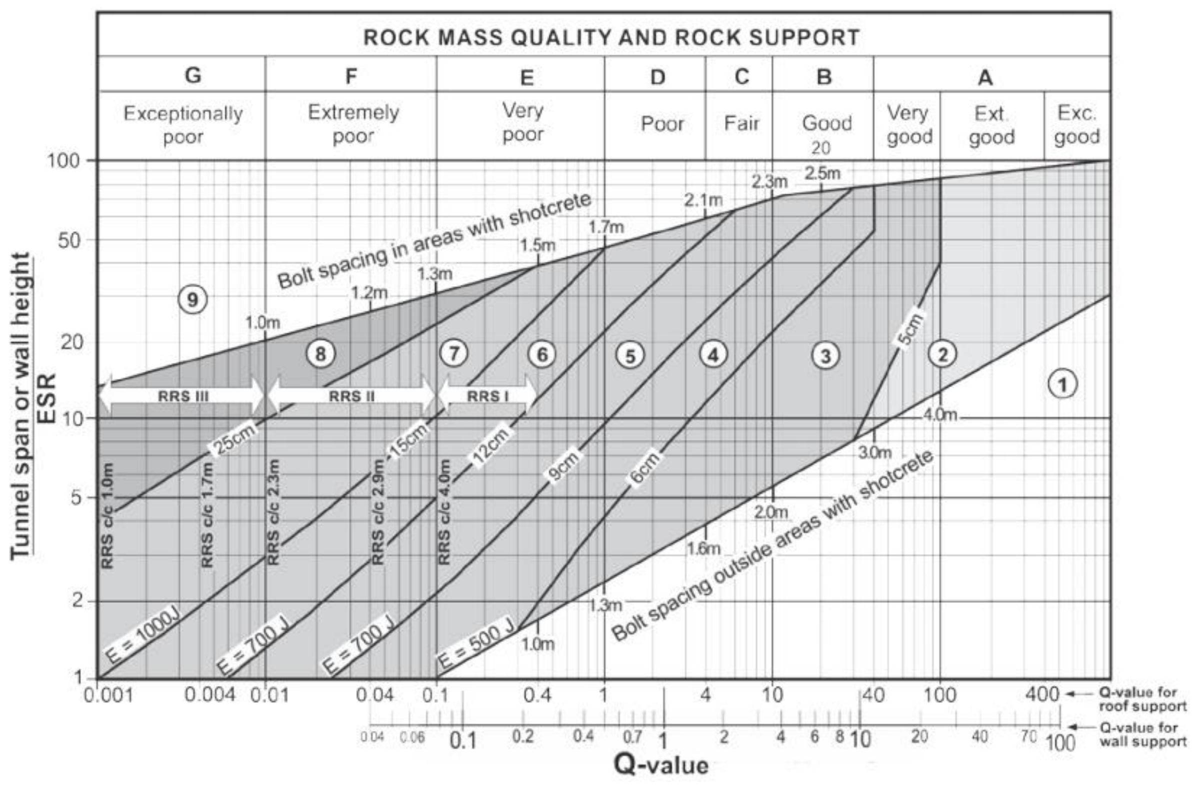
The Q-value is determined by: $Q= (RQD/J_n) * (J_r/J_a) * (J_w/SRF)$

Where,
RQD= Rock Quality Designation
J_n = Joint Set Number
J_r = Joint roughness number
J_a = Joint alteration number
J_w = Joint water parameter
SRF = Stress Reduction Factor

Table 2-7 Values for ESR

Type of Underground Openin	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 tunnels with little traffic, access tunnels	1.3
Power Stations, road and railway with heavy traffic, civil defence shelters	1.0
Nuclear Power plants, railroad stations, sports arenas etc	0.8

Figure 2-6 Support System according to Q



The Q-value determines the quality of the rock mass, but the support of an underground excavation is based not only on the *Q-value* but is also determined by the different terms in the above equation. This leads to a very extensive list of classes for support recommendations.

Limitations:

- It is difficult to obtain the Stress Reduction Factor SRF in the Q-system and any of its value covers a wide range of in-situ stress for rocks of certain strength. As the importance of in situ stress on the stability of underground excavation is insufficiently represented in the Q-system, hence it cannot be used effectively in rock engineering design.
- Q-system is not suitable for soft rocks.
- Use of open logarithmic scale of Q varying from 0.001 to 1000 as compared to the linear scale of up to 100 induces difficulty in using the Q-system.

2.8 Rock Supports

<u>Rock Bolts</u>

Rockbolt is the most widely used support element in support systems in underground mines and civil tunnels. Rock Bolt Design is indeed mainly based on experience and it appears that rock bolting design is simply a business of selection of rock bolt types and the determination of bolt length and spacing, but, one essentially uses, either explicitly or implicitly, a methodology in specific rock bolting design.

Rock Bolts generally consist of plain steel rods with a mechanical or chemical anchor at one end and a face plate and nut at the other. They are always tensioned after installation. For short term applications the bolts are generally left ungrouted. For more permanent applications or in rock in which corrosive groundwater is present, the space between the bolt and the rock can be filled with cement or resin grout. (Hoek, 2006)

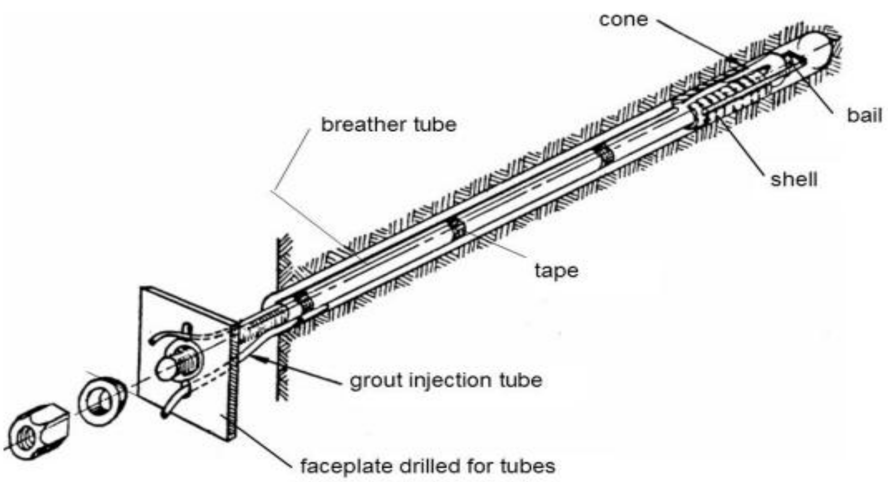


Figure 2-7 Rockbolt (Hoek, 2006)

Shotcrete

In recent years the mining industry has become a major user of shotcrete for underground support. It can be expected to make its own contributions to this field as it has in other areas of underground support. The simultaneous working of multiple headings, difficulty of access and unusual loading conditions are some of the problems which are peculiar to underground mining and which require new and innovative applications of shotcrete technology.

Shotcrete is the generic name for cement, sand and fine aggregate concretes which are applied pneumatically and compacted dynamically under high velocity. (Hoek, 2006)

Dry-Mix Shotcrete:

The dry mix process involves the mixing of cement and wet aggregates at required proportion before supplying it to the shotcreting device. The thoroughly mixed ingredients are then placed on the device hopper. During the shotcreting operation, the mix, under the action of compressed air is taken from the hopper to the nozzle through the delivery hose of the equipment.

Once the dry mix reaches the nozzle, water under high pressure is sprayed to the mix through a perforated ring attached to the equipment. While spraying, the water wets the dry mix. Thus, the required wet concrete or mortar mix is jetted at a higher velocity on the surface.dry mix shotcrete is applied in areas where there requires fewer placements and no or limited vehicle access.

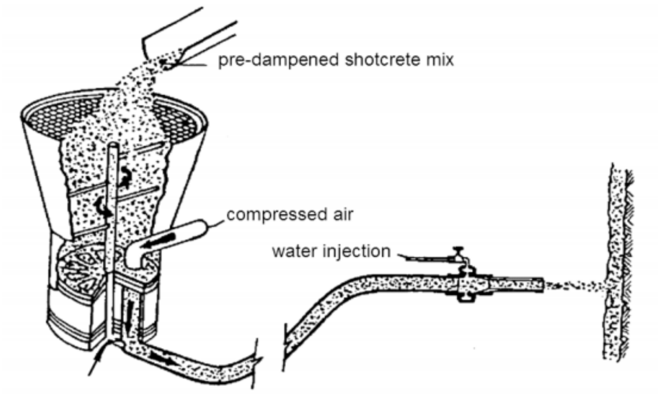


Figure 2-8 Dry Mix Shotcrete**(Hoek, 2006)**

Wet Mix Shotcrete:

The wet mix process involves the mixing of all ingredients to form mortar or concrete with required water content. Here, the mix to be shotcrete is prepared before placing it in the shotcreting equipment. The delivery equipment used can be a positive displacement type or a pneumatic -feed. The process involves forcing the wet mix to the nozzle through the delivery hose by means of compressed air. The mix is then shot at high velocity on the surface to be shotcreted.

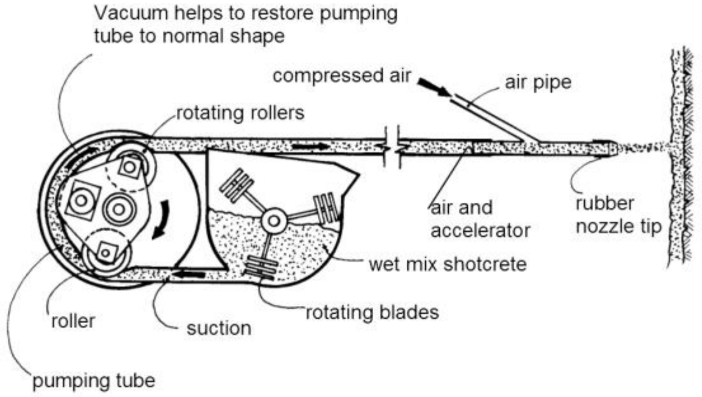


Figure 2-9 Typical wet shotcrete machine**(Hoek, 2006)**

Steel Sets

Steel sets can only respond to loads imposed on them by the inward movement of the rock, hence they are referred to as passive support. Since they are generally placed some distance behind the advancing face, most of the short-term movement in the rock has already taken place before the sets are in place and the only load that they are called upon to carry is the dead weight of rock failing around the opening. In hard rock mining, steel sets have very limited application since most support duties can be performed more effectively by rock bolts, or shotcrete or by some combination of these systems. The exception is in mining through faults or in very badly broken ground associated with faults of shear zones. In such cases, it may be impossible to anchor the rock bolts or dowels in the rock mass and steel sets may be required in order to carry the dead weight of the failed material surrounding the opening.(Karki et al., 2018)

Figure 2-10 Use of Steel sets in weak, disintegrated rock at HRT of Super Madi

Forepoling

Fore poling has been frequently used to stabilize the ground around the cutting face, and to control the settlement of ground surface. This method involves the driving of pipes and the injecting of grouting materials into the ground ahead of face prior to excavation. Following instruments are used for fore poling(Karki et al., 2018):

- Pneumatic Rotary Drilling machine
- Perforated Pipe
- Jack Hammer
- Grouting machine

If the strength of the ground is so low that the excavated space is unstable even for a short time, a pre-driven support is applied in such a way that an excavation increment occurs under the protection of a previously driven canopy. Forepoling is achieved by spiling, pipe roof, grouting and freezing.

Spiling consists of drilling a canopy of spiles, i.e. steel rods or pipes into the face. A typical length is 4m. In order for the spiles to act not only as beams (i.e. in longitudinal direction) but also to form a protective arch over the excavated space, the surrounding soil is grouted through the steel pipes or sealed with shotcrete. Thus, a connected canopy is formed that consists of grouted soil reinforced with spiles. Spile rods can also be placed into the drill hole. The remaining annular gap is filled with mortar, whose setting however may prove to be too slow.

As with the case with steel sets, spiling rods were only used where the solid was very loose and unstable like in the headrace tunnel's lower part.

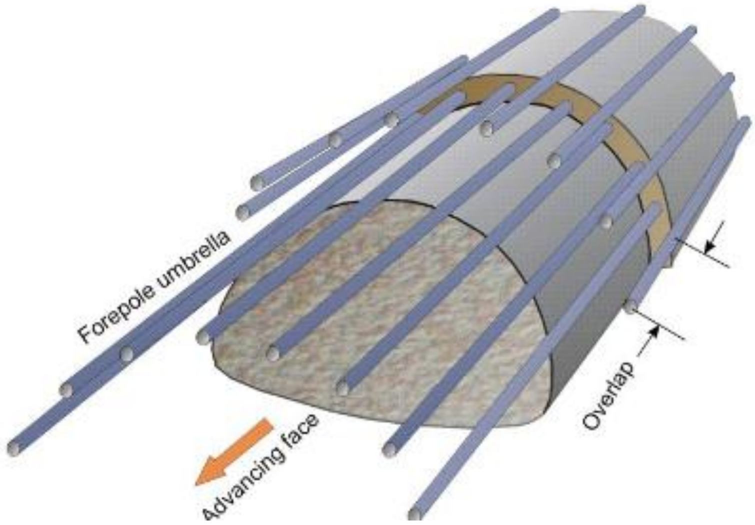


Figure 2-11 Forepoling **(Basirat, Hassani, & Mahmoodian, 2016)**

2.9 Squeezing Assessments

The squeezing of rock is a process of large deformation which occurs around the tunnel due to stress concentration and material properties, and it is a major factor for predicting rock behavior in underground excavations (Ghiasi et al., 2012) .According to the phenomenological definition of squeezing rock adopted by the Commission on Squeezing Rocks in Tunnels, ISRM, "Squeezing of rock is the time-dependent large deformation, which occurs around the tunnel, and is essentially associated with creep caused by exceeding a limiting shear stress. Deformation may terminate during construction or continue over a long time period."

When an underground opening is excavated, the existing stress regime is disturbed. As the stress cannot pass through the opening, it redistributes itself around the opening. This causes the concentration of stress along the contour of the opening(Shrestha, 2005). It is illustrated in the figure below:

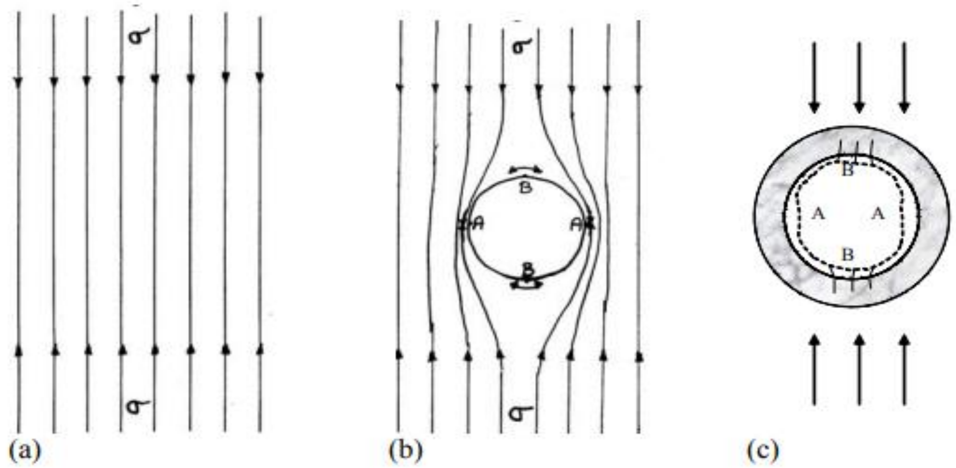


Figure 2-12 Vertical stress distribution (a) before excavation (b) after excavation and (c) A is squeezing failure location and B is tensile failure location indicated by the dotted lines. (Shrestha, 2005)

2.9.1 Empirical Methods of Squeezing Assessment

2.9.1.1 Singh et. Al. 1992 approach

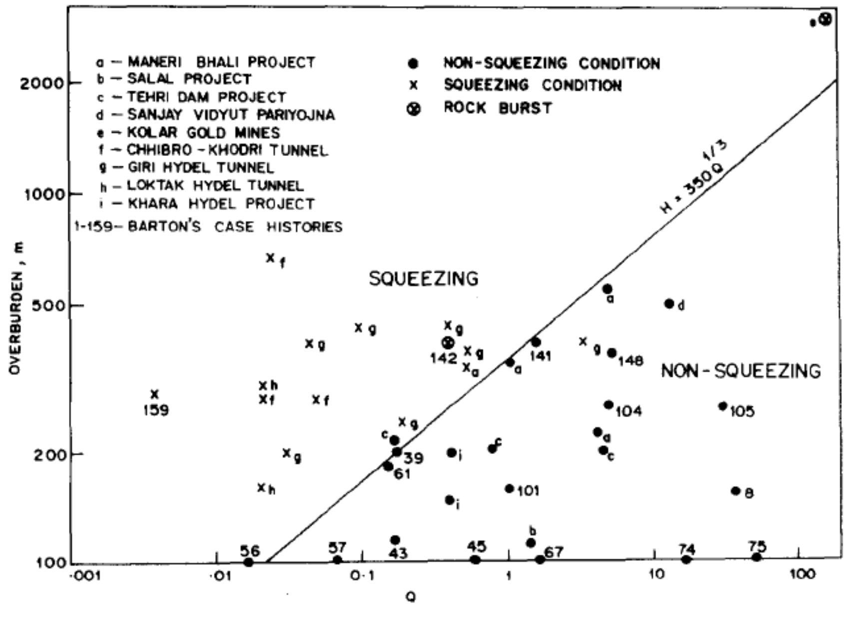


Figure 2-13 Criteria for predicting squeezing ground condition(Singh et al., 1992).

A clear line of demarcation between the elastic and the squeezing conditions can be seen. The equation for this line has been obtained as

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

Thus, a rock mess may undergo squeezing when the depth of the tunnel section exceeds 350 $Q^{1/3}$. (Singh et al., 1992)

The rock mass uniaxial compressive strength **σ**cm estimated as

σcm = 0.7 γ $Q^{1/3}$ [MPa] with:

γ = rock mass unit weight. (Giovanni Barla, n.d.)

2.9.1.2 Goel et. Al(1995) approach

The squeezing ground criteria given by Goel et al. (1995) are based on rock mass number (N) and the size of the opening. Rock mass number (N) is obtained by making the stress reduction factor (SRF) = 1 in Q classification (Karki et al., 2018).

The equation of that line which demarcates the squeezing is $H = 275N^{0.33}B^{-1}$, where N is rock mass number, B is width of tunnel in m and H is overburden depth in m. The data points lying above the line represents squeezing conditions, whereas those below this line represent non-squeezing condition.(Khadka, 2019)

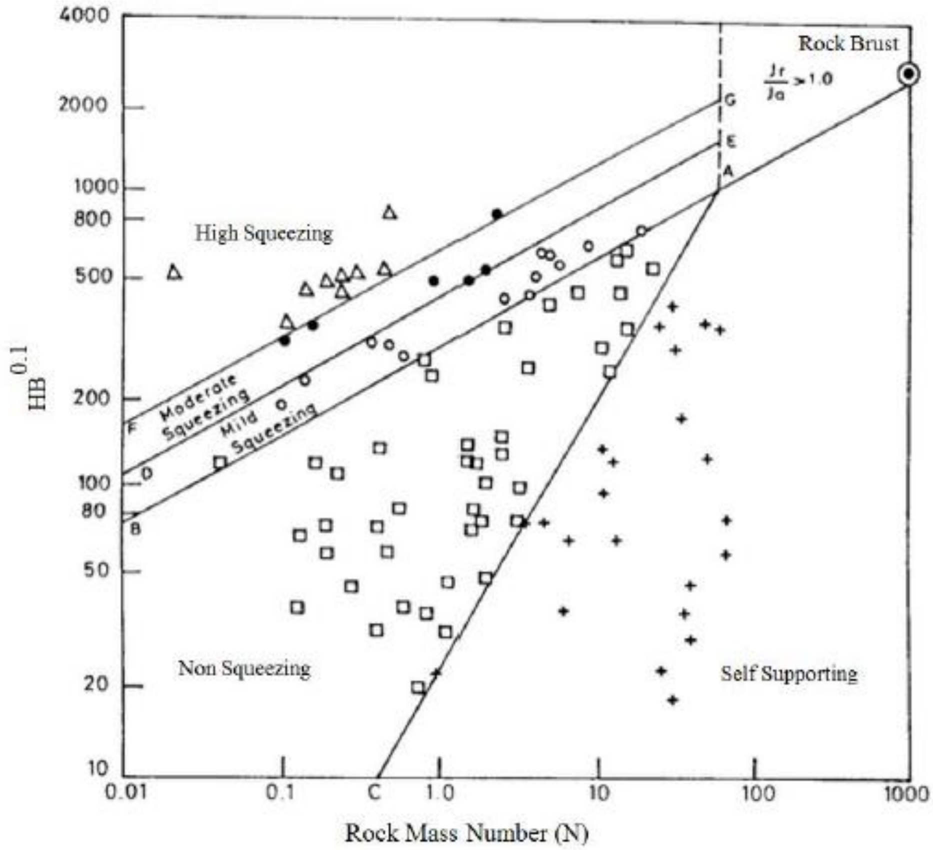


Figure 2-14 Prediction of Squeezing according to Goel et. al. (1995)

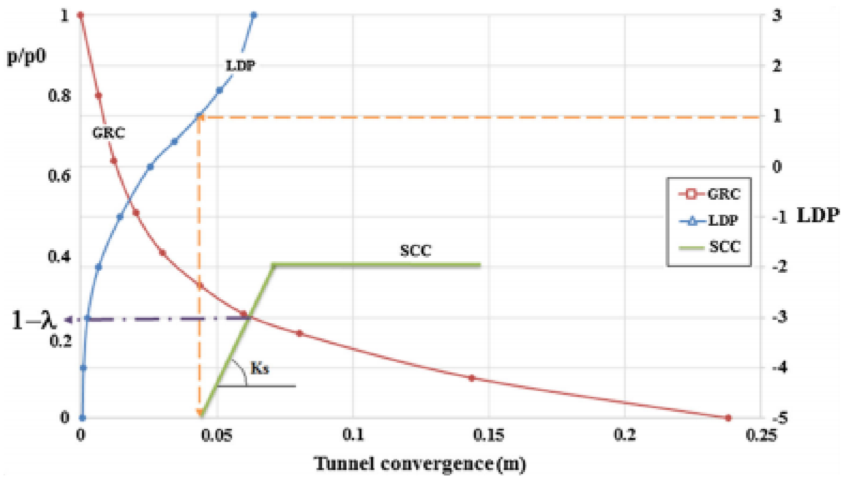
2.10 Analytical Method (Convergence Confinement Method)

The convergence-confinement method is part of the rational approach and uses an analytical type calculation. It is based on the analysis of the stress and strain state that develops in the rock around a tunnel.

The convergence–confinement method represents an efficient way for the analysis and design of tunnel lining. CCM has a dimensionless coefficient λ, which represents stress relaxation in the tunnel walls at different excavation steps . This parameter is considered as a constant number in previous studies and effects of various factors such as ground materials, depth, radius and cross-section of tunnel are not taken into account.

As determination of the load transferred to the support requires an analysis of the interaction of the load-deformation characteristics of the elements comprising the system, (i) the tunnel as it moves forward; (ii) the section of excavation perpendicular to the tunnel axis; and (iii) the support installed at that section. The three basic components of the Convergence-Confinement method are, therefore:

1. Ground reaction curve (GRC).
2. Longitudinal displacement profile (LDP)
3. Support characteristic curve (SCC).



a) Longitudinal Displacement Profile (LDP)

The LDP is the graphical representation of the radial displacement that occurs along the axis of an unsupported cylindrical excavation for sections located ahead and behind of face. The horizontal axis indicates the distance x from the section analyzed to the tunnel face; the vertical axis indicates the corresponding radial displacement ur. The diagram indicates that at some distance behind the tunnel face the effect other face is negligibly small, so that beyond this distance the unlined tunnel section has converged by the final amount uM. Similarly, at some distance ahead of the face, the advancing tunnel has no influence on the rock-mass and the radial displacement is zero.

b) **Ground Reaction Curve (GRC)**

Considering now the section of unlined tunnel represented in Figure, the GRC is defined as the relationship between the decreasing internal pressure p and the increasing radial displacement of the wall Ur. The relationship depends on the mechanical properties of the rock mass and can be obtained from elasto-plastic solutions of rock deformation around and The GRC is shown as the curve OEM in the lower diagram of Figure extending from point O where the internal pressure p is equal to the initial stress to point M where the internal pressure is equal to zero (i.e., the tunnel is unsupported) and the tunnel closure (i.e. the radial displacement) u_M is maximum. Point E defines the internal pressure p and corresponding closure at which the elastic limit of the rock is reached (at the tunnel wall). If the internal pressure falls below this value, a failed region of extent R_{pl} develops around the tunnel.

c) **Support Characteristic Curve (SCC)**

The SCC is similarly defined as the relationship between the increasing pressure on the support and the increasing radial displacement of the support. This relationship depends on the geometrical and mechanical characteristics of the support. The SCC is shown as the curve KR in the lower diagram of Figure. Point K corresponds to a support pressure equal to zero (i.e. when the support is first installed) and point R to the pressure that produces failure of the support.

Inspection of the LDP, GRC and SCC in Figure leads to two conclusions of practical interest:

- The support will not be subject to a radial pressure larger than p_L as defined by point L in the lower diagram. This pressure would be achieved only in the hypothetical case of an infinitely rigid support installed at the face itself i.e. the SCC would be a vertical starting from point H.
- A support will take no load if placed beyond point M, since the maximum possible convergence has occurred already. These two cases correspond to the two limiting cases of load that the rock mass can transmit to the support. In general, as is seen from the LDP, GRC and SCC in Figure, the further that the support is installed from the tunnel face, the lower the final load pD on the support.

These two cases correspond to the two limiting cases of load that the rock mass can transmit to the support. In general as it is seen from the LDP, GRC and SCC, the further that the support is installed from the tunnel face, the lower the final load p^D on the support assuming that no time-dependent weakening or disintegration of the rock mass occurs.

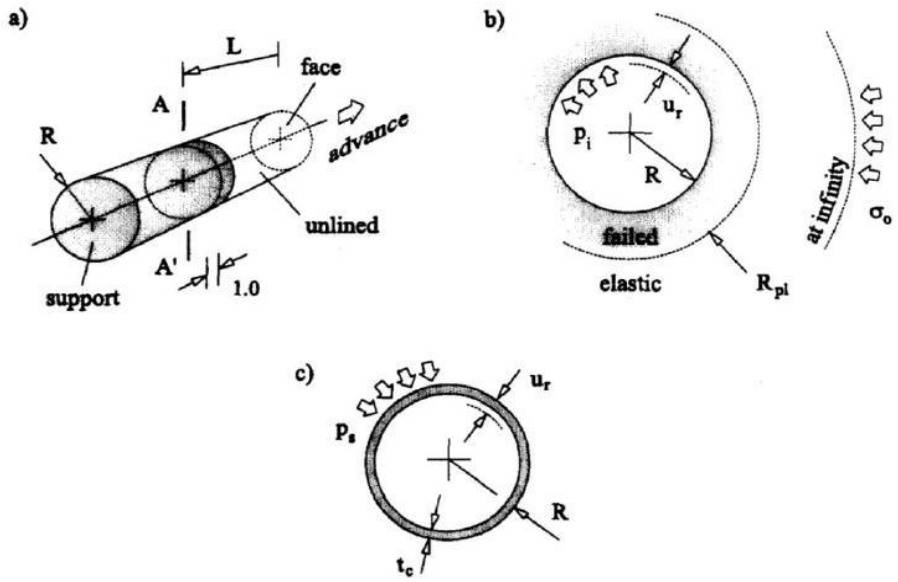


Figure 2-15 Cylindrical tunnel of radius r driven in rock mass, b) Cross-section of rockmass at section A-A ' c) Cross-section of circular support at section A-A'

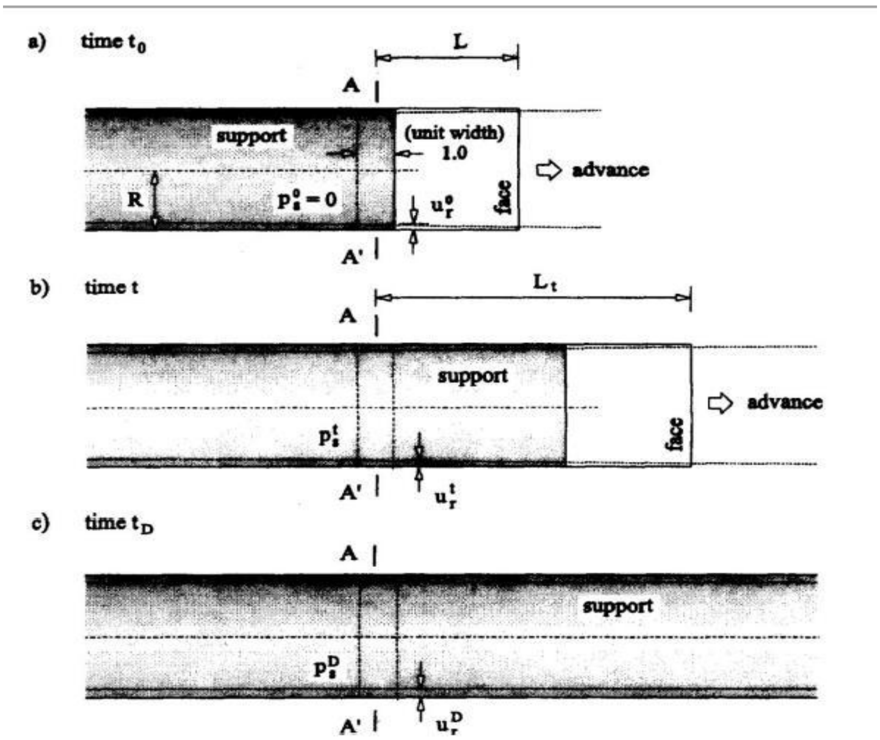


Figure 2-16 Loading of the support at section A-A' due to progressive advance of tunnel face(Carranza-Torres & Fairhurst, 2000)

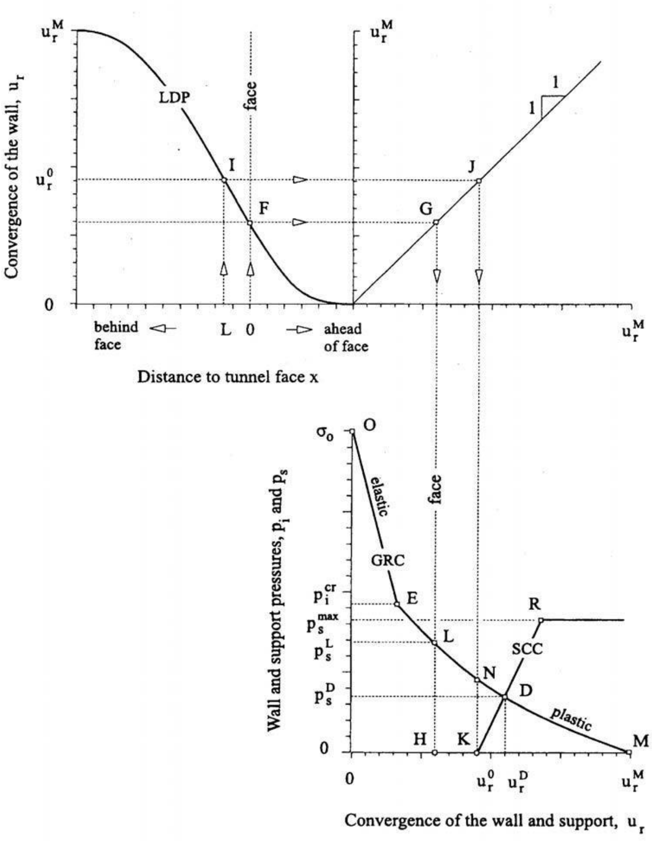


Figure 2-17 Schematic representation of LDP, GRC and SCC(Carranza-Torres & Fairhurst, 2000)

2.10.1 Limitation of CCM

The CCM is based on two assumptions; first, the state of stress is often referred to as uniform or hydrostatic with constant magnitude and second, the tunnel cross section is circular (Carranza-Torres and Fairhurst, 2000). But in most of the cases, the far field stresses are unequal and tunnel cross section is non-circular. In these cases too, CCM can be used with some special assumptions that are described further in this section. The measured values of vertical stresses σ_z as a function of overburden depth z for different regions of the world can be expressed by best fit relationship shown in Figure:

$$\sigma_z = 0.027z$$

Where, σ_z is expressed in MPa and z in meters.
In this relationship, if the stress is assumed to be associated with the weight of overburden material, the factor 0.027 ought to be the density of rock mass in MN/m3. This value corresponds to the unit weight of silicates, a major components of many rocks (Carranza-Torres and Fairhurst, 2000).The value of k can be defined as:

$$k = \sigma x \;/\; \sigma z$$

The value of k varies from minimum of 0.5 to maximum of 3.5. This condition suggests that the principal stresses at the site are often unequal. In such cases, the average of these stress can be taken as input stress in CCM:

$$\sigma_0 = (\sigma_x + \sigma_z)/2$$

The uniform state of stress assumed by the CCM can be expressed as $\sigma_o = \sigma_x = \sigma_z$ and k = 1.

The result obtained from CCM in case of non-uniform stress field can be verified with respect to the term limiting stress ratio, k_{lim}. If normal stress ratio, k is less than k_{lim}, the mean radius of plastic region around tunnel and the mean convergence at the crown and sidewall of the tunnel are same as the corresponding values obtained from CCM using the relationship in equation 5.9. If k >k_{lim}, there is no apparent relationship to the case of uniform loading (Carranza-Torres and Fairhurst, 2000).

2.11 The Hoek-Brown Failure Criteria

The Hoek–Brown failure criterion is an empirically derived relationship used to describe a non-linear increase in peak strength of isotropic rock with increasing confining stress. (Eberhardt, 2012)

The original Hoek-Brown failure criterion in terms of principle stress relationship is defined by the following equation (Hoek and Brown, 1980);

$$\sigma 1' = \sigma 3' + \sigma ci * \left(m\frac{\sigma 3'}{\sigma ci} + s\right)^{0.5}$$

Where, σ_1\` and $\sigma_3$\`are the major and minor effective principle stresses at failure, σ_{ci} is the uniaxial compressive strength of intact rock material and m and s are material constants, where s=1 for intact rock.

This criterion has been updated several times over the years to fit more relevant parameters. An important landmark in the field of rock mechanics has to be the formulation of the Generalized Hoek-Brown criteria in 1995.It incorporated both the original criterion for fair to very poor quality rock masses and the modified criterion for very poor quality rock masses with increasing fines content(Evert Hoek, 2002). The generalized Hoek-Brown Criteria is defined as (E Hoek et al., 1995):

$$\sigma 1' = \sigma 3' + \sigma c\left(m_b\frac{\sigma 3'}{\sigma c} + s\right)^{a}$$

$\sigma 1'$= major principal effective stress at failure, $\sigma 3'$ = minor principal effective stress at failure, σc = uniaxial compressive strength of intact pieces of rock, mb, s and a are constants which depend on the composition, structure and surface conditions of the rock mass

Where m_b is a reduced value of the material constant mi and is given by,

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

The basis of values for the material constant mi and Geological Strength Index, GSI, are given inTable 2-9 and Table 2-10 respectively. sand a are constants for the rock mass given by the following relationships;

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

$$a = 0.5 + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-GSI-20}{3}}\right)$$

D is the factor which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from zero for undisturbed in situ rock masses to 1 for very disturbed rock masses.

The Hoek-Brown failure criterion is only applicable to intact rock or to heavily jointed rock masses which can be considered homogeneous and isotropic. (Hoek et al., 1995)

Table 2-8 Values of D for different conditions(Evert Hoek, 2006)

Appearance of rock mass	Description of rock mass	Suggested value of D
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	D = 0
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	D = 0 D = 0.5 No invert

	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	D = 0.8
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	D = 0.7 Good blasting D = 1.0 Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	D = 1.0 Production blasting D = 0.7 Mechanical excavation

Table 2-9 Values of mi for different types of rocks(Evert Hoek, 2006)

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerates* (21 ± 3) Breccias (19 ± 5)	Sandstones 17 ± 4	Siltstones 7 ± 2 Greywackes (18 ± 3)	Claystones 4 ± 2 Shales (6 ± 2) Marls (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
		Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4) Metasandstone (19 ± 3)	Quartzites 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolites 26 ± 6		
	Foliated**		Gneiss 28 ± 5	Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4

IGNEOUS	Plutonic	Light	Granite 32 ± 3 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 ± 3) Basalt (25 ± 5)	Obsidian (19 ± 3)
		Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

* Conglomerates and breccias may present a wide range of m_i values depending on the nature of the cementing material and the degree of cementation, so they may range from values similar to sandstone to values used for fine grained sediments.
* *These values are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if failure occurs along a weakness plane.

2.12 Post Peak Behaviour of rock mass

To obtain reliable results from modeling of rock mass and support, it is essential to be acquainted with the post peak behavior of the rock mass. Cai, et al. (2007), suggested that understanding of the rock mass strength behavior, including the peak and residual strength, will facilitate the cost-effective design of tunnel support.

Residual parameters, and therefore their incorporation in the model are thus very important.

In general, rock masses, except when highly disturbed, exhibit strain-softening post-peak behavior, so that the residual strength parameters are lower than the peak parameters. Both are required for design. Strain-softening behavior describes the gradual loss of load-bearing capacity of a material(Cai et al., 2007)

The 3 typical behaviours are shown in the figures below:

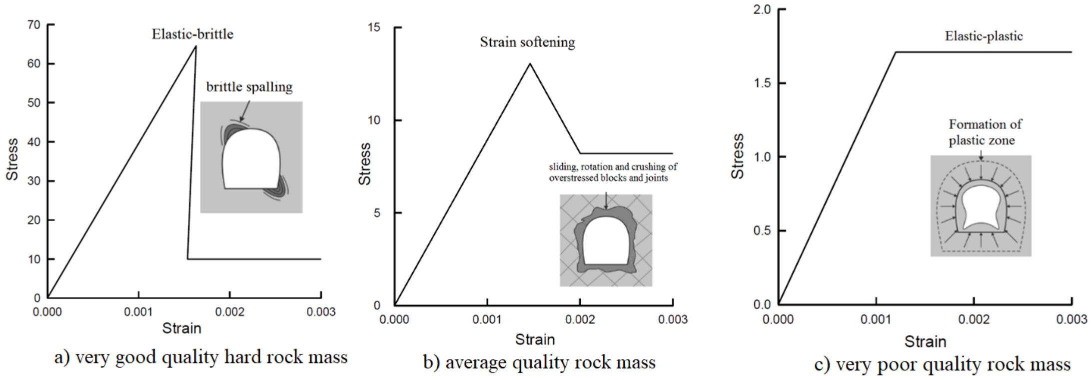


Figure 2-18 Suggested Post Failure Characteristic according to (Hoek, 2006)

The characteristics for varying rock masses are as follows:

- Elastic-brittle – good quality hard rock mass
- Strain-softening - average quality rock mass
- Elastic-plastic – very poor quality rock mass

These conditions are modeled using the residual GSI parameter.

In 1998 Russo et al. proposed to set the residual GSI value at 36% of the peak GSI value. This empirical relation may underestimate the residual GSI values for poor quality rock masses on the other hand, for very good quality rock masses; it may overestimate the residual GSI values(Cai et al., 2007).

Cai et al., 2007 provided an empirical relation to estimate the residual GSI from the laboratory and field data. The peak GSI value is reduced based on the reduction of the two major controlling factors in the GSI system, i.e., residual block volume V_{rb} and residual joint condition factor J_{rc}, to obtain the residual GSI_r value. The other parameter m_i and sig_c remain unchanged. The formula for residual GSI is $GSI_r = GSI \times e^{-0.134 \times GSI}$

Similarly, Khadka, 2019 suggested reducing the peak GSI as follows for rocks in the Himalayan region:

- No reduction for extremely weak rocks with GSI values less than 30 i.e elastic-plastic model
- Reducing the peak GSI between 60 and 70% for very poor to poor rock (30<GSI<50)
- Reduced between 40 and 50% for fair and good rocks (50<GSI<65)

2.13 Estimation of Parameters for 2D Numerical modeling

Numerical models are very sensitive to slight variations in input parameters. To obtain results that correspond well to observations in the real world, the parameters we use must be accurate. Ideally, the parameters that we need should be obtained by tests in the laboratory and field observations. However, it is often not possible to get first hand data in such manner due to logistical constraints and many other factors. Thus researchers in the years gone by have bestowed upon us many findings and results which aid us in estimating the parameters we need. The parameters used in modeling are listed below with their descriptions.

2.13.1 Geological Strength Index (GSI)

Since 1998, Evert Hoek and Paul Marinos, dealing with incredibly difficult materials encountered in tunneling in Greece, developed the GSI system to its present form to include poor-quality rock masses (Marinos et al., 2007). Initially, the GSI system was only for good-hard rock but was eventually extended to include poor rocks as well. GSI alone is not a tunnel design tool; its only function is the estimation of rock mass properties.

Classically, GSI needed to be estimated using the chart provided by Hoek (figure below). However, there are many situations where engineering staff rather than geological staff are assigned to collect data, which means that the mapping of rock masses or core is carried out by persons who are less comfortable with these qualitative descriptions(E. Hoek et al., 2013). Thus there is a need of a way to "quantify" GSI using equations.

Hoek et al., 2013 has provided the following relation between Jr, Ja and RQD to calculate the GSI.

$$GSI = \frac{52 \times \frac{Jr}{Ja}}{1 + \frac{Jr}{Ja}} + \frac{RQD}{2}$$

We also have a relation between GSI and RMR as per (E. Hoek & Diederichs, 2006) given as:

$$GSI = RMR - 5$$

However, since most projects utilize the Q value, it is often necessary to obtain RMR from the Q values. There are numerous relations given by researchers between RMR and Q. Goel et al., 1996 compared the numerous relations and found out that the one given by Rutledge and Preston had the highest correlation for himalayan rocks. the relationship is as follows :

$$RMR = 5.9 \, In \, Q + 43$$

Also, according to (Sayeed & Khanna, 2015), for the rocks of lesser Himalayas, the following relation has high correlation(r =0.86) ;

$$RMR = 4.52 \, lnQ + 43.6$$

Table 2-10shows the GSI values for the rock mass

2.13.2 Uniaxial Compressive Strength (UCS)

The UCS of intact rock is a mechanical property of rock. It is a parameter that goes into estimating the modulus of deformation of rock mass. The direct method of obtaining this value is from unconfined compressive test of the intact rock core. Apart from this there are other indirect measures of the UCS. Schmidt hammer rebound number is a common method of estimating this property. Schmidt hammer values are, however, influenced by the material to a fairly large depth behind the surface. (Hack & Huisman, n.d.).

Field estimates of UCS are tabulated in Table 2-11

Table 2-10 GSI values for Rocks

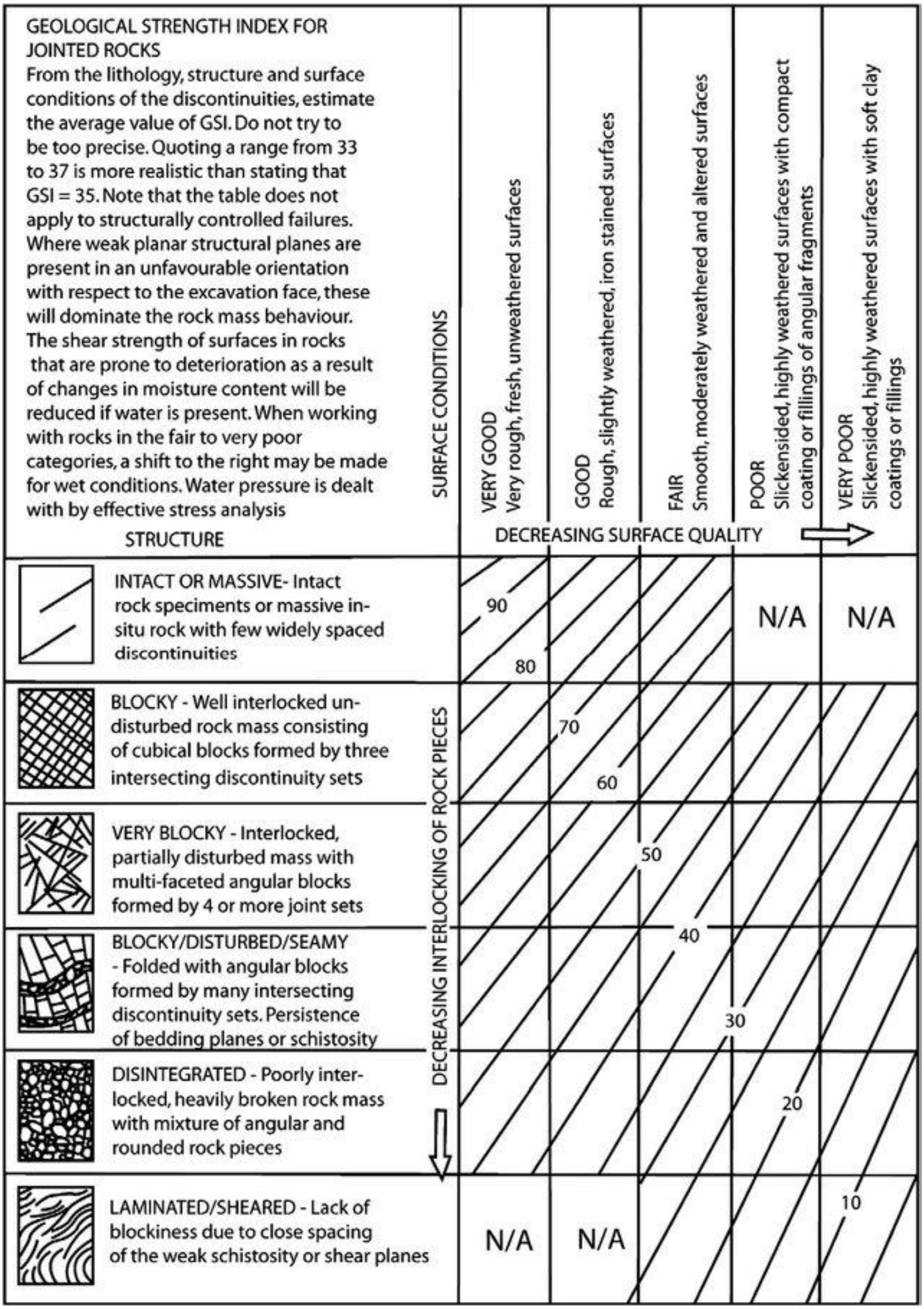


Table 2-11 Field estimation of intact UCS (“Rock Mass Properties,” n.d.)

Term	UCS (MPa)	Point Load Index (MPa)	Field Estimate
Extremely Strong	>250	>10	Specimen can only be chipped with geological Hammer
Very Strong	100-250	4-10	Specimen requires many blows of a geological hammer to fracture
Strong	50-100	2-4	Specimen requires more than one blow of a geological hammer to fracture it
Medium Strong	25-50	1-2	Cannot be scraped with a pocket knife, specimen can be fractured with single blow from a geological hammer
Weak	5-25	-	Can be peeled with a pocket knife with difficulty, shallow indentation made by a firm blow with point of a geological hammer
Very weak	1-5	-	Crumbles under firm blows with point of a geological hammer
Extremely weak	0.25-1	-	Indented by thumbnail

Point load tests for UCS < 25MPa yield ambiguous results

2.13.3 Poisson’s Ratio

Poisson’s ratio (υ) is defined as the negative of the ratio of transverse strain to axial strain when an isotropic material is subjected to uniaxial stress(Ji et al., 2018). It was found, that the value of Poisson’s ratio for the rock mass was about 20% higher than the value for the intact rock.(Vásárhelyi, 2009)

Knowing the Poisson’s ratio of the intact rock (*vi*) and GSI, the Poisson’s ratio for rock mass can be calculated as (Vásárhelyi, 2009):

$$\nu_{rm} = -0.002GSI + \nu_i + 0.2$$

If ν_i is not measured then the following formula may be used,

$$\nu_{rm} = -0.002GSI - 0.003m_i + 0.457$$

Aydan et al.2018 have also suggested a formula to obtain the Poisson’s ratio from RMR value.

$$\nu_{rm} = 0.5 - 0.2 \times \frac{RMR}{RMR + 0.2 \times (100 - RMR)}$$

2.13.4 Rock Mass Deformation Modulus

The deformation modulus of a rock mass is an important input parameter in any analysis of rock mass behaviour that includes deformations. The most common in situ test for the determination of the deformation modulus of a rock mass is the plate loading test or jacking test. Field tests to determine this parameter directly are time consuming, expensive and the reliability of the results of these tests is sometimes questionable (E. Hoek & Diederichs, 2006). Owing to the importance of this parameter and the difficulty in obtaining it, many authors have derived empirical relation to estimate the deformation modulus.

The following are the relations based on GSI:

Evert Hoek et al., 2002 gave the following relations:

$$E_{rm} = (1 - \frac{D}{2}) \times \sqrt{\frac{\sigma_{ci}}{100}} \times 10^{\frac{GSI - 10}{40}} \text{ for } \sigma_{ci} < 100\ MPa$$

$$E_{rm} = (1 - \frac{D}{2}) \times 10^{\frac{GSI - 10}{40}} \text{ for } \sigma_{ci} > 100\ MPa$$

The more recent relation in use is given by Hoek & Diederichs, 2006. This relation was based on data obtained from in-situ measurements from China and Taiwan. There are two forms of this equation, the simplified and general forms. The simplified relation doesn't include the intact rock modulus and should be used if reliable measure of σ_{ci} isn't available. The general relation utilizes the intact rock modulus:

Simplified Relation:

$$E_{rm}(MPa) = 100000 \times \frac{1 - D/2}{1 + e^{(75+25D-GSI)/11}}$$

Generalized relation:

$$E_{rm}(MPa) = E_i \times (0.02 + \frac{1-D/2}{1+e^{(60+15D-GSI)/11}})$$

Intact modulus is calculated as :

$$E_i = MR * \sigma_{ci}$$

2.13.5 Defining Stress:

The vertical stress (σ_v) is obtained by multiplying the density by the overburden. The stress ratio (k) is obtained by the formula involving Poisson's Ratio (ν). The horizontal stress is obtained using the following relation.

$$\sigma_h = k * \sigma_v + \sigma_{tectonic}$$

2.14 Numerical Modelling

Although the numerical modeling cannot be used directly to analyze the squeezing phenomenon in the tunnels, its application can be utilized to find the deformation of the tunnel in squeezing environment and the results can be compared with the results that have been found from analytical, semi-analytical and empirical approaches. Empirical are truly based on practical aspects, however, the geology might be unique on excavation i.e. rock mass has complexity in nature(Karki et al., 2018) . So, numerical analysis helps in defining such parameters and complexity in nature of ground conditions.

The advantages of numerical analysis over other analysis are(Basnet, 2013):

- it is quantitative analysis,
- it provides better understanding of mechanism,
- it can be used to verify the results obtained from other methods,
- it provides the extension of measurement results from field and laboratory, etc.

The most commonly used numerical methods for tunnel and underground excavations design in rock engineering are(G. Barla, 2016):

1. Continuum methods
   It is one of the commonly used types of numerical models. Rock mass is modeled as a basically continuous medium, only a limited number of discontinuities (joints, faults etc.) may be included here. This method includes:
   - Finite Difference Method, FDM
   - Finite Element Method, FEM
   - Boundary Element Method, BEM

   In both finite element and finite difference modeling, a high number of material models may be used including linearly-elastic models, elastic-plastic models (Mohr-Coulomb, Generalized Hoek-Brown, Drucker-Prager strength models, etc.), and visco-plastic models.

2. Discontinuum methods
   Rock mass is modeled as system of individual blocks interacting along their boundaries. These models represent the nature of the rock mass closer to the reality. This method includes:
   - Discrete Element Method, DEM
   - Discontinuous Deformation Analysis, DDA
   - Particle Flow Method, PFC

   In discrete element modelling, blocks may be rigid or deformable according to a number of material models as mentioned above for finite element or finite difference modelling. Linear and non-linear force-displacement relations for movements in both the normal and shear direction govern motion along discontinuities.

3. Hybrid Continuum
    - Hybrid FEM/BEM
    - Hybrid DEM/BEM
    - Hybrid FEM/DEM

2.14.1 RS2 Software

RS2 is an extremely versatile 2D elasto-plastic finite element stress analysis program for designing underground or surface excavations and their support systems. RS2 can be used for rock or soil applications and includes finite element slope stability and groundwater seepage analysis.

RS2 is a powerful 2D finite element program for soil and rock applications. *RS2* can be used for a wide range of engineering projects including excavation design, slope stability, groundwater seepage, probabilistic analysis, consolidation, and dynamic analysis capabilities.

Complex, multi-stage models can be easily created and quickly analyzed – tunnels in weak or jointed rock, underground powerhouse caverns, open pit mines, and slopes, embankments, MSE stabilized earth structures, and much more. Progressive failure, support interaction and a variety of other problems can be addressed.

One of the major features of *RS2* is finite element slope stability analysis using the shear strength reduction method. This option is fully automated and can be used with various failure criteria, including Mohr-Coulomb and Generalized Hoek-Brown.

2.14.2 Methods of Analysis (Geotechnical Engineering Office, 2018)

Rock mass classification systems are typically used as the starting point in a design process, beginning at the feasibility study stage and continuing through the project planning 104 and preliminary design stages. They can also form the basis for determining rock support, especially the initial support, during construction for situations falling within the empirical database upon which the system is based. For more complicated settings, such as large span caverns in poor quality rock masses or where there are multiple junctions, rock mass classification systems should be used in combination with other appropriate design tools, such as analytical solutions and numerical analyses.

2.15 Engineering Geology and Mapping for Underground Caverns

The first thing before planning of any cavern is performing the required geotechnical investigation. Geological study of the site reveals crucial information such as rock types, presence and orientation of discontinuities, rock mass classification etc. This can help in estimation of many parameters and properties that are required later on during detailed analysis. Engineering geological study is primarily divided into surface and sub surface studies.

Without reliable geological information, planning decisions may be incorrect. There are numerous cases where tunnel projects benefited because either the horizontal or vertical alignment was dramatically changed as a result of geotechnical information.

Geological Mapping:

- It shows the distribution of rocks and soil, and the boundary between the different rocks should be marked
- The orientation of the different rock layers should be measured and showed by means of strike and dip sign
- Important structures such as folds, faults and fractures should be marked with indication of their orientation

Electrical Resistivity Tomography (ERT) :

Electrical resistivity tomography (ERT) or electrical resistivity imaging (ERI) is a geophysical technique for imaging subsurface structures from electrical resistivity measurements made at the surface, or by electrodes in one or more boreholes. If the electrodes are suspended in the boreholes, deeper sections can be investigated.

This measures the apparent resistivity of the ground materials and a resistivity image can be created, which maps the resistivity of the material with their location. ERT of headworks area of Super Madi can be seen in Figure 2-19.

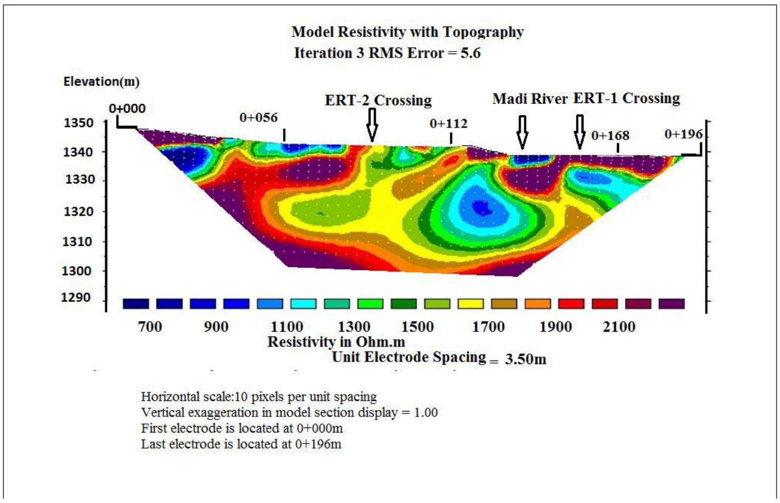


Figure 2-19 : 2-D ERT Profile with Corresponding Interpretive Cross-section at Headworks area (Gautam, 2012)

2.16 Location, Selection and Orientation of Underground Caverns

The basic approach to design any underground opening is to harness the self supporting capacity of the rock mass present. For good rock, the use of rock support should be to enhance this self supporting capacity rather than the support supporting the rock mass. The cavern installation should be optimised with respect to the topography and geology. The important factors in this optimisation are (Norwegian Tunnelling Society, 2016)

- adequacy of rock cover,
- avoidance of weakness zones or the crossing of them in the shortest possible distance
- avoidance of adverse orientation relative to major joint sets and weakness zones
- sufficient depth below the groundwater table (specifically for oil storage caverns)
- Avoidance of rock with abnormally low stresses, giving reduced confinement and aching effects
- avoidance of rock with very high stresses giving rise to rock burst

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 (Gautam, 2012)

When designing the layout for a cavern project, one normally seek to place the axis of the caverns as to have a favourable orientation in comparison with the major joint systems as well as the major stress (Norwegian Tunnelling Society, 2016). The figure below illustrates the optimum orientation of the cavern axis with respect to joints and discontinuities.

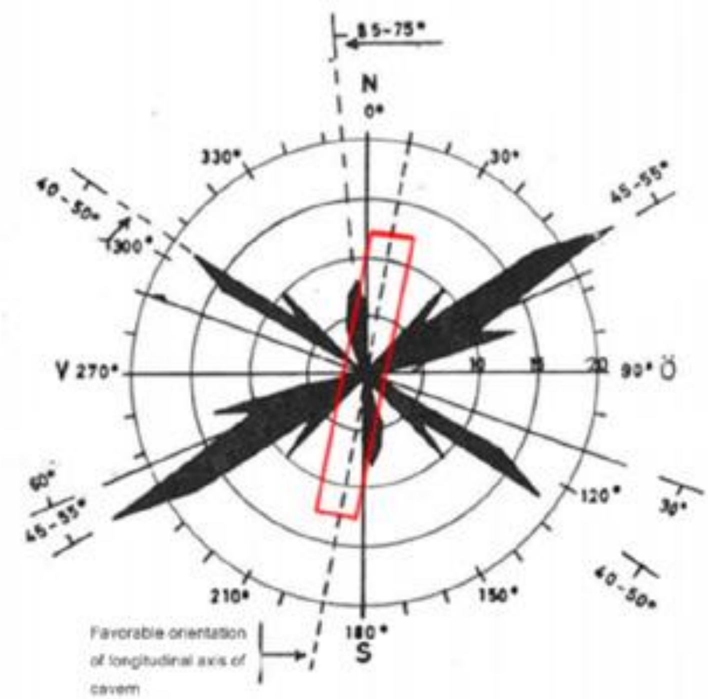


Figure 2-20 Favourable orientation of the axis of a cavern (Norwegian Tunnelling Society, 2016)

Figure 2-20 shows an example of a rosette plot. A rosette plot is basically a histogram of the discontinuities present. In this figure, we see two major discontinuities along 45-55° and 120-130° .For openings situated at shallow or intermediate depths, the longitudinal axis of cavernsis ideally oriented along the bisection line of the largest intersection angle of strikes of the two dominant sets of discontinuities(Geotechnical Engineering Office, 2018).

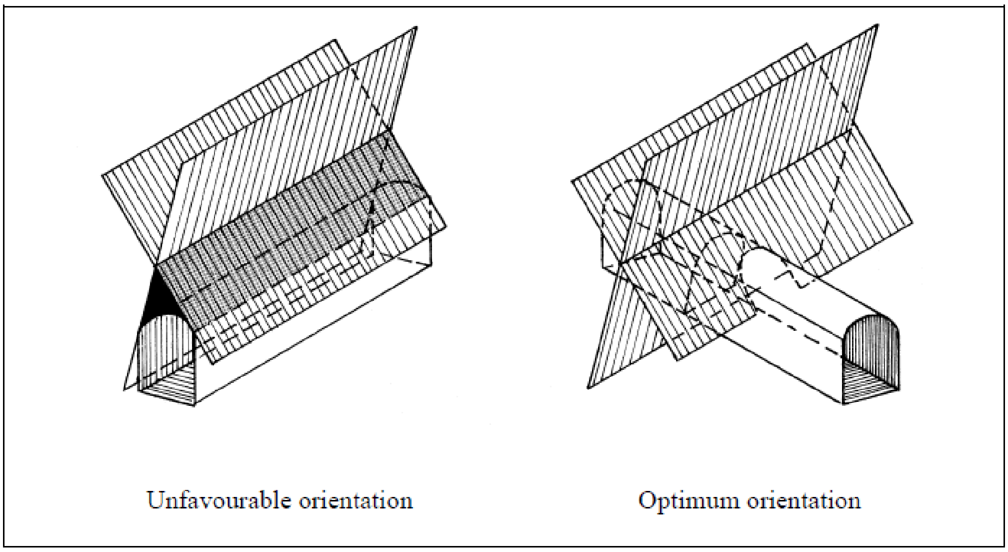


Figure 2-21 Orientation of Cavern with Respect to Discontinuity (Geotechnical Engineering Office, 2018)

Figure 2-21 illustrates the same thing. If the cavern is aligned along the discontinuity, then there is possibility of failure along the entire length whereas if we align the cavern as illustrated in figure 1-18, there is only possibility of localized failure where the alignment passes through the discontinuity

In the case of a hydropower project, the proximity to the weir and intake should also be taken into account for location of the cavern of settling basin.

The basic design concept for an underground cavern is to aim at evenly distributed compressive stresses in the rock mass surrounding the excavation. This is best obtained by giving the space a simple form with an arched roof.

Depending on the orientation of the main stresses the most favourable shapes are illustrated below.

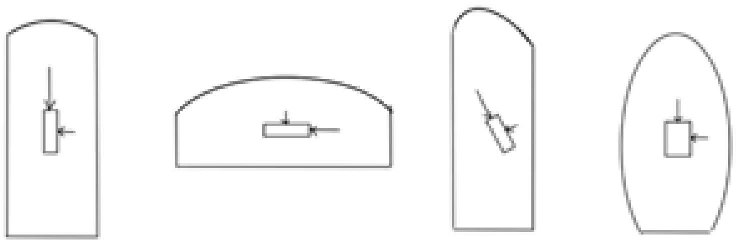


Figure 2-22 Optimum shape of cavern cross section depending on the orientation of the major stresses(Norwegian Tunnelling Society, 2016)

Cavern walls are normally vertical. This suits the method of excavation (benching) and yields little unusable space. Wall stability is a function of wall height, the in-situ stresses and the orientation and engineering properties of the principal joint sets. The flat wall surface precludes any substantial arching action and high walls tend to be unstable.

Many projects do not only involve a singular cavern, but several caverns, tunnels shafts etc. When caverns are placed in proximity of each other there will be an accumulation of stress in the rockmass separating the different openings. This might cause excessive strains in the rock mass causing instability, slabbing and spalling of the surface towards the openings.

Figure 2-23 Instability of pillar between underground openings(Norwegian Tunnelling Society, 2016)

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(Gautam, 2012). Typical pillar widths between caverns are between half andfull cavern span or height, whichever is the greater. As a project progresses on the basis of improved geological data, the pillar widths should be optimized, and appropriate lateral confinements to the pillars should be provided if necessary(Geotechnical Engineering Office, 2018).

A cavern development project should be planned in a way such that the flow of stresses around the chosen geometry of the excavation is smooth. Stress concentration, tensile stresses and induced slip zones should be avoided, especially in highly anisotropic stress fields (Geotechnical Engineering Office, 2018)

2.17 Failure and instability of underground caverns

Failures in underground excavations can be broadly divided into failure due to presence of discontinuities and failure of rock mass.

The tunnel passing through the faults have different stability problems. They are high deformation, over breaks, running ground and Squeezing (Bimal Chhushyabaga et al., 2020). Fault-related instability of underground openings are often presented as rock slide along the fault plane, the downfall of blocks, wedges cut by faults or minor joints inward the cavern(Liu et al., 2017). A rock slip along the fault plane is a typical response to insufficient shear strength of a fault.

Many faults and weakness zones contain materials quite different from the 'host' rock from hydrothermal activity and other geologic processes (Palmström, 1995).

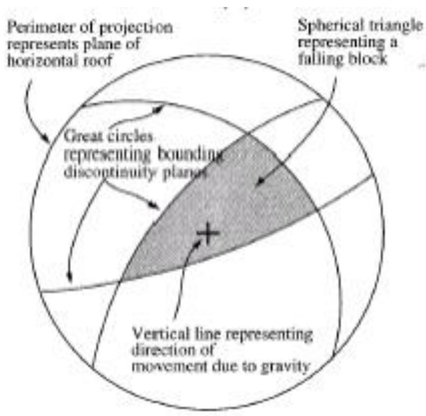


Figure 2-24 Formation of wedge in rock mass due to discontinuity. (Eberhardt, 2002)

Presence of intersecting discontinuities lead to formation of wedges, as illustrated in the Figure 2-24. This wedge is a block of rock mass that may fail at any time. In jointed rock mass stability of the blocks is primarily due to the interlocking of the discontinuities, thus loosening of one block may open up planes of movement for adjacent blocks making them unstable. This can lead to a chain reaction effect of wedge failure which will only cease when sufficient interlocking of discontinuities is established to attain static equilibrium, when a natural arch is formed or when the excavation is filled with rock (Bedi, 2004). This type of failure is also termed as structurally controlled failure.

The other is where failures are induced from overstressing, i.e. the stresses developed in the ground exceed the local strength of the material, which may occur in two main forms, namely(Bedi, 2004):

- Overstressing of massive or intact rock (which takes place in the mode of spalling, popping, rock burst etc.).
- Overstressing of particulate materials, i.e. soils and heavy jointed rocks (where squeezing and creep may take place).

Undisturbed ground is considered to be in equilibrium. Its inherent stress regime is one of the systems present in the ground. Stress can be characterized by magnitude and direction (Shrestha, 2005). When openings are excavated in rock, the existing stress regime is disturbed and stress redistributes itself around the opening as illustrated below in :

The process of spalling failure around a deep excavation in a rock mass is one of the main problems in tunnel construction. It consists in a brittle failure, produced by the crushing of compressed rock, during which strain localization appears as a shear crack (Fantilli & Vallini, 2006).

Rock burst is common in deep underground excavations and is characterised by a violent ejection of block rocks from excavation walls. Structural response of the rock mass, which can be divided into the following three phases as illustrated below in figure Figure 2-25.

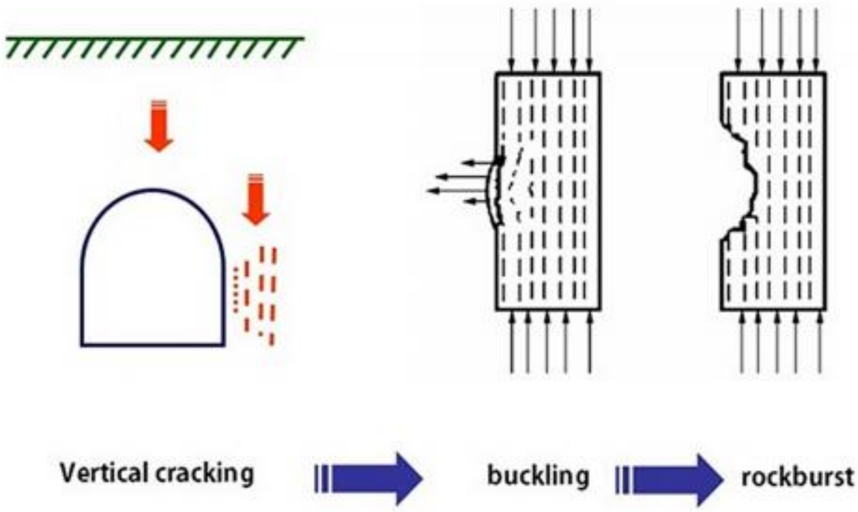


Figure 2-25 Rock Burst Mechanism (He & Sousa, 2014)

The squeezing of rock is a process of large deformation which occurs around the tunnel due to stress concentration and material properties (Ghiasi et al., 2012).

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 (Gautam, 2012).

2.18 Stability Analysis of Underground Cavern

Analyses of underground excavation are of three types: empirical, analytical and numerical. Empirical approaches are often implemented during the planning stages and used for preliminary design of support. For further detailed study in complicated cases, the empirical methods should be used in conjunction with analytical solutions and numerical analyses.

2.18.1 Empirical Approaches

There are a number of empirically based rock mass classification systems available. They can be applied to cavern design in appropriate circumstances. These rock mass classification are also valuable input data for the various empirical relations that help predict squeezing, support pressures, rock bursts etc.

For squeezing prediction, Singh et al 1992, Goel et. al 1995, Jiminez et. al. 2011 make use of the Q system. Similarly, the estimation of strain by Panthi & Shrestha, 2018 take into account various parameters like support pressure, shear modulus, deformation modulus.

The estimation of support pressure has also been attempted to be explained by empirical relations. Barton's Q is an input for calculation of support pressure in the relations given by Barton in 1974 or Bhasin and Grimstad.

NGI (2015) has advised that if Q<1, then deformation measurement and numerical analysis should be carried out in addition to the use of the Q-system.

RMR too has applications similar to the ones discussed above. One of the major applications however can be the estimation of “stand up time”. For tunnels with an arched roof the stand-up time is related to the rock mass class in the figure 9 below.

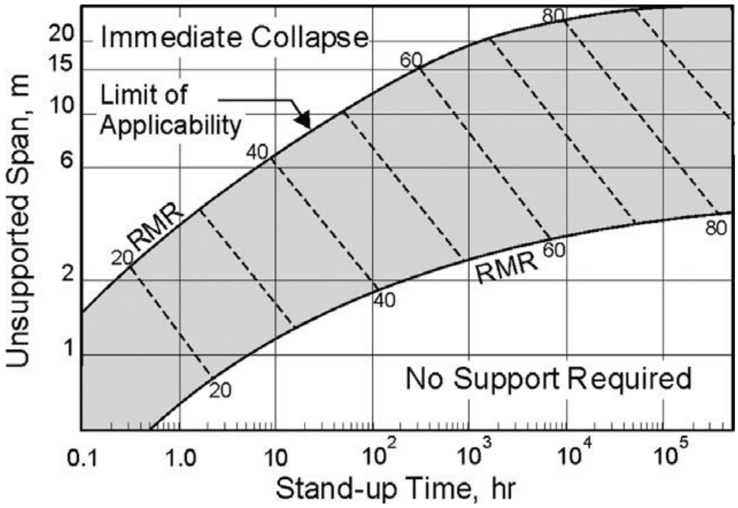


Figure 2-26 Stand-up time versus unsupported span for various rock mass classes according to RMR(Singh & Goel, 2011)

2.18.2 Analytical approaches

Analytical solutions can be useful for understanding of rock mass stability and assessing support requirements. The use of analytical solutions is particularly suitable for relatively simple scenarios. One of the most popular methods is the Convergence Confinement Method (CCM) by Carranza-Torres & Fairhurst, 2000. However, this method is based on two important assumptions:

- the excavation is circular
- the far-field principal stresses normal to the long axis of the tunnel are of constant magnitude and independent of the radial orientation

Apart from this method, Caverns typically situated at modest depths in hard rock where the stability is governed by weakness zones and intersection of discontinuities are studied using analytical solutions described below(Geotechnical Engineering Office, 2018) :

Limit Equilibrium Method: This method in various forms is often used to determine the stability of blocks and wedges at excavated rock surfaces and the support required to achieve stability, as explained by Hoek and Brown in 1980

Block Theory: Also referred to as the key block analysis, the theory can be used to determine which blocks in a cavern roof or wall control stability. It can be applied to predict the likely location and appearance of key blocks using statistical discontinuity data or discontinuity maps taken from an excavation. The method can assist in the recognition of key blocks in an excavation based on the observed discontinuity patterns on the excavation. Key block analysis is often carried out using stereographic projections. This method is described in detail by Goodman & Shi, 1985.

2.18.3 Numerical Analysis:

Situations where numerical methods are required cover, but are not limited to, the following:

- caverns with large spans,
- caverns constructed under a thin rock cover, in mixed ground conditions or intersected by weakness zones,
- caverns of complex geometry,
- multiple underground openings in the vicinity,
- irregular pillars or pillar ribs or intersection,
- presence of external imposed loads (e.g. foundation loads of existing structures) or other asymmetrical loading acting on the cavern, and
- Staged excavations.

The numerical methods are explained in Section 2.14

2.19 Literature regarding hydropower powerhouse

The structural complex where all the equipment for providing electricity is suitably arranged is a powerhouse. It accommodates electro-mechanical equipment such as turbine, generator, switch-gear, control room, engineer’s room, reception room and operator’s accommodation. Two basic requirements of powerhouse planning are functional efficiency and aesthetic beauty. One of the first choices is whether to locate the powerhouse in a building above ground called as surface powerhouse or to locate it as the underground powerhouse situated in caverns, excavated below the ground. The main purpose of the powerhouse building is to protect electro-mechanical equipment from adverse weather, allow for easy access for operations, and prevent mishandling of the equipment by unauthorized persons.

2.19.1 Classification of powerhouse

a) **Surface powerhouse**

Powerhouse complex is constructed above ground so the surface powerhouse has less space restrictions then the underground power stations. It is best suited when the powerhouse is located inside the rock mass which makes it more stable against flood effects and other external forces. But the foundation of surface powerhouse should be carefully examined. If solid bed rock is not available in surface powerhouse option, special foundation treatment is essential such as pile foundation, mat foundation etc.

b) **Sub surface powerhouse**

Some powerhouses are semi-underground structures being partly on surface and partly underground.

c) **Underground powerhous**e

Under special circumstances such as when gorge or valley forms is narrow providing not enough space for powerhouse, underground powerhouse are advantageous. The underground powerhouse is adopted in following conditions.

- If there are surface hazards like rock slides, snow avalanches etc. are frequent at the powerhouse location.
- The water conveyance length and penstock length can be shortened by providing the underground powerhouse.
- Bends, anchor blocks, support piers etc. can be minimized by providing underground powerhouse.
- If quality rock is available, the underground powerhouse shall be more appropriate then the surface powerhouse option.
- In some places where the cost of land is too expensive at the powerhouse area, underground powerhouse shall be more feasible option.

2.19.2 Powerhouse Structure

A powerhouse structure can be broadly visualized consisting of two main divisions. :

- Super structure and
- Sub structure

<u>Super structure</u>

The portion of the structure that is above ground level which receives the live load, dead load and other loads is referred to as Super structure. Following are super structures that we have planned to design in our project:

- Beam
- Column
- RCC gantry beam
- Corbel
- Slab
- Staircase

<u>Sub Structure</u>

Sub structure is an underlying or supporting structure to superstructure. It is below ground level. Foundation is part of substructure. Substructure is the lower portion of the building which transmits the dead load, live loads and other loads to the underneath sub soil. Following are the sub structures that we have planned to design in the project:

- Mat foundation
- Machine foundation
- Shear wall

2.19.3 Powerhouse Components Dimensioning

Three essential constituents (bay) of superstructure of powerhouse are:

- Machine hall or the unit bay
- Erection or the loading bay, and
- Control bay

Machine Hall

Length: Length of machine hall depends upon the number of units, the distance between the units, sizes of machines, and clearance. For the vertical alignment unit, the center to center distance between the units is controlled by the total width of the scroll casing layout. The standard distance of scroll casing is about 4.5D to 5D, where D is the outlet diameter of turbine. The minimum clearance of about 2 to 3 m is added to this distance. So, the center to center distance between the units is taken as (5D + 2.5) m. For higher specific speed, this distance can be reduced up to (4D + 2.5) m. The total length of the machine hall can be calculated by knowing the total number of unit required for particular project. One extra unit is placed for maintenance purpose, so space required for this also considered.

Width: Width of machine hall is also determined by the size and clearance space from the walls needed as gangway. Since gangway requirements are of the order of 2.5 m, as a first approximation and the width of the machine hall can be presumed to be (5D + 2.5) m. The width is kept as less as possible to keep small span of the girder and roof structure.

Height: The height of the machine hall is fixed up by head room requirement of crane operation. In general, 2 to 2.5 m head requirement is for crane operation. The hall must have a height which will enable the cranes to lift the rotor of the generator of the runner of the turbine clear off the floor without any other machines sets forming obstruction. To this clearance space is to be added the depth of crane girder and head room for the operating cabin.

Loading Bay

Loading bay, also known as erection bay or service bay, is a space where the heavy vehicles can be loaded and unloaded, the dismantled parts of the machines can be placed and where small assembling of the equipment’s can be done. The loading bay should be sufficient to receive the large parts like rotor and runner. The loading bay floor will be having a width at least equal to the center to center distance between the machines.

Control Bay

Control bay is the main room and control other equipment’s like runner, gate valves, generator etc. it may be adjacent to the unit bay i.e. machine halls as it sends instructions to the operation bay from where the operation control is received.

2.19.4 Input Parameters in SAP

The grid is formed according to plan of power house and control room, material we use is M25, Fe 500 for the concrete and steel respectively.

Required section for the components such as beams, columns, shear walls, slabs ..etc. of different units are assigned.

Load Pattern: In load pattern the different cases of dead load, live load and earthquake load acting on the powerhouse have been considered.

Mass Source:

Dead load is multiply with the factor 1 According to IS 456:2000 For Live Load < 3kN/m2 factor 0.25 For Live Load > 3kN/m2 Factor 0.5

Load Combinations:

We use the following 14 load combinations

a. 1.5 DL
b. 1.5(DL+LL)
c. 1.2(DL+LL+EQx)
d. 1.2(DL+LL-EQx)
e. 1.2(DL+LL+EQy)
f. 1.2(DL+LL-EQy)
g. 1.5(DL+EQx)
h. 1.5(DL-EQx)
i. 1.5(DL+EQy)
j. 1.5(DL-EQy)
k. 0.9DL+1.5EQx
l. 0.9DL-1.5EQx
m. 0.9DL+1.5EQy
n. 0.9DL-1.5EQy

Envelope is formed using these 14 load combinations. The function of envelope is to show the maximum value among above load combinations.

Restraint the base joint
The bases of the column are restrained.

Diaphragm the joints
The diaphragm constraints are defined to the joints at slab level so that all these joints move together with the slab in the same direction. For each floor diaphragm are formed except at the position where columns are restrained.

Meshing the area
Meshing of the area is done to transfer the slab load uniformly to the beam so that slab and beam deflect in a same pattern.

2.19.5 Seismic Parameters according to Code IS 1893

Seismic Weight: Seismic weight is the total dead load plus appropriate amount of specified amount of imposed load. The seismic weight of each floor is its full dead load plus appropriate amount of imposed load. While computing the seismic weight of each floor, the weight of columns and walls in any storey shall be equally distributed to the floors above and below the storey. The seismic weight of the whole building is the sum of seismic weight of each floor.

Any weight supported in between storeys shall be distributed to the floors above and below in inverse proportion to its distance from the floors.

The total design lateral force along any principal direction shall be determined by the following expression:

$$\mathrm{V_B} = \mathrm{A_h} . \mathrm{W}$$

Where,

A_h= design horizontal acceleration spectrum
W = seismic weight of the building
The design base shear computed shall be distributed along the height of the building as per the following expression:

$$\mathrm{Q} = \frac{W_i H_i^2}{\sum_{j=1}^{n} W_j H_j^2}$$

Where,

Q = design lateral force at floor I,
W_i = Seismic weight of floor i,
hi = height of floor i measure from the base and ,
n = number of storeys in the building is the number of levels at which masses are located.

2.19.6 Design Codes
For the analysis and design of the building references have been made to Indian Standard code since National Building Codes of Nepal do not provide sufficient information and refers frequently to the Indian standard codes. Indian Standard codes used in the analysis and design of this building are described below:

ii. **IS:875- 1987 (Reaffirmed 2003)- Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures**: This code is divided into five different parts for five different kinds of loadings. The different parts of the code that will be used are:

Part 1: Dead Loads- Unit Weight of Building Materials and Stored Materials

Part 2: Imposed Loads Imposed load is the load assumed to be produced by the intended use or occupancy of a building including the weight of moveable partitions, distributed, concentrated loads, loads due to impact and vibrations and dust loads (Excluding wind, seismic, snow, load due to temperature change, creep, shrinkage, differential settlements etc.).

Part 3: Wind Loads this code gives the wind force and their effect (Static and Dynamic) that should be taken into account when designing buildings, structures and components.

iii. **IS 1893 (Part 1): 2002 Criteria for Earthquake Resistant Design of Structures (General Provision and Building):** This code deals with the assessment of seismic loads on various structures and earthquake resistant design of buildings

iv. **IS 13920: 1993 (Reaffirmed 2003) Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Force:** This standard covers the requirements for designing and detailing of monolithic reinforced concrete buildings so as to give them adequate toughness and ductility to resist sever earthquake shock without collapse.

v. **IS 456: 2000 (Reaffirmed 2005) Plain and Reinforced:** This Indian Standard code of practice deals with the general structural use of plain and reinforced concrete based on Limit State Design Method.

vi. **SP 16: Design Aids for Reinforced Concrete to IS 456-1978:** This handbook explains the use of formulae mentioned in IS 456 and provides several design charts and interaction diagrams for flexure, deflection control criteria, axial compression, and compression with bending and tension with bending for rectangular cross-sections.

vii. **SP 34:1987:** Deals with Concrete Reinforcement and Detailing.

viii. **IS 4247 Part-1:** Deals with-structural design of Surface Electric Power Stations data for design.

ix. **IS 4247 Part – 2:** Deals with structural design of Surface Electric Power Stations Super Structure)

x. **IS 4247 Part – 3**: Deals with structural design of Surface Electric Power Stations – Sub Structure.

Chapter 3 Description of Case Study

3.1 General

Topography regime and abundance of water resources favors Nepal as a high potential for development of Hydropower projects. The steep terrain and fast flowing rivers fulfills the basic requirements. But due to the mountainous topography, it is difficult to divert the water to the feasible site. As such underground structures like tunnels and caverns provide economic and safe solution. For the study of tunnels for this project, a hydropower lying in middle Himalayan region have been selected. A brief description of the selected site has been provided in the following section.

In this chapter, we have explored the empirical and analytical methods of analysis of tunnel sections. Empirical methods of predicting squeezing and calculating the support pressures have been done and analytically, we have computed the deformation and support pressures. These results have been compared and inferences have been drawn.

From the empirical study of squeezing, there appears to be no squeezing rock for any of the section by all three methods. Support pressure values vary according to approach used. Goel and Jethwa approach indicate much lower values of support pressure than that given by Grimstad approach and Barton's approach.

The analytical approach indicates that the Q supports offer more factor of safety albeit at the cost of slightly higher deformations.

3.2 Description of Case Study

3.2.1 Project Description

Super Madi Hydroelectric Project is located in Kaski District of Nepal. The project lies about 23 km north-east of Pokhara.

The proposed project has installed capacity of 44MW, design discharge of 18m3/sec and net head of 295m. Two underground settling basins are fed with the discharge by two inlet tunnels from the headpond. An outlet pond after the settling basins feed water into headrace tunnel. The headrace tunnel of length 5905m of finished diameter of 3.6~4.4m and steel penstock pipe of average diameter 2.6m and length 1381m feed water to the three units of vertical axis Pelton Turbines installed in the semi-surface powerhouse to generate 44000kW power

Geographic map of Nepal with location of project in a global sense has been presented in **Figure 3-1**. 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 (Gautam, 2012).

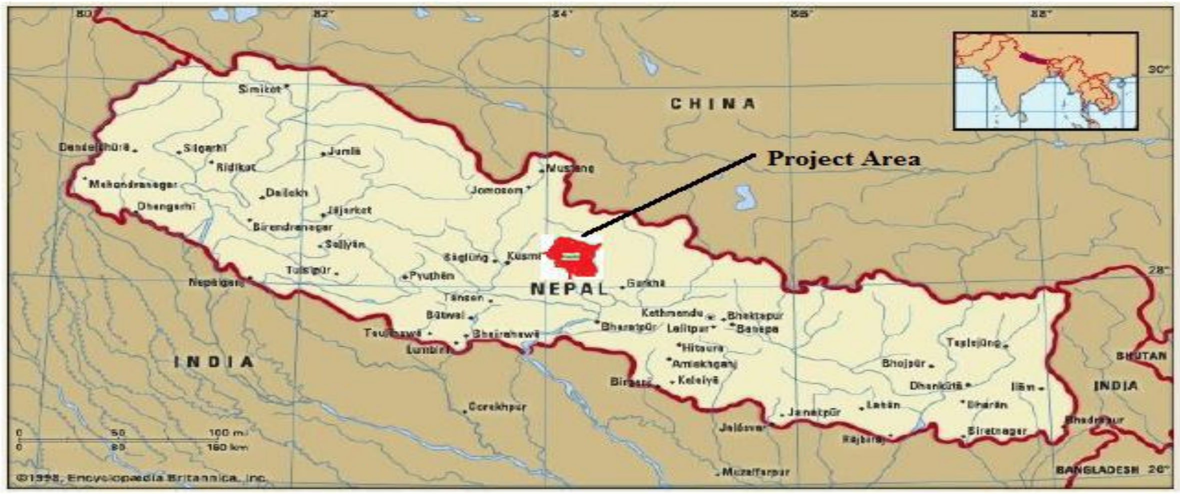


Figure 3-1 Location of Project Site

3.2.2 Geology of the Tunnel

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. Project area has medium to high-grade metamorphic rocks such as silimanite gneiss, kayanite schist and gneiss, augen and banded gneiss (Gautam, n.d.) . 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.

As seen from the profile per the feasibility study by Himal Hydro, the two major rock types found here are Banded Gneiss (between chainages 0m-1250m and 1600m -2000m))and Garnetiferous Schist (between chainages 3500m -5600m) . There exist 2 weakness zones; one near Kalbandi Khola and other between chainage 2000m-2500m. Near the powerhouse and penstock, the geology is crushed rocks.

The profile of SMHEP is shown in **Figure 3-2**

The support system according to RMR guidelines are given below:

RMR Range	Class	Support System
80-100	I	Occasional Spot Bolting as per requirement, no shotcrete lining
60-80	II	Pattern bolting at 1.5-2 m c/c , shotcrete lining of 50mm
40-60	III	Pattern bolting at 1.5 m c/c , shotcrete lining of 100mm
20-40	IV	Pattern bolting at 1 m c/c shotcrete of thickness 150 mm with steel ribs
0-20	V	Pattern bolting at 1 m c/c, shotcrete of thickness 200mm , provide steel ribs at 0.75m c/c , forepoling if face fails

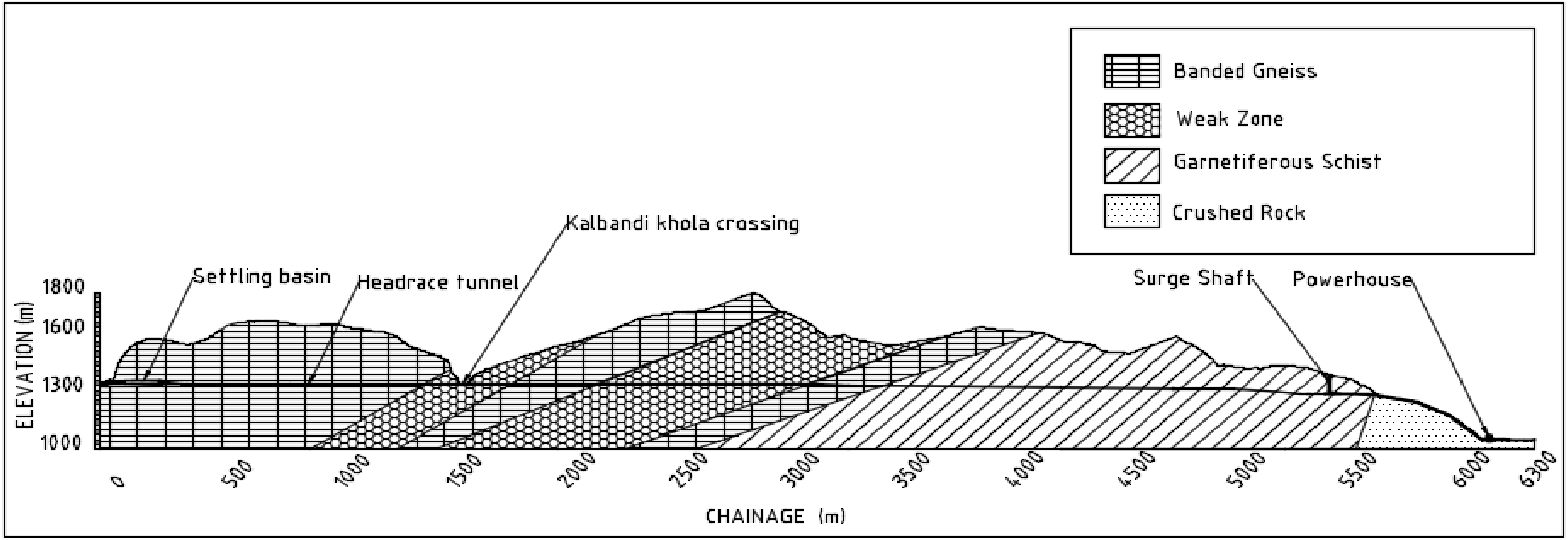


Figure 3-2 Profile of Super Madi Hydroelectric Project (Himal Hydro,2009)

3.3 Empirical Methods:

3.3.1 Squeezing Analysis of the ventilation tunnel of surge shaft:

3.3.2 Singh et. al. approach:
This approach by Singh et. al. relates the critical overburden required for squeezing to the rock mass classification by Q- value. This is an empirical relation based on study of 24 tunnel sections in the Himalayas. An empirical criterion was developed that gives a log-log plot between the tunnel depth H in metres and the logarithmic mean of the rock mass quality Q(Singh et al., 1992).

The critical overburden over which squeezing occurs is given by the equation $H = 350 \times Q^{1/3}$ where, H is overburden in metres

If $H > 350 \times Q^{1/3}$, squeezing rock

If $H < 350 \times Q^{1/3}$, non squeezing rock

Goel et. al. approach

The criterion of squeezing according to Goel is based on rock mass number (N). The rock mass number is basically the Q-value of the rock by setting the SRF as 1. This approach also takes into account the overburden (H) and width(B) of the opening. The equation that demarcates whether a tunnel section will squeeze or not is $H = 275 \times N^{0.33} \times B^{-0.1}$

If $H > 275 \times N^{0.33} \times B^{-0.1}$, squeezing occurs

If $H < 275 \times N^{0.33} \times B^{-0.1}$, no squeezing occurs

Table 3-1 Degree of squeezing according to Goel et. al., 1995

S. No.	Ground Condition	Correlation for predicting ground condition
1	Self-Supporting	$H < 23.4 \cdot N^{0.88} \cdot B^{-0.1}$ and $1000 \cdot B^{-0.1}$ and $B < 2 \cdot Q^{0.4}$
2	Non Squeezing	$23.4N^{0.88}B^{-0.1} < H < 275 \cdot N^{0.33}B^{-0.1}$
3	Mild Squeezing	$275 \cdot N^{0.33}B^{-0.1} < H < 450 \cdot N^{0.33}B^{-0.1}$ and $J_r/J_a < 0.5$
4	Moderate	$450 \cdot N^{0.33}B^{-0.1} < H < 630 \cdot N^{0.33}B^{-0.1}$ and $J_r/J_a < 0.5$
5	High Squeezing	$H > 630 \cdot N^{0.33}B^{-0.1}$ and $J_r/J_a < 0.25$
6	Mild Rock Burst	$HB^{0.1} > 1000$ and $J_r/J_a > 0.5$ and $N > 1.0$

Jiminez & Recio , 2001 approach

Jiminez and Recio in 2011 studied squeezing in 62 tunnel in the Himalayan region of India and Nepal and formulated their criterion for the prediction of squeezing. As with previous methods, this also relates the critical overburden over which squeezing will occur with the rock mass quality (i.e. the Q value). The line separating the squeezing and non-squeezing of tunnel section is given by the equation

$H = 424.4 \times Q^{0.32}$

Table 3-2 Squeezing according to empirical approaches:.

S.no.	Chainage		SRF	Q	N	B (m)	H (Goel et. al	H (Singh et. al.)	H(Jiminez et.al)	Overburden (m)	Ground Condition
1	0+000.00	0+000.00	10	0.023	0.234	4.20	100.162	127.687	147.597	25.8	No Squeezing
2	0+000.00	0+000.80	10	0.023	0.234	4.20	100.162	127.687	147.597	25.8	No Squeezing
3	0+000.80	0+002.10	10	0.023	0.234	4.20	100.162	127.687	147.597	27.12	No Squeezing
4	0+002.10	0+003.50	10	0.023	0.234	4.20	100.162	127.687	147.597	28.68	No Squeezing
5	0+003.50	0+004.90	10	0.023	0.234	4.20	100.162	127.687	147.597	30.39	No Squeezing
6	0+004.90	0+006.00	10	0.023	0.234	4.20	100.162	127.687	147.597	32.13	No Squeezing
7	0+006.00	0+006.00	10	0.023	0.234	4.20	100.162	127.687	147.597	33.48	No Squeezing
8	0+006.75	0+008.00	10	0.023	0.234	4.20	100.162	127.687	147.597	34.41	No Squeezing
9	0+008.00	0+009.25	5	0.229	1.146	4.20	214.183	264.865	249.183	35.96	No Squeezing
10	0+009.25	0+010.55	5	0.229	1.146	4.20	214.183	264.865	249.183	37.53	No Squeezing
11	0+010.55	0+011.95	5	0.229	1.146	4.20	214.183	264.865	249.183	39.21	No Squeezing
12	0+011.95	0+013.15	5	0.250	1.250	4.20	220.486	272.343	256.441	40.93	No Squeezing
13	0+013.15	0+014.60	5	0.229	1.146	4.20	214.183	264.865	249.183	42.51	No Squeezing
14	0+014.60	0+016.20	5	0.229	1.146	4.20	214.183	264.865	249.183	44.66	No Squeezing
15	0+016.20	0+017.80	10	0.078	0.781	4.20	149.623	187.702	219.598	46.95	No Squeezing
16	0+017.80	0+019.45	10	0.056	0.563	4.20	134.104	168.972	197.037	49.26	No Squeezing
17	0+019.45	0+021.15	10	0.078	0.781	4.20	149.623	187.702	219.598	51.64	No Squeezing
18	0+021.15	0+022.65	10	0.086	0.859	4.20	154.453	193.515	226.615	54.15	No Squeezing
19	0+022.65	0+024.20	10	0.078	0.781	4.20	149.623	187.702	219.598	56.35	No Squeezing
20	0+024.20	0+025.60	10	0.070	0.703	4.20	144.459	181.479	212.094	58.68	No Squeezing
21	0+025.60	0+027.20	10	0.078	0.781	4.20	149.623	187.702	219.598	60.76	No Squeezing
22	0+027.20	0+028.30	10	0.078	0.781	4.20	149.623	187.702	219.598	64.97	No Squeezing
23	0+028.30	0+029.45	10	0.094	0.938	4.20	158.998	198.979	233.216	63.25	No Squeezing
24	0+029.45	0+030.75	10	0.094	0.938	4.20	158.998	198.979	233.216	66.9	No Squeezing
25	0+030.75	0+032.25	10	0.078	0.781	4.20	149.623	187.702	219.598	68.71	No Squeezing
26	0+032.25	0+033.85	10	0.076	0.756	4.20	148.010	185.758	217.254	71.37	No Squeezing
27	0+033.85	0+035.30	10	0.086	0.859	4.20	154.453	193.515	226.615	74.24	No Squeezing

28	0+035.30	0+036.80	10	0.075	0.750	4.20	147.601	185.266	216.660	76.82	No Squeezing
29	0+036.80	0+038.40	10	0.075	0.750	4.20	147.601	185.266	216.660	79.96	No Squeezing
30	0+038.40	0+040.20	10	0.083	0.833	4.20	152.877	191.618	224.325	82.23	No Squeezing
31	0+040.20	0+041.70	10	0.070	0.703	4.20	144.459	181.479	212.094	85.19	No Squeezing
32	0+041.70	0+043.20	10	0.056	0.563	4.20	134.104	168.972	197.037	87.45	No Squeezing
33	0+043.20	0+044.60	10	0.056	0.563	4.20	134.104	168.972	197.037	89.77	No Squeezing
34	0+044.60	0+045.80	10	0.078	0.781	4.20	149.623	187.702	219.598	91.81	No Squeezing
35	0+045.80	0+046.95	10	0.056	0.563	4.20	134.104	168.972	197.037	93.8	No Squeezing
36	0+046.95	0+048.45	10	0.056	0.563	4.20	134.104	168.972	197.037	95.65	No Squeezing
37	0+048.45	0+049.80	10	0.078	0.781	4.20	149.623	187.702	219.598	98.02	No Squeezing
38	0+049.80	0+051.10	5	0.208	1.042	4.20	207.485	256.908	241.467	100.12	No Squeezing
39	0+051.10	0+052.50	5	0.229	1.146	4.20	214.183	264.865	249.183	101.99	No Squeezing
40	0+052.50	0+053.90	2.5	0.750	1.875	4.20	317.996	387.075	293.156	103.8	No Squeezing
41	0+053.90	0+055.25	2.5	0.750	1.875	4.20	317.996	387.075	293.156	105.79	No Squeezing
42	0+055.25	0+056.85	2.5	0.688	1.719	4.20	308.905	376.446	284.858	107.64	No Squeezing
43	0+056.85	0+058.15	2.5	0.750	1.875	4.20	317.996	387.075	293.156	109.89	No Squeezing
44	0+058.15	0+059.50	2.5	0.750	1.875	4.20	317.996	387.075	293.156	111.91	No Squeezing
45	0+059.50	0+061.00	2.5	0.500	1.250	4.20	277.795	339.974	256.441	113.99	No Squeezing
46	0+061.00	0+062.70	2.5	0.450	1.125	4.20	268.208	328.703	247.678	116.29	No Squeezing
47	0+062.70	0+064.15	2.5	0.667	1.667	4.20	305.753	372.757	281.980	118.91	No Squeezing
48	0+064.15	0+065.55	5	0.500	2.500	4.20	277.795	339.974	322.350	121.23	No Squeezing
49	0+065.55	0+067.15	5	0.458	2.292	4.20	269.854	330.639	313.226	123.46	No Squeezing
50	0+067.15	0+068.85	5	0.458	2.292	4.20	269.854	330.639	313.226	125.97	No Squeezing
51	0+068.85	0+070.50	2.5	0.688	1.719	4.20	308.905	376.446	284.858	128.72	No Squeezing
52	0+070.50	0+072.05	2.5	0.688	1.719	4.20	308.905	376.446	284.858	131.25	No Squeezing
53	0+072.05	0+073.55	2.5	0.688	1.719	4.20	308.905	376.446	284.858	133.8	No Squeezing
54	0+073.55	0+074.95	2.5	0.750	1.875	4.20	317.996	387.075	293.156	136.26	No Squeezing
55	0+074.95	0+076.45	2.5	0.688	1.719	4.20	308.905	376.446	284.858	138.66	No Squeezing
56	0+076.45	0+077.75	2.5	0.688	1.719	4.20	308.905	376.446	284.858	141.24	No Squeezing
57	0+077.75	0+079.35	2.5	0.688	1.719	4.20	308.905	376.446	284.858	143.49	No Squeezing
58	0+079.35	0+079.85	2.5	0.688	1.719	4.20	308.905	376.446	284.858	146.39	No Squeezing

59	0+079.85	0+081.45	2.5	0.550	1.375	4.20	286.762	350.503	264.635	147.25	No Squeezing
60	0+081.45	0+082.95	2.5	0.600	1.500	4.20	295.201	360.399	272.344	150.01	No Squeezing
61	0+082.95	0+084.55	2.5	0.625	1.563	4.20	299.246	365.138	276.038	152.58	No Squeezing
62	0+084.55	0+085.95	2.5	0.750	1.875	4.20	317.996	387.075	293.156	155.33	No Squeezing
63	0+085.95	0+087.35	2.5	0.733	1.833	4.20	315.623	384.301	290.990	157.65	No Squeezing
64	0+087.35	0+088.35	2.5	0.733	1.833	4.20	315.623	384.301	290.990	160.14	No Squeezing
65	0+088.35	0+089.70	2.5	0.667	1.667	4.20	305.753	372.757	281.980	161.76	No Squeezing
66	0+089.70	0+091.00	2.5	0.733	1.833	4.20	315.623	384.301	290.990	163.93	No Squeezing
67	0+091.00	0+092.10	2.5	0.600	1.500	4.20	295.201	360.399	272.344	166.16	No Squeezing
68	0+092.10	0+093.50	2.5	0.500	1.250	4.20	277.795	339.974	256.441	167.88	No Squeezing
69	0+093.50	0+095.30	2.5	0.667	1.667	4.20	305.753	372.757	281.980	170.43	No Squeezing
70	0+095.30	0+096.70	2.5	0.667	1.667	4.20	305.753	372.757	281.980	173.36	No Squeezing
71	0+096.70	0+098.00	2.5	0.667	1.667	4.20	305.753	372.757	281.980	175.67	No Squeezing
72	0+098.00	0+099.60	2.5	0.333	0.833	4.20	242.676	298.605	224.325	177.81	No Squeezing
73	0+099.60	0+101.00	2.5	0.333	0.833	4.20	242.676	298.605	224.325	180.45	No Squeezing
74	0+101.00	0+102.20	2.5	0.333	0.833	4.20	242.676	298.605	224.325	182.77	No Squeezing

3.4 Estimation of Support Pressures

3.4.1 Goel and Jethwa, 1991 approach

Using the measured support pressure values from 30 instrumented Indian tunnels, Goel and Jethwa (1991) proposed the following equation(Singh &Goel, 2011) :

$$P_v = \frac{7.5 \times B^{0.1} \times H^{0.5} - RMR}{20 \times RMR}$$

This equation is based on RMR value of rock mass. In the above equation, Pv is the support pressure in MPa, B is the span of opening in m, H is the overburden in m.

The results for this approach are tabulated below:

3.4.2 Barton et. al. 1974 approach

The ultimate support pressure for tunnel at roof and wall is provided by Barton et al. (1974) by correlating the support capacity of the opening with the rock mass quality given by Q-system of 200 case histories.

For roof support pressure:

$$P_v = \frac{0.2}{J_r} \times Q^{-0.33}\ MPa$$

For wall support pressure:

$$P_v = \frac{0.2}{J_r} \times {Q_w}^{-0.33} MPa$$

Where

Qw = Wall factor * Q

Jr= Joint Roughness Number

3.4.3 Bhasin and Grimstad approach

For the poor rock mass (that is value of Q less than 4) Bhasin and Grimstad recommended support pressure correlation depending upon the span of the underground opening and given as:

Pv=40×W/Jr×Q−0.33 kPa

Where,

Pv is the Support Pressure in kPa; W is the width of underground opening; and Jr is the Joint Roughness Number

Table 3-3 Support Pressure using empirical Approaches

Chainage		Overburden (m)	RMR (Rutledge and	Q value	Q with wall factor	Jr	Support Pressure using Barton et al (1974)(Mpa)		Support Pressure using	Support Pressure using Goel
Starting	Ending						Roof Pressure (Pv)	Wall Pressure (Ph)		
0+000.00	0+000.00	25.8	20.85	0.0234	0.02344	1.5	0.4601	0.4601	0.3865	0.0554
0+000.00	0+000.80	25.8	20.85	0.0234	0.02344	1.5	0.4601	0.4601	0.3865	0.0554
0+000.80	0+002.10	27.12	20.85	0.0234	0.02344	1.5	0.4601	0.4601	0.3865	0.0581
0+002.10	0+003.50	28.68	20.85	0.0234	0.02344	1.5	0.4601	0.4601	0.3865	0.0612
0+003.50	0+004.90	30.39	20.85	0.0234	0.02344	1.5	0.4601	0.4601	0.3865	0.0644
0+004.90	0+006.00	32.13	20.85	0.0234	0.02344	1.5	0.4601	0.4601	0.3865	0.0677
0+006.00	0+006.75	33.48	20.85	0.0234	0.02344	1.5	0.4601	0.4601	0.3865	0.0701
0+006.75	0+008.00	34.41	20.85	0.0234	0.02344	1.5	0.4601	0.4601	0.3865	0.0718
0+008.00	0+009.25	35.96	34.31	0.2292	0.57292	1.5	0.2168	0.1602	0.1821	0.0257
0+009.25	0+010.55	37.53	34.31	0.2292	0.57292	1.5	0.2168	0.1602	0.1821	0.0273
0+010.55	0+011.95	39.21	34.31	0.2292	0.57292	1.5	0.2168	0.1602	0.1821	0.0290
0+011.95	0+013.15	40.93	34.82	0.2500	0.62500	1.5	0.2107	0.1557	0.1770	0.0295
0+013.15	0+014.60	42.51	34.31	0.2292	0.57292	1.5	0.2168	0.1602	0.1821	0.0323
0+014.60	0+016.20	44.66	34.31	0.2292	0.57292	1.5	0.2168	0.1602	0.1821	0.0343
0+016.20	0+017.80	46.95	27.96	0.0781	0.07813	1.5	0.3093	0.3093	0.2598	0.0561
0+017.80	0+019.45	49.26	26.02	0.0563	0.05625	1.5	0.3447	0.3447	0.2895	0.0668
0+019.45	0+021.15	51.64	27.96	0.0781	0.07813	1.5	0.3093	0.3093	0.2598	0.0613
0+021.15	0+022.65	54.15	28.52	0.0859	0.08594	1.5	0.2997	0.2997	0.2517	0.0617
0+022.65	0+024.20	56.35	27.96	0.0781	0.07813	1.5	0.3093	0.3093	0.2598	0.0662
0+024.20	0+025.60	58.68	27.34	0.0703	0.07031	1.5	0.3202	0.3202	0.2690	0.0713
0+025.60	0+027.20	60.76	27.96	0.0781	0.07813	1.5	0.3093	0.3093	0.2598	0.0707
0+027.20	0+028.30	64.97	27.96	0.0781	0.07813	1.5	0.3093	0.3093	0.2598	0.0748
0+028.30	0+029.45	63.25	29.03	0.0938	0.09375	1.5	0.2912	0.2912	0.2446	0.0686

0+029.45	0+030.75	66.9	29.03	0.0938	0.09375	1.5	0.2912	0.2912	0.2446	0.0719
0+030.75	0+032.25	68.71	27.96	0.0781	0.07813	1.5	0.3093	0.3093	0.2598	0.0783
0+032.25	0+033.85	71.37	27.77	0.0756	0.07563	1.5	0.3126	0.3126	0.2626	0.0817
0+033.85	0+035.30	74.24	28.52	0.0859	0.08594	1.5	0.2997	0.2997	0.2517	0.0808
0+035.30	0+036.80	76.82	27.72	0.0750	0.07500	1.5	0.3135	0.3135	0.2633	0.0869
0+036.80	0+038.40	79.96	27.72	0.0750	0.07500	1.5	0.3135	0.3135	0.2633	0.0896
0+038.40	0+040.20	82.23	28.34	0.0833	0.08333	1.5	0.3027	0.3027	0.2543	0.0885
0+040.20	0+041.70	85.19	27.34	0.0703	0.07031	1.5	0.3202	0.3202	0.2690	0.0962
0+041.70	0+043.20	87.45	26.02	0.0563	0.05625	1.5	0.3447	0.3447	0.2895	0.1056
0+043.20	0+044.60	89.77	26.02	0.0563	0.05625	1.5	0.3447	0.3447	0.2895	0.1076
0+044.60	0+045.80	91.81	27.96	0.0781	0.07813	1.5	0.3093	0.3093	0.2598	0.0984
0+045.80	0+046.95	93.8	26.02	0.0563	0.05625	1.5	0.3447	0.3447	0.2895	0.1111
0+046.95	0+048.45	95.65	26.02	0.0563	0.05625	1.5	0.3447	0.3447	0.2895	0.1127
0+048.45	0+049.80	98.02	27.96	0.0781	0.07813	1.5	0.3093	0.3093	0.2598	0.1033
0+049.80	0+051.10	100.12	33.75	0.2083	0.52083	1.5	0.2237	0.1654	0.1879	0.0784
0+051.10	0+052.50	101.99	34.31	0.2292	0.57292	1.5	0.2168	0.1602	0.1821	0.0774
0+052.50	0+053.90	103.8	41.30	0.7500	1.87500	1.5	0.1466	0.1084	0.1232	0.0568
0+053.90	0+055.25	105.79	41.30	0.7500	1.87500	1.5	0.1466	0.1084	0.1232	0.0578
0+055.25	0+056.85	107.64	40.79	0.6875	1.71875	1.5	0.1509	0.1115	0.1267	0.0601
0+056.85	0+058.15	109.89	41.30	0.7500	1.87500	1.5	0.1466	0.1084	0.1232	0.0599
0+058.15	0+059.50	111.91	41.30	0.7500	1.87500	1.5	0.1466	0.1084	0.1232	0.0609
0+059.50	0+061.00	113.99	38.91	0.5000	1.25000	1.5	0.1676	0.1239	0.1408	0.0688
0+061.00	0+062.70	116.29	38.29	0.4500	1.12500	1.5	0.1735	0.1283	0.1458	0.0719
0+062.70	0+064.15	118.91	40.61	0.6667	1.66667	1.5	0.1524	0.1126	0.1280	0.0662
0+064.15	0+065.55	121.23	38.91	0.5000	1.25000	1.5	0.1676	0.1239	0.1408	0.0725
0+065.55	0+067.15	123.46	38.40	0.4583	1.14583	1.5	0.1725	0.1275	0.1449	0.0753
0+067.15	0+068.85	125.97	38.40	0.4583	1.14583	1.5	0.1725	0.1275	0.1449	0.0765

0+068.85	0+070.50	128.72	40.79	0.6875	1.71875	1.5	0.1509	0.1115	0.1267	0.0704
0+070.50	0+072.05	131.25	40.79	0.6875	1.71875	1.5	0.1509	0.1115	0.1267	0.0716
0+072.05	0+073.55	133.8	40.79	0.6875	1.71875	1.5	0.1509	0.1115	0.1267	0.0728
0+073.55	0+074.95	136.26	41.30	0.7500	1.87500	1.5	0.1466	0.1084	0.1232	0.0723
0+074.95	0+076.45	138.66	40.79	0.6875	1.71875	1.5	0.1509	0.1115	0.1267	0.0750
0+076.45	0+077.75	141.24	40.79	0.6875	1.71875	1.5	0.1509	0.1115	0.1267	0.0761
0+077.75	0+079.35	143.49	40.79	0.6875	1.71875	1.5	0.1509	0.1115	0.1267	0.0771
0+079.35	0+079.85	146.39	40.79	0.6875	1.71875	1.5	0.1509	0.1115	0.1267	0.0784
0+079.85	0+081.45	147.25	39.47	0.5500	1.37500	1.5	0.1624	0.1200	0.1364	0.0831
0+081.45	0+082.95	150.01	39.99	0.6000	1.50000	1.5	0.1578	0.1166	0.1326	0.0826
0+082.95	0+084.55	152.58	40.23	0.6250	1.56250	1.5	0.1557	0.1151	0.1308	0.0829
0+084.55	0+085.95	155.33	41.30	0.7500	1.87500	1.5	0.1466	0.1084	0.1232	0.0806
0+085.95	0+087.35	157.65	41.17	0.7333	1.83333	1.5	0.1477	0.1092	0.1241	0.0820
0+087.35	0+088.35	160.14	41.17	0.7333	1.83333	1.5	0.1477	0.1092	0.1241	0.0831
0+088.35	0+089.70	161.76	40.61	0.6667	1.66667	1.5	0.1524	0.1126	0.1280	0.0856
0+089.70	0+091.00	163.93	41.17	0.7333	1.83333	1.5	0.1477	0.1092	0.1241	0.0846
0+091.00	0+092.10	166.16	39.99	0.6000	1.50000	1.5	0.1578	0.1166	0.1326	0.0895
0+092.10	0+093.50	167.88	38.91	0.5000	1.25000	1.5	0.1676	0.1239	0.1408	0.0941
0+093.50	0+095.30	170.43	40.61	0.6667	1.66667	1.5	0.1524	0.1126	0.1280	0.0892
0+095.30	0+096.70	173.36	40.61	0.6667	1.66667	1.5	0.1524	0.1126	0.1280	0.0904
0+096.70	0+098.00	175.67	40.61	0.6667	1.66667	1.5	0.1524	0.1126	0.1280	0.0913
0+098.00	0+099.60	177.81	36.52	0.3333	0.83333	1.5	0.1916	0.1416	0.1609	0.1081
0+099.60	0+101.00	180.45	36.52	0.3333	0.83333	1.5	0.1916	0.1416	0.1609	0.1092
0+101.00	0+102.20	182.77	36.52	0.3333	0.83333	1.5	0.1916	0.1416	0.1609	0.1103

3.5 Analytical Method

The following parameters were input to obtain the Longitudinal Displacement Profile, Ground Reaction Curve and Support Characteristic Curves for 4 sections of the ventilation tunnel (Bagalethar Adit) of the surge shaft cavern. The SCC has been obtained for both Q-supports and RMR Supports. The factor of safety for each support assessment can be defined as the ratio of maximum support pressure the support system can withstand to the support pressure exerted on it.

Table 3-4 Input Parameters for CCM method (chainage 0+101.00m)

Tunnel Geometry	
Chainage (m)	0 + 101.00
Overburden (m):	182.77
Loading (MPa)	4.93
Face Effect (m)	1.2

Rock Mass Parameters	
Intact UCS (MPa)	78
mi	28
Poisson's Ratio	0.35
GSI	30
Unit Weight (MN/m3)	0.027

Q- Support Parameters

Shotcrete	
σ_{cc} (MPa)	30
E_c (MPa)	30000
υ_c	0.25
t_c (mm)	80

Rock bolts	
d_b (mm)	25
L (m)	2.5
T_{bf} (MN)	0.254
Q_b (m/MN)	0.143
E_b(GPa)	210
n_b	10
S_b(m)	1.5

RMR Support Parameters

Shotcrete	
σ_{cc} (MPa)	30
E_c (MPa)	300000
υ_c	0.25
t_c (mm)	150

Rock bolts	
d_b (mm)	25
l (m)	4
T_{bf} (MN)	0.254
Q_b (m/MN)	0.143
E_b (GPa)	210
n_{bolt}	10
s_l (m)	1.5

Steel Set	
A_s (m^2)	0.00191
R (m)	2.43
E_s (MPa)	210000
σ_{ys} (MPa)	150
S (m)	1

The output parameters for were obtained as:

m_b	s	E_{rm}	G_{rm}	K_ψ	S_0	Pi^{cr}	pi^{cr}
2.7477	0.0007	3.7243	1.3794	1.0000	0.0231	0.0018	0.3684

3.6 Computations:

3.6.1 Longitudinal Displacement Profile:

The calculations of the LDP at chainage 0 + 101.00 are as follows. The maximum displacement is 4.65 mm when the face has advanced 19.44 m from the section. For our purpose, support is installed when face is 1.2 m from section (face effect). So the radial displacement for L_f = 2.11 m

Table 3-5 Computation for LDP (Chainage 0+ 101.00m)

point	L_f/R	L_f (m)	u_r(mm)
1	-4.0000	-9.7200	0.01
2	-2.9091	-7.0691	0.05
3	-1.8182	-4.4182	0.21
4	-0.7273	-1.7673	0.75
5	0.3636	0.8836	1.86
6	1.4545	3.5345	3.12
7	2.5455	6.1855	3.97
8	3.6364	8.8364	4.38
9	4.7273	11.4873	4.55
10	5.8182	14.1382	4.62
11	6.9091	16.7891	4.64
12	8.0000	19.4400	4.65

3.6.2 Ground Reaction Curve:

The calculations of the GRC is done in 2 (two) parts; elastic and plastic. The following two tables

Table 3-6 Calculations for the elastic part of the GRC

point	p_l	u_r^{el}
1	4.9348	0.00
2	0.3684	4.02

Table 3-7 Calculations for the plastic part of GRC

point	p_i (MPa)	p_i^r (MPa)	p_i^f (MPa)	P_i	R_{pl}/R	u_r^{pl} (mm)
1	0.3684	0.3684	0.3684	0.0018	1.0000	4.02
2	0.3349	0.3352	0.3347	0.0017	1.0038	4.05
3	0.3014	0.3020	0.3009	0.0015	1.0077	4.09
4	0.2680	0.2687	0.2672	0.0013	1.0119	4.12
5	0.2345	0.2355	0.2334	0.0012	1.0163	4.16
6	0.2010	0.2023	0.1996	0.0010	1.0211	4.20
7	0.1675	0.1692	0.1657	0.0009	1.0263	4.25
8	0.1340	0.1361	0.1319	0.0007	1.0320	4.30
9	0.1005	0.1030	0.0980	0.0006	1.0384	4.36
10	0.0670	0.0700	0.0640	0.0004	1.0458	4.43
11	0.0335	0.0371	0.0299	0.0003	1.0549	4.52
12	0.0000	0.0044	-0.0044	0.0001	1.0678	4.66

3.6.3 Support Capacity Curves

The support capacity curves are calculated as:

For Q Support

Table 3-8 Calculations for SCC (Q system)

Shotcrete		Rock bolt		Combined Support	
p_c^{max}	0.97	s_c	1.53	K_{c+b}	443.53
K_c	440.92	$P_b max$	0.11	U_{c+b}	2.20
u_c^{max}	2.20	K_b	2.61	P_{c+b}	0.98
		$u_b max$	42.49		
p_i(MPa)	u_r (mm)	p_i(MPa)	u_r (mm)	P_i	u_r
0	2.01	0	2.01	0	2.1120
0.9714	4.22	0.11	44.50	1.7998	4.2495
0.9714	10.82	0.11	171.95	1.7998	10.6620

For RMR Support

Shotcrete		Rockbolt		Steel Sets	
p_c^{max}	1.79	s_c	1.53	p_{ss}^{max}	0.12
K_c	839.62	p_b^{max}	0.11	K_{ss}	67.93
u_c^{max}	2.14	K_b	2.40	$u_{ss} max$	1.74
		$u_b max$	46.18		

p_i(MPa)	u_r (mm)	p_i(MPa)	u_r (mm)	p_i(MPa)	u_r (mm)
0	2.11	0	2.112	0	2.11
1.7947	4.25	0.11	48.3	0.1179	3.85
1.7947	10.66	0.11	186.84	0.1179	9.05

Combined Support	
K_{s+b+c}	909.95
U_{s+b+c}	1.74
P_{s+b+c}	1.58
P_i	u_r
0	2.11
1.5794	3.85
1.5794	9.05

The results for the various sections are shown below

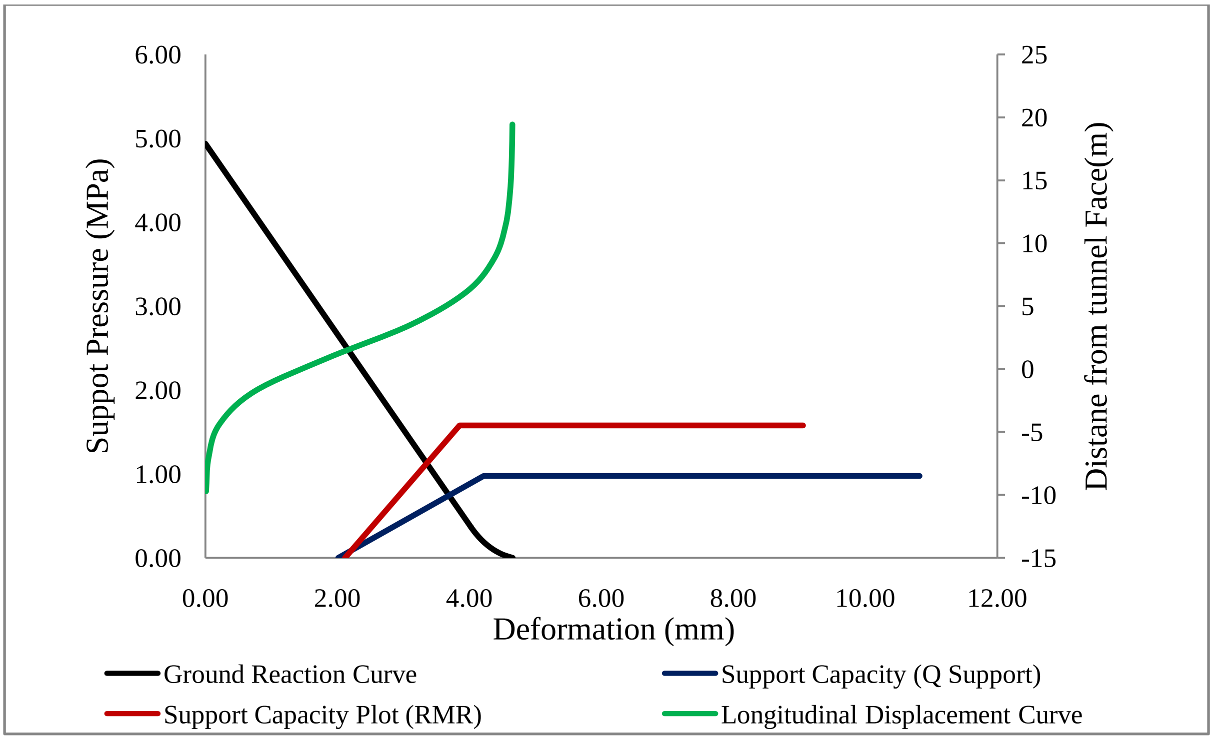


Figure 3-3 Convergence Confinement Method For Chainage (0 + 101.00 m)

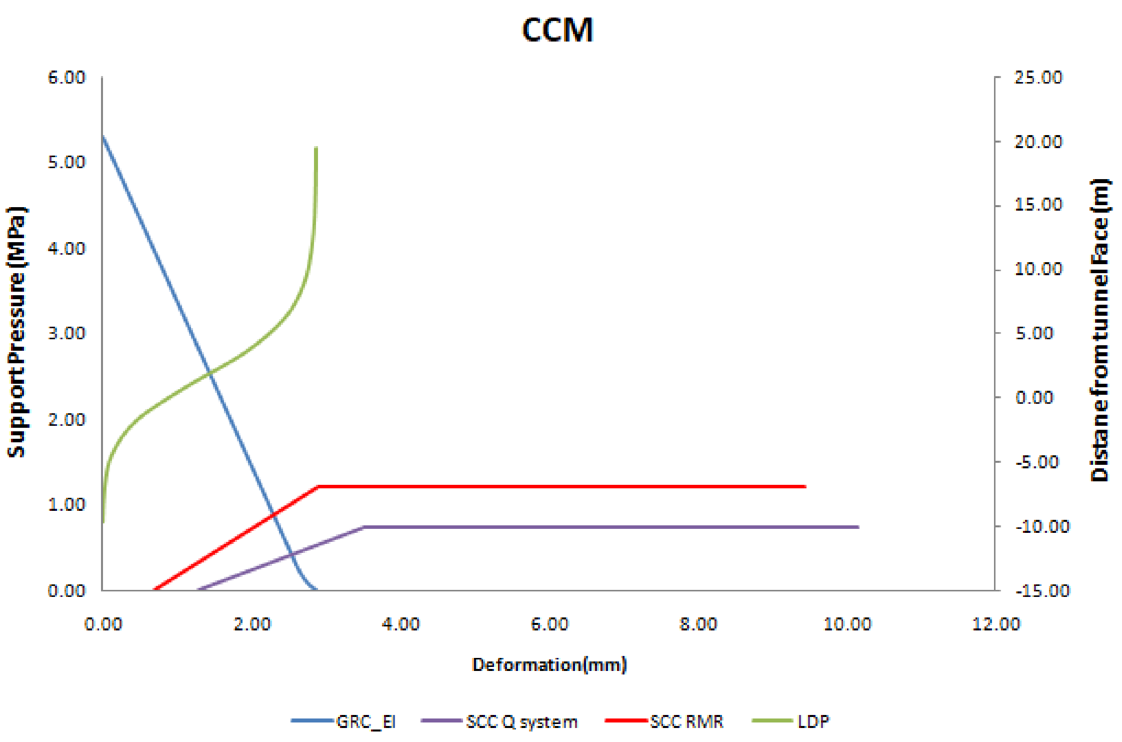


Figure 3-4 Convergence confinement method for chainage 0 + 58.15 m

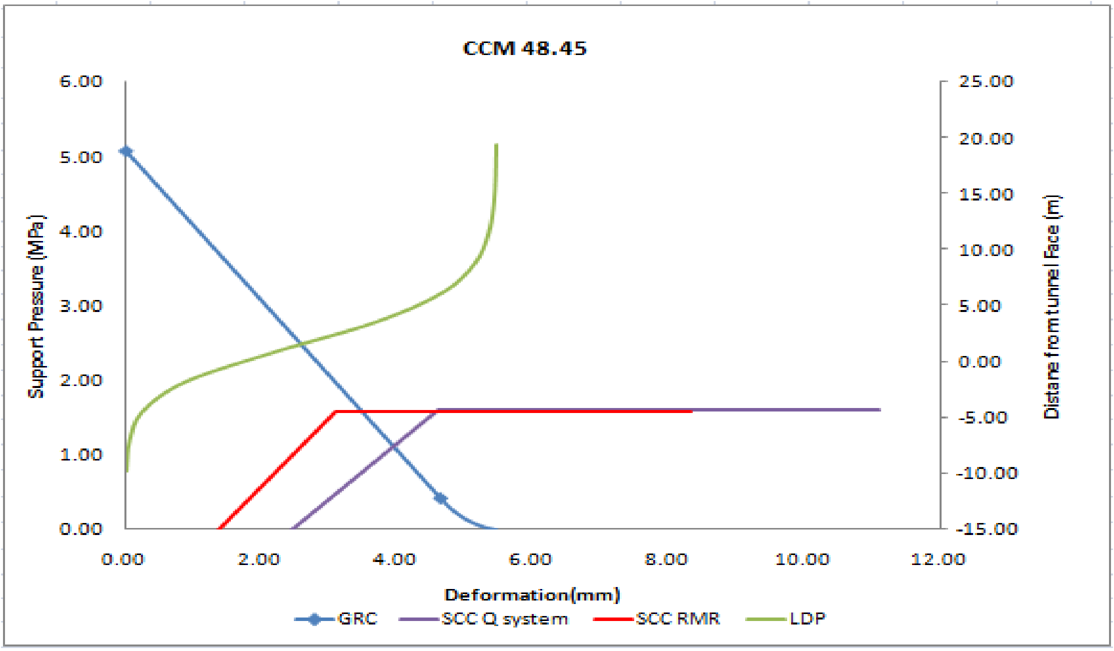


Figure 3-5 Convergence confinement method for chainage 0 + 48.45 m

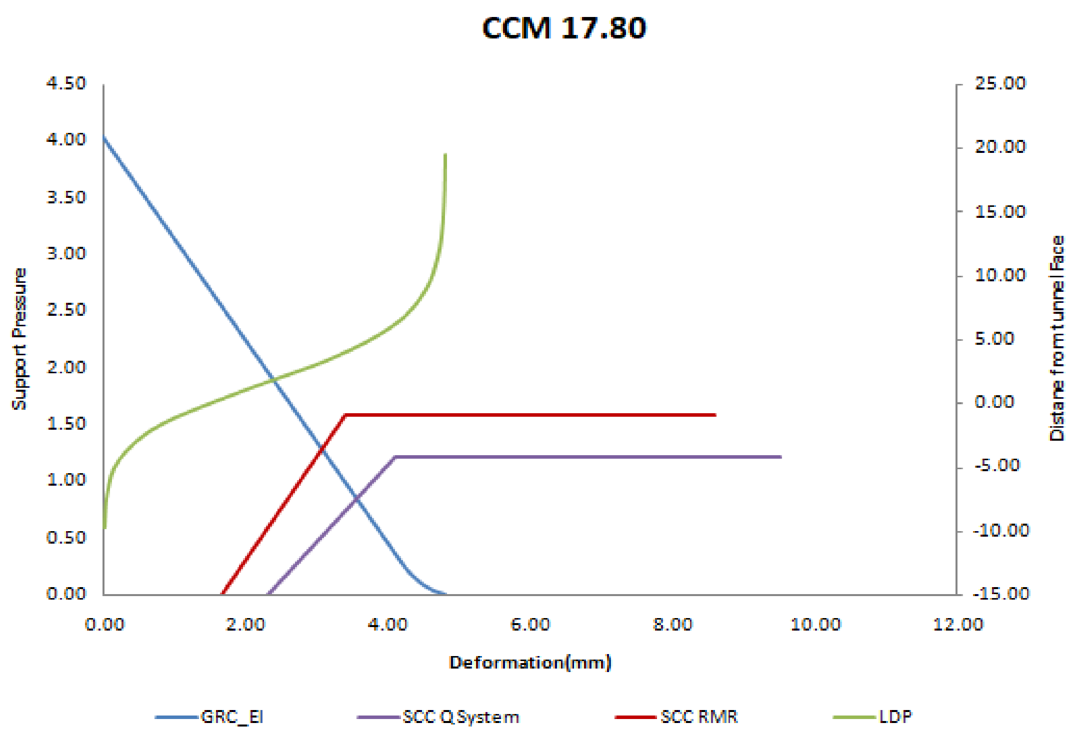


Figure 3-6 Convergence Confinement Method for Chainage 0 + 17.8 m

From the CCM Method, the following results were obtained :

Table 3-9 Results from CCM Analysis

Chainage	Design guideline	Displacement (mm)	Maximum Pressure (MPa)	Support Pressure (MPa)	Factor of Safety
0 + 17.80	RMR	3.08	1.657	1.285	1.29
	Q	3.56	1.318	0.840	1.57
0 + 48.45	RMR	1.73	1.57	1.57	1.0 (yielded)
	Q	3.94	1.599	1.103	1.45
0 + 58.15	RMR	2.29	1.20	0.880	1.375
	Q	2.54	0.869	0.415	2.096
0 + 101.00	RMR	3.35	1.569	1.129	1.39
	Q	3.69	1.107	0.743	1.49

Comparison of the results from Analytical and Empirical Approaches

We have calculated the Maximum Support Pressure from Empirical Approach. Then we calculated the support pressures from the Analytical method using CCM for the support system defined from RMR and also the support system defined using Q chart. Values of Support Pressure from both the approaches are presented below:

Table 3-10 Comparision of support pressure from analytical and empirical method

Chainage	Empirical (MPa)	Analytical (MPa)	
		RMR guideline	Q System
0 + 17.80 m	0.3447	1.285	0.840
0 + 48.50 m	0.3093	1.570	1.183
0 + 58.15 m	0.1084	0.880	0.415
0 + 101.0 m	0.1916	1.129	0.743

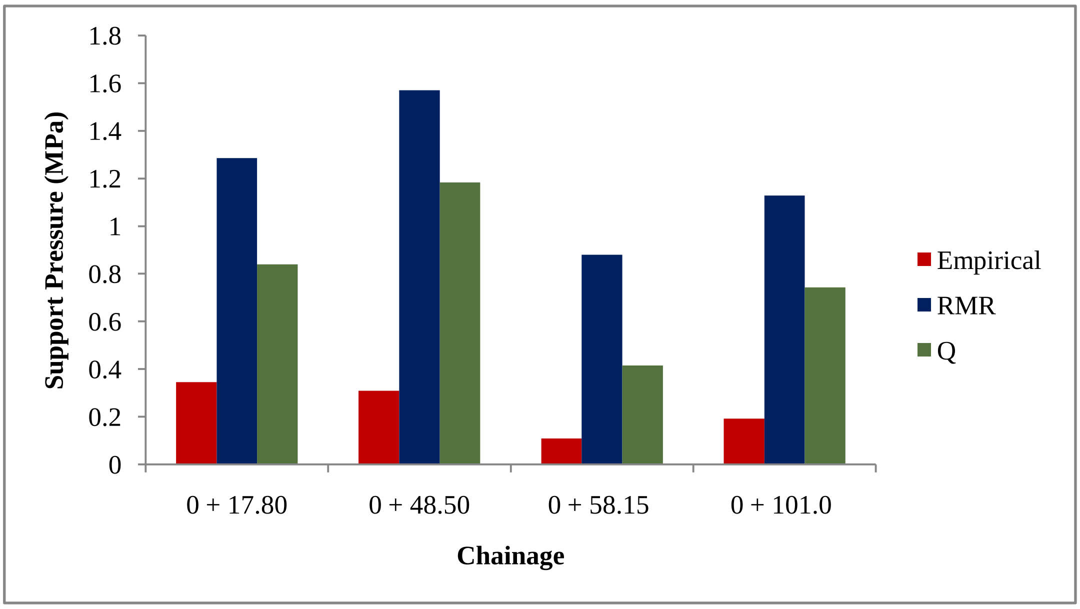


Figure 3-7Comparison of Support Pressure

Interpretation of the results from Analytical and Empirical Approaches

The comparison of results of RMR and Q support show that the RMR supports can withstand more pressure than the Q support. However the support system recommended by RMR guideline is also stiffer than the Q support. This means that the supports provided by the RMR guidelines show elastic behavior in a shorter range of deformation than that of the Q support. Although deformation for the RMR guidelines is slightly lesser than Q support, the factor of safety for the Q supports is actually higher. This may be explained by the fact that the Q supports act over a wider range of deformation as evident by the lesser slope of the SCC and the fact that the support is activated after more deformation has occurred which means that ground has relaxed more and pressure is lesser. Evident from the graphs, the support pressure taken by the Q support is also lower than the RMR supports.

For comparison with empirical approach, the support pressure given by Bhasin and Grimstad has been chosen for it is an empirical approach that is specific for rock mass with Q value less than 4 (which is the case for the sections we have taken).

The discrepancy between empirical and analytical approach is significant. It may be explained by the fact that empirical relations are dependent on two or three measurable quantities and discount many factors that actually affect the final result while the analytical approach in form of CCM takes into account the ground conditions and the support properties.

Chapter 4 Assessment and Analysis of Underground Structure

4.1 General

This chapter deals with the numerical analysis of the underground structures (tunnel sections and the settling basin cavern sections). Numerical modeling in rock and civil engineering is used as a tool that facilitates the site engineers to evaluate the rock mass behavior and its effects on engineering structures and support systems. There are different numerical models available for use like FEM, FDM, DEM etc; we have carried out FEM using RS2 software. In FEM, the rock mass is modeled as a continuum and the discontinuities modeled discretely in the continuum model

This chapter can be understood as composed of 3 distinct parts: analysis of tunnel section, analysis of a squeezing case of tunnel and analysis of the settling basin cavern.

4.2 Modeling of tunnel

Modeling of tunnel using RS2 is done by considering uniform geometry having same physical property of rock layer throughout the section. Numerical modeling in includes following steps:

a) Defining the geometry
b) Boundary condition
c) Defining material properties
d) Discretization and Mesh generation
e) Defining loading condition
f) Computation
g) Interpretation

a) Defining the geometry: Required shape of the tunnel is defined using the co-ordinate system as per required dimensions of the section considered in S.I units.

b) Boundary condition: The defined section of the tunnel is confined by the external rectangular boundary with the expansion factor of 3 neglecting the existing terrains and rock masses.

c) Defining material properties: Intact strength of rock mass, Modulus of elasticity, Poisson's ratio, Hoek-Brown constants and other properties of the rock are entered in the dialogue box. Only plastic analysis on the sections are done thus Generalized Hoek-Brown is opted as it is more suitable and realistic approach compared to Mohr-Coulomb criteria.

d) Discretization and Mesh generation: RS2 consists of two-dimensional automatic finite element mesh generator, which generates mesh based on either triangular or quadrilateral finite elements. The rock mass is discretized into a number of elements and connected by means of mesh by the software automatically as per our request. The density of the mesh is higher near the tunnel boundary as higher precision is required. Graded 3 noded striangular mesh was created for modelling with gradation factor of 0.1

e) Defining loading condition: Field stress can be taken constant or gravity stress is RS2. The gravity stress option is used to define stress that varies linearly with depth from the user defined ground surface elevation. This is suitable for surface or near surfaces shallow condition and the areas where topography has effect in magnitude and direction of stress. However, when the user knows the value of horizontal and vertical stress, they can use constant field stress option.

f) Computation: Once the model is properly loaded, boundary condition is provided and material properties defined then it can be computed.

g) Interpretation: After the computed the results are available for interpretation. Within the interpretation we can gets various outputs like principal stress distribution, total displacement, strength factor distribution, etc on the periphery of the excavation boundary.

4.3 Failure criteria used in RS2

4.3.1 Generalized Hoek-Brown failure criteria

In case of rock masses that do not exhibit significant anisotropy in strength and deformability, the assumption of isotropy in their properties is reasonable. The failure of jointed rock mass in response to the induced stress around it is explained by the Generalized Hoek-Brown criteria. It predicts strength envelopes that agree well with values determined from laboratory tri-axial tests of intact rock, and from observed failures in jointed rock masses.

The Generalized Hoek-Brown criterion is non-linear and relates the major and minor effective principal stresses (sigma1 and sigma3) according to the following equation(Hoek & Brown, 1980a, 1980b):

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b\frac{\sigma_3}{\sigma_{ci}} + s\right)^a \text{(Hoek \& Brown, 1980a,1980b)}$$

where:

- σ_1 and σ_3 are the axial (major) and confining (minor) effective principal stresses respectively
- σ_{ci} is the uniaxial compressive strength (UCS) of the intact rock material
- $m_b$is a reduced value (for the rock mass) of the material constant mi (for the intact rock)
- s and a are constants which depend upon the characteristics of the rock mass

The parameters used in analysis were estimated using the following equations:

- GSI was obtained using the relation given by Hoek et. al 2013 : $GSI = \frac{52 \times \frac{Jr}{Ja}}{1+\frac{Jr}{Ja}} + \frac{RQD}{2}$
- Poisson's Ratio was obtained using Ayden et. al. : $\nu_{rm} = 0.5 - 0.2 \times \frac{RMR}{RMR+0.2\times(100-RMR)}$
- Rock Mass deformation modulus was obtained using the relation given by Hoek and Diedrichs, 2006 : $E_{rm}\,(MPa) = E_i \times (0.02 + \frac{1-\frac{D}{2}}{1+e^{\frac{60+15D-GSI}{11}}})$
- The horizontal stress was obtained using the following relation: $\sigma_h = k * \sigma_v + \sigma_{tectonic}$

4.4 Methods of numerical modeling used for the project

As for the investigation of tunnel sections two different approaches were used in multi stage analysis.

- Tunnel Lining Design (Internal Pressure Reduction Method)

Tunnel Lining Design is also applied in multi stage model analysis but in this case the gradual excavation process of the considered section is represented by the decrease in the magnitude of in-situ stress the section will be able to bear. Thus, at every stage the field stress is reduced by a certain load factor till it becomes zero denoting full-face excavation. This process will also yield the required amount of deformation of tunnel wall necessary at the point of and prior to the support installation.

Numerical Modelling of Case Studies

Table 4-1 Parameters for Numerical Modeling

Parameter	Chainage 0+17.8 m	Chainage 0+58.15 m	Chainage 0+101.0 m
UCS (MPa)	78	78	78
Peak GSI	31	44	35
Residual GSI	20	24	22
m_i	28	28	28
D	0	0	0
ν	0.36	0.34	0.35
σ_h(MPa)	6.7	7.557	8.63
σ_v (MPa)	1.3	3.022	4.93
E_{rm} (MPa)	3555.9	8571.0	4644

Table 4-2 Hoek Brown Parameters

Parameter	Chainage 0+17.8 m	Chainage 0+58.15 m	Chainage 0+101.0 m
m_b (Peak)	2.382	3.789	2.748
m_b (Residual)	1.608	1.855	1.727
s (Peak)	0.0004	0.0020	0.0007

s (Residual)	0.0001	0.0002	0.0002
a (Peak)	0.520	0.509	0.516
a (Residual)	0.544	0.533	0.538

Table 4-3 Support system for the sections

Chaiange	Support System
0+17.8 m	4 m long systematic bolting (fully grouted with 25 mm diameter) with a spacing of 1.5m between bolts; fiber-reinforced sprayed concrete of 150mm thickness at crown and sides.
0+58.15 m	4 m long single bolting (fully grouted with 25 mm diameter); fiber-reinforced sprayed concrete of 80 mm thickness at crown and sides.
0+101.0 m	4 m long systematic bolting (fully grouted with 25 mm diameter) with a spacing of 1.5m between bolts; fiber-reinforced sprayed concrete of 150mm thickness at crown and sides.

Numerical Analysis for chainage 0+017.8 using Generalized-Hoek-Brown

For this section, the simulated model of tunnel was developed using input parameters as given in Table 5-1 above in RS2 software. The horizontal and vertical stresses are validated using gravity loading through simulating model before excavation. The stress sigma 1 before excavation was 6.7 MPa, and sigma 1 at crown and sidewalls of tunnel is 5.2 MPa and 8.2 MPa, respectively, after excavation. The maximum stress sigma 3 before was 1.3 MPa, and sigma 3 at crown and sidewalls of tunnel was 0.72 MPa and 1.36 MPa, respectively, after excavation.For this section, the maximum stress concentration develops at roof of the tunnel. The maximum deformation of 1.6 mm after excavation and before support installation was seen at the side walls.

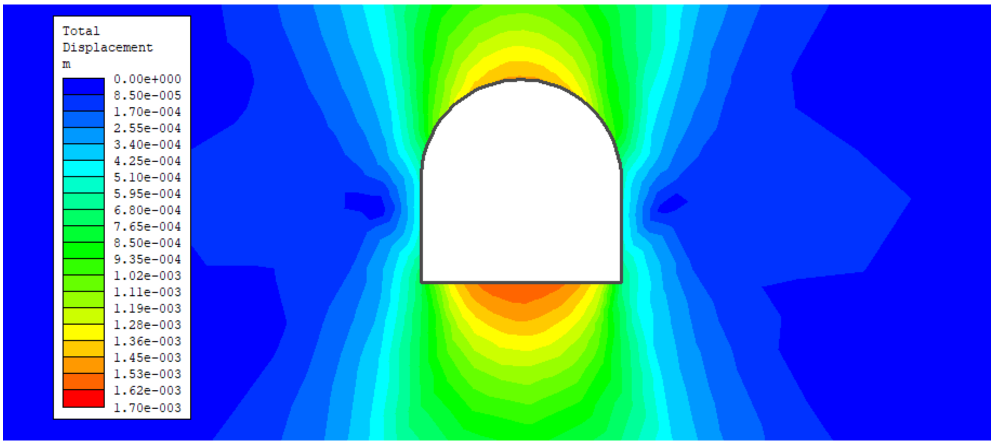


Figure 4-1 Total displacement before installation of support

The maximum deformation of 7.08 mm was seen at crown of the tunnel after installation of support.

From fig 4-5, it is found that for steelset elements, the tunnel lies within the factor of safety, but in case of shotcrete elements the tunnel fails in shear and bending moment. So, the modified support needs to be introduced and we increased the thickness of concrete in invert level and also introduced composite support & we again computed the results and found all the stresses lying within Factor of Safety as shown in **Figure 4-5**

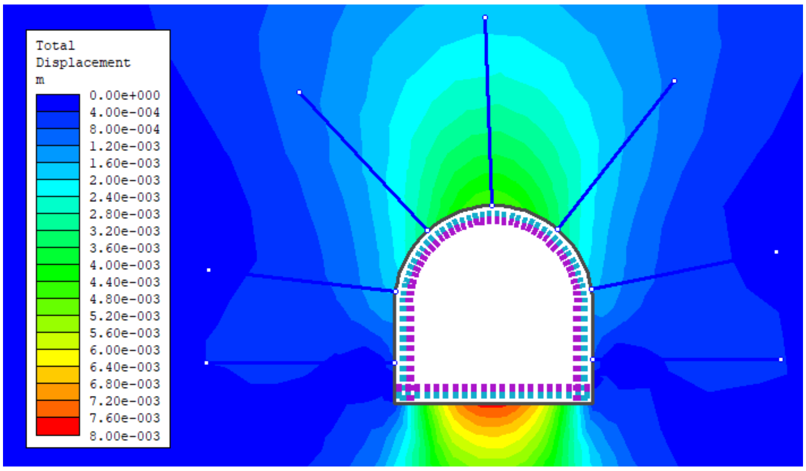


Figure 4-2 Total displacement after installation of support

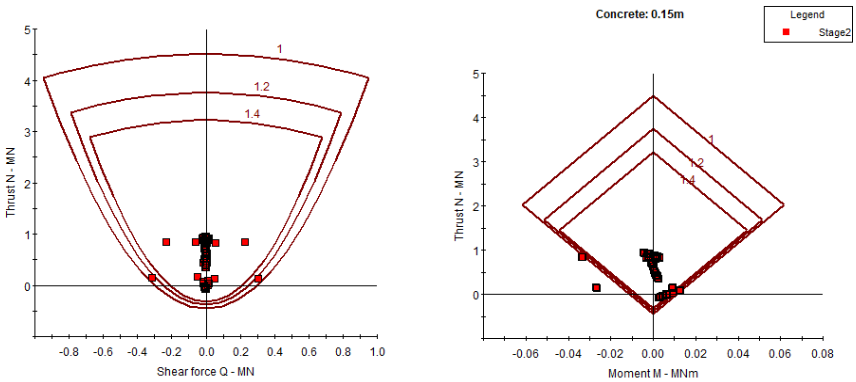


Figure 4-3 Support capacity plot of chainage 0+017.8m

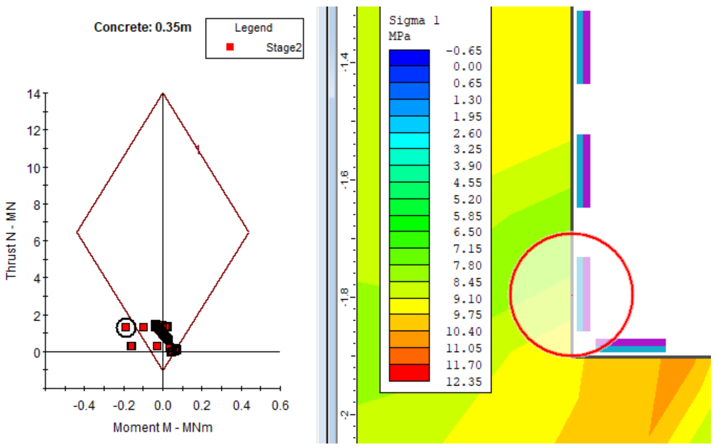


Figure 4-4 support capacity plots for chainage 0 + 17.8m

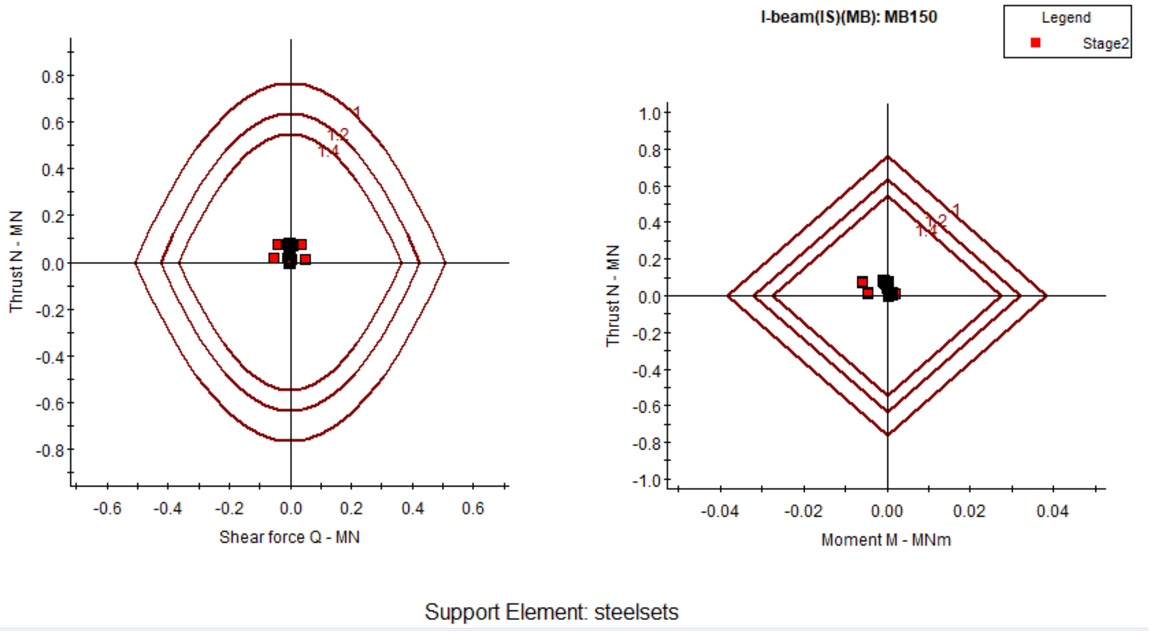


Figure 4-5 Support capacity curve of chainage 0+017.8

The results for the other two chainages are given below in **Table 4-4**. Figures can be found in Annex C.

Table 4-4 Results from numerical analysis

Chainage		0 + 58.15 m	0 + 101.00
σ_1(MPa) before excavation		7.57	8.63
σ_3 (MPa) before excavation		3.022	4.93
σ_1(MPa) after excavation	Crown	6.2	6.75
	Wall	8.4	9.5
σ_3 (MPa) after excavation	Crown	3.06	5.39
	Wall	2.5	3.9
Maximum deformation before support (mm)		0.2	0.01
Maximum deformation after supports installed (mm)		4	1.4

4.5 Analysis of section of the Headrace tunnel for squeezing case.

Squeezing is a common problem encountered during tunneling in highly weathered and weak rocks combined with considerable overburden. Squeezing is the time dependent deformation that occurs in underground excavations. Squeezing results in the reduction of cross-section of the tunnel. Regular support elements like concrete liners, steel sets rockbolts are usually ineffective in controlling squeezing ground conditions. Another issue that comes about as a result of squeezing ground is face instability. Face instability is the failure of tunnel face as excavation progresses. However, the full 3D effect of face instability is not possible to study via 2D analysis.

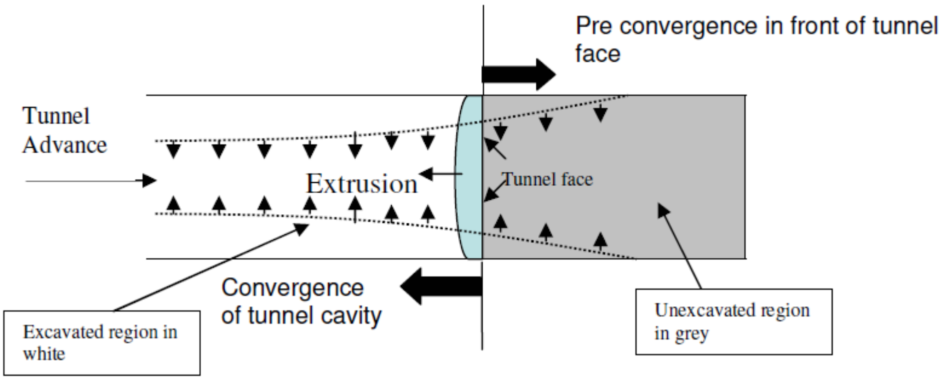


Figure 4-6Deformation during tunnel excavation(Tan, 2005)

To reinforce the rock mass, forepoling is a adopted. Forepoling is the installation of reinforcement (be it spiling rods, steel rods) over the crown of the tunnel before the face is excavated. This helps in stabilizing the rock (or soil) around the excavation and helps in reducing the face failure.

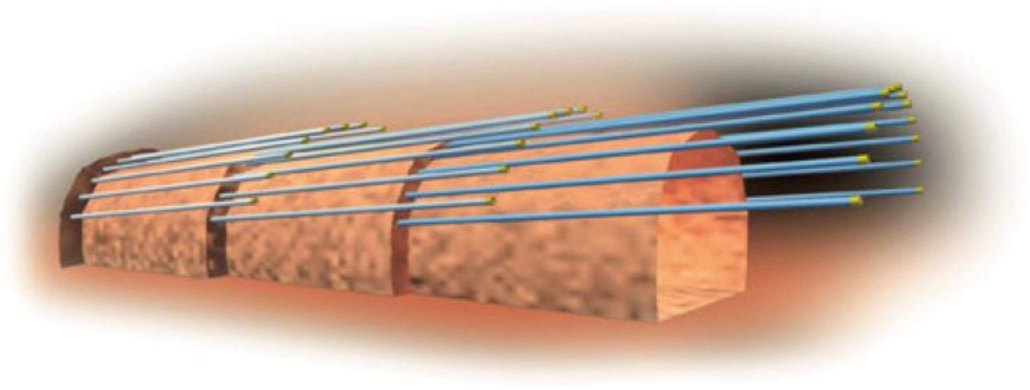


Figure 4-7 3D view of umbrella arch forepoling (Tan, 2005)

In deep tunnels in weak rocks, yielding supports are also an option. These supports have sliding joints in between and are first allowed to deform. After the sliding gaps close, then only the supports take axial force. This type of supports are employed when the width of concrete or shotcrete layers become so thick as they incur heavy finanaces. A iconic example of the use of such liners is in the Yacambú-Quibor tunnel in theNorthern Andes in Venezuela. With overburden exceeding 1000m in some sections, the tunnel underwent massive squeezing (in excess of 50% of tunnel dimension). After many trials and errors, the final support system adopted was embedding steel sets in concrete 60 cm thick. Two (2) sliding gaps each 20-30 c m were placed. This support closes in on the sliding joints and when fully closed, the joints are grouted with shotcrete to finish the final layer of the support system.

Figure 4-8 Yielding support with sliding gap present (Hoek,2002)

However, this system of yielding support hasn't been adopted in Nepal.

4.5.1 Modeling of Support Systems

Most numerical modeling programs don't offer direct support elements of forepoling and yielding supports. There are certain methods and approximations to implement such support in 2D modeling. These are explored below:

Forepoling

The problem of forepoling is ultimately a 3 dimensional one. The face stabilizing effect of the forepole umbrella is not possible to study via 2D models. Nevertheless, crude approximations of the forepole umbrella have been explored before. Most notably, the idea of representing the forepole umbrella as a region of rock mass with improved parameters that is installed above the crown of the tunnel is a viable method suggested by Hoek. The process of weighted averages is used to calculate an equivalent rock mass strength for the reinforced rock zone above the tunnel crown. The parameters are easily obtained by the use of RocLab software.

A full solution requires the use of a program such as FLAC3D but such programs are seldom used for routine tunnel design. Consequently, it is worth considering whether two dimensional models such as RS2 can provide any guidance on this complex issue(Evert Hoek, n.d.).

Yielding supports

Yielding supports (supports with sliding joint) are modeled in RS2 by defining the composite supports in terms of a beam liner first with equivalent elastic modulus and thickness. Then, the sliding gap of the support system is defined using the "strain at locking" parameter. What this basically is is that the support system will only take the axial loading if the specified strain is attained. The sliding gaps do not have any definite location in the model. Locking occurs when the total average strain along the liner is equal to the locking strain.

In our case, forepoles were used extensively in the final portion of the Headrace Tunnel. The problem statement is defined in the section below

Case: Headrace Tunnel of SMHEP (Chainage 5 + 561)

The final segment of the headrace tunnel (after the surge shaft cavern) is situated in very poor rock mass. According to the facemap of the chainage, the rock mass is heavily disintegrated highly weathered gneiss clinging to residual soil. This portion of the headrace tunnel lies in a huge old landslide**.** Also, there was medium inflow of water in the tunnel section. Support Class VI was implemented in the section. Also, since blasting was not possible, the section was excavated manually. The following is a picture from the tunnel section.

Figure 4-9 Section of Headrace tunnel (Photograph by authors)

From the photograph, we can see issues in the section. Most notably is the failure of face that has to be controlled by excavating the face in two benches. Although not clear in the photo, there is extensive use of forepoling to stabilize the face before excavation. This will be explored in the modeling portion of this chapter.

Modeling of the tunnel section

The section of the tunnel is inverted D shape with width 4.2 m and height 4.2m. The crown is of radius 2.1m. The tunnel is excavated by the core replacement method. According to Hoek, 2002., the forepole equivalent rock mass is installed over the crown of the excavation in the same stage as the core is softened to 50% of the deformation modulus. In the next stage, the support elements are installed to obtain the full profile of the tunnel. The parameters of rock mass in this section are as follows

Table 4-5 Input Properties for modeling

UCS	GSI	m_i	D	ν	E_{rm}	m_b	a	s
78	16	28	0	0.32	998.81	1.394	0.557	0.001

In this case, the steps in the tunnel roof required to install the forepoles are approximately 0.6 m deep and hence we will consider a rock beam 1 m wide and 0.6 m deep. The forepoles have an outer diameter of 114 mm and an inner diameter of 100 mm and are spaced at 0.5 m.

The calculation of the improved rock mass as a representation of the forepole umbrella (as suggested by Hoek, 2002) is detailed in the table below:

Table 4-6 Calculation for reinforced rock mass as result of forepoling

Component	Area (m^2)	Strength (MPa)	Product
Rock	0.6	6.189	3.7134
Steel pipe	0.005	200	1
Grout	0.015	30	0.45
Sum	0.62		5.1634

The compressive strength of the improved rock mass that represents the forepole is 5.1634/0.62 = 8.32 MPa

The Hoek-Brown parameters for this rock mass can easily be obtained from the software RocLab by Rocscience. The properties are tabulated below:

Table 4-7 Equivalent Hoek-Brown parameters for the improved rock mass

UCS	GSI	m_i	D	ν	E_{rm}	m_b	a	s
78	26	27.3	0	0.32	1776.67	1.934	0.529	0.003

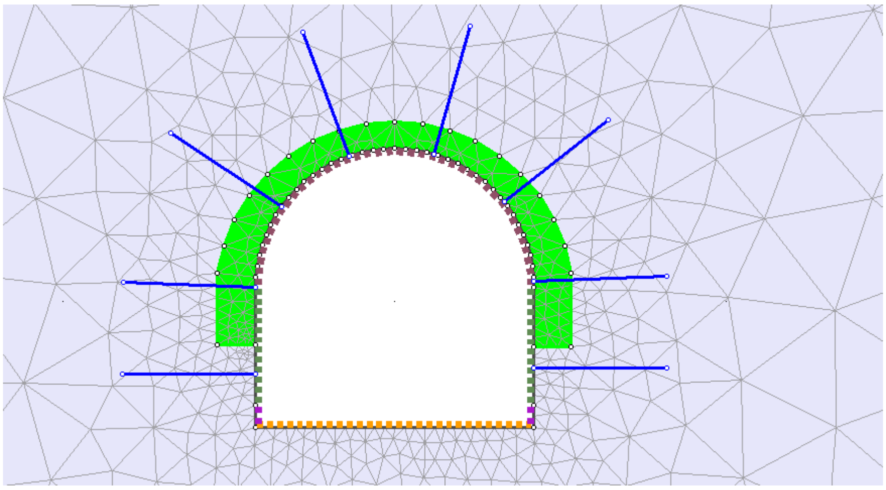


Figure 4-10RS2 model of tunnel section with forepole umbrella added

The first step was to find out the deformation of the tunnel without any support installed. This is necessary to proceed with the process of core replacement. The maximum displacement of 40 mm was observed in the invert. This is nearly 1% strain.

Next the model was carried out installing the forepole umbrella in conjunction with the support liners. The roof was supported by installing a shotcrete layer of 200 mm thickness with steel sets while on the walls, concrete of thickness 200 mm on the upper section increasing to thickness 450 mm on lower section to invert along with steel set was applied. The application of forepole drastically reduced the amount of support required for the stabilization of the tunnel and also significantly reduced the deformation of the boundary. However, the invert of the tunnel required very thick layers of concrete (about 800mm). This will significantly decrease the tunnel section in addition to incurring heavy financial cost to the project and is not recommended.. The deformations are tabulated below in table :

An option to limit the thickness in the invert would be delayed support installation which allows the invert to further deform and at a later time, the converged section can be excavated prior to the lining (Karki et al., 2020)

Table 4-8 Results of modeling

Displacement		Remark
Crown	Wall	
35 mm	18 mm	Unsupported
16 mm	2 mm	Supports + Forepole

The outputs are as shown in the figures below:

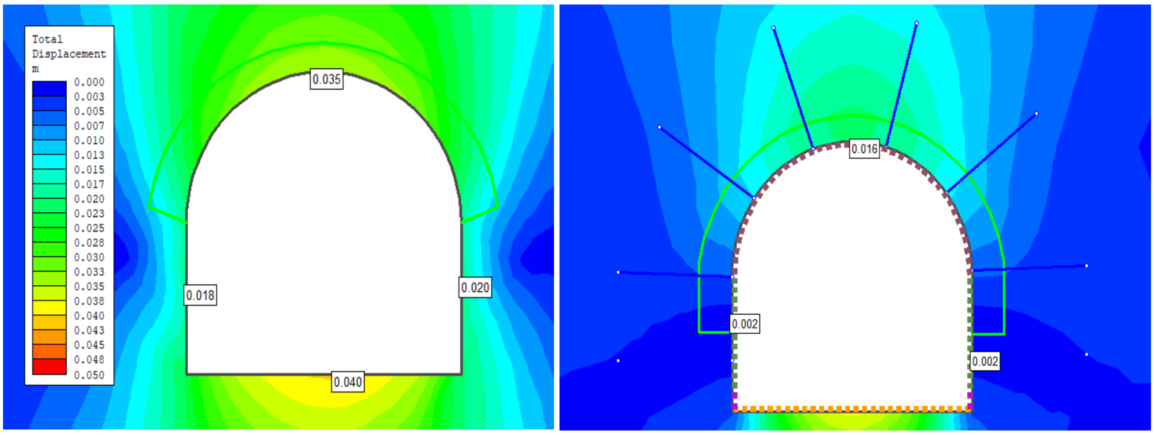


Figure 4-11 Deformation of the tunnels (left) without support (right) with forepole umbrella and support

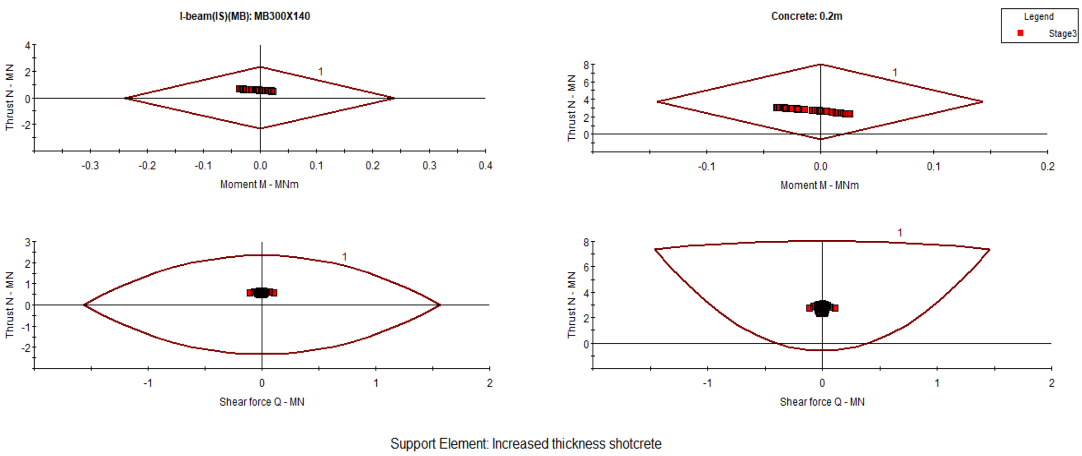


Figure 4-12 Support Capacity Plot for roof liner

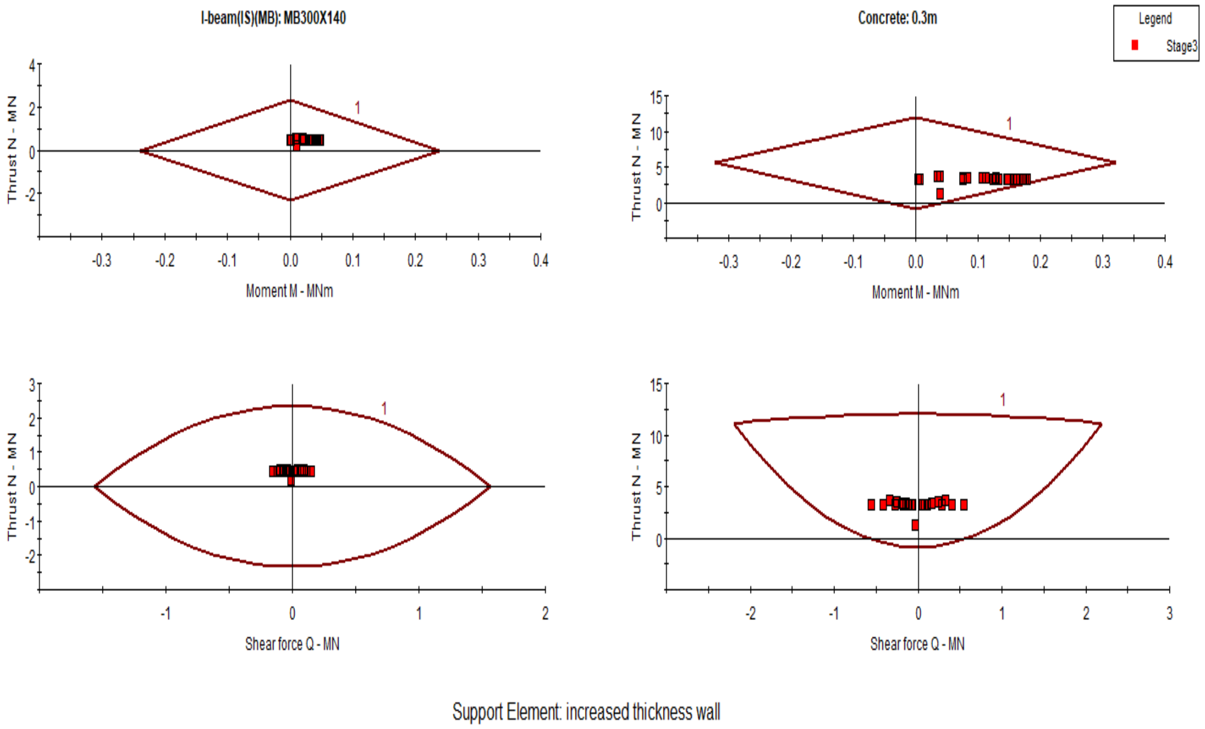


Figure 4-13 Support Capacity Plot for wall liner

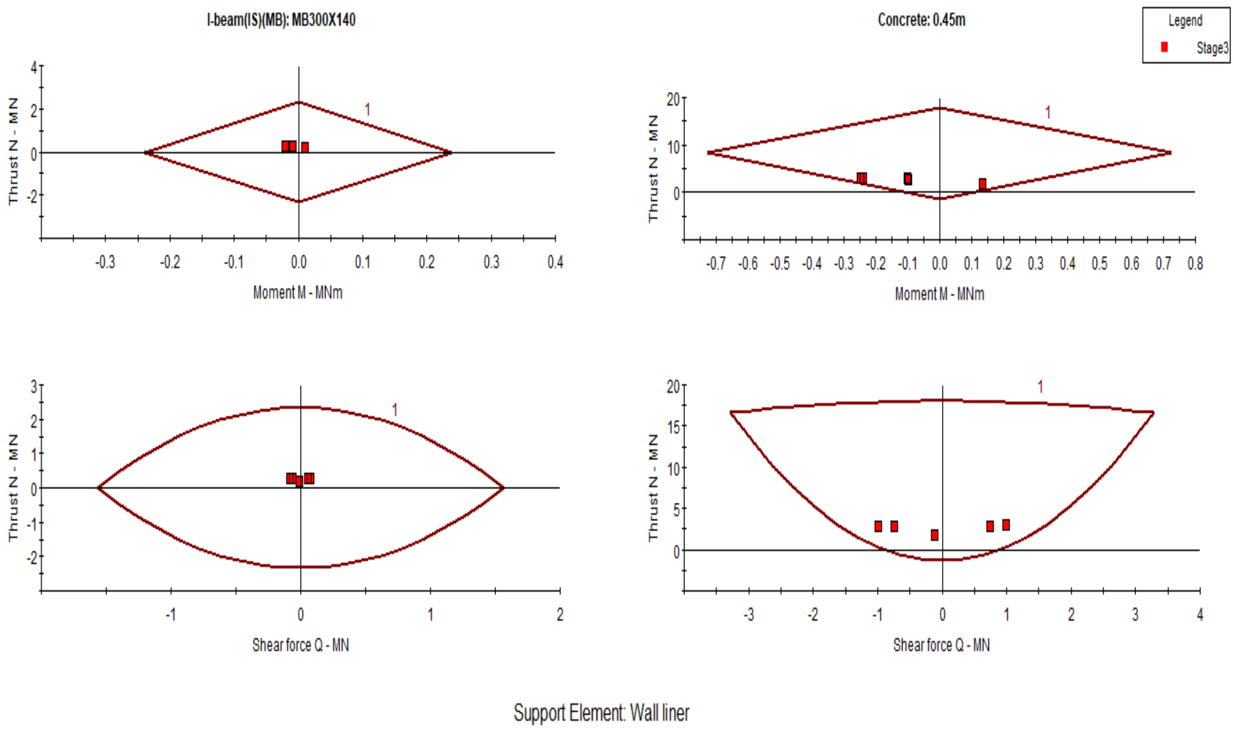


Figure 4-14 Support Capacity Plot for 450mm concrete

4.6 Analysis of the Settling Basin caverns

4.6.1 Description of the Settling Basin Cavern of SMHEP

The settling basin of Super Madi Hydroelectric Project lies entirely underground. The settling basin starts at around 50 m from the inlet portal from the intake. There are two settling basins sitting parallel to each other. One cavern is excavated for each basin. The approach tunnels to the settling basin are both 4.2m x 4.2 m inverted D-shaped. The adit tunnel for both the settling basins is at the end portion of the basin at a level 1348 masl. The settling basin also features an inspection deck at the same level. The designed normal water level is 1343.3 masl but it is designed for 10 yrs flood 1347.47 masl.

The invert level of the approach tunnel where it connects to the settling basin is 1336.57 masl. The inlet transition zone is of 28 m length with a slope of 1:5.693. It ends at the RL of 1332.00 masl where the settling zone begins. The settling zone is of 124 m length with a longitudinal slope of 1:50. The flushing system consists of a 4.2m x4.2m inverted D-shaped tunnel for both the settling basins.

The water from the settling basin exits into the headrace tunnel. The water level in the headrace tunnel is 1340.70 masl.

The section at the start of the settling zone is of height 20.66 m. The crown is arched and situated at 1352.65 masl. The walls and bottom of the basin are lined with C25 Concrete. 4 m long, 25 mm diameter rockbolts are given as support along the wall and crown at 1.5m c/c. The crown is also provided with 150 mm steel fibre reinforced shotcrete. The width of one basin is 13.10 m.

The maximum overburden over the left settling basin is roughly 210 m and over the right settling basin is 180 m.

According to the feasibility study done by Himal Hydro in 2009, the major rock here is banded gneiss along with occasional very thin bands of schist. 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. Feasibility study estimates that the rock mass has Q value of 10-14.

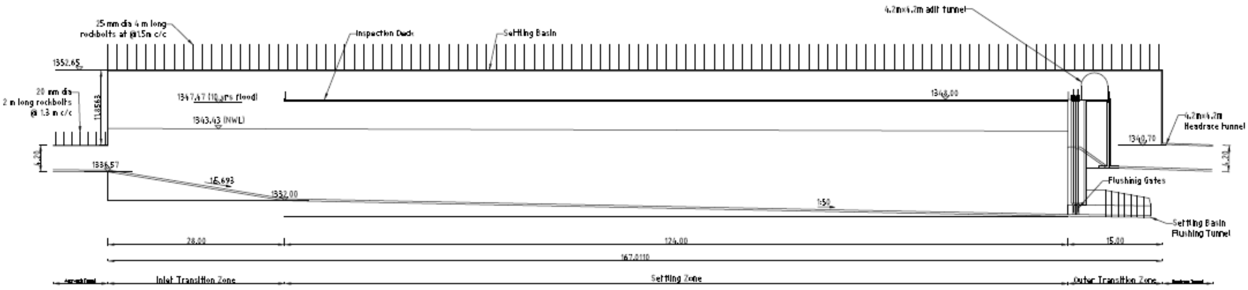


Figure 4-15Longitudinal section of settling basin

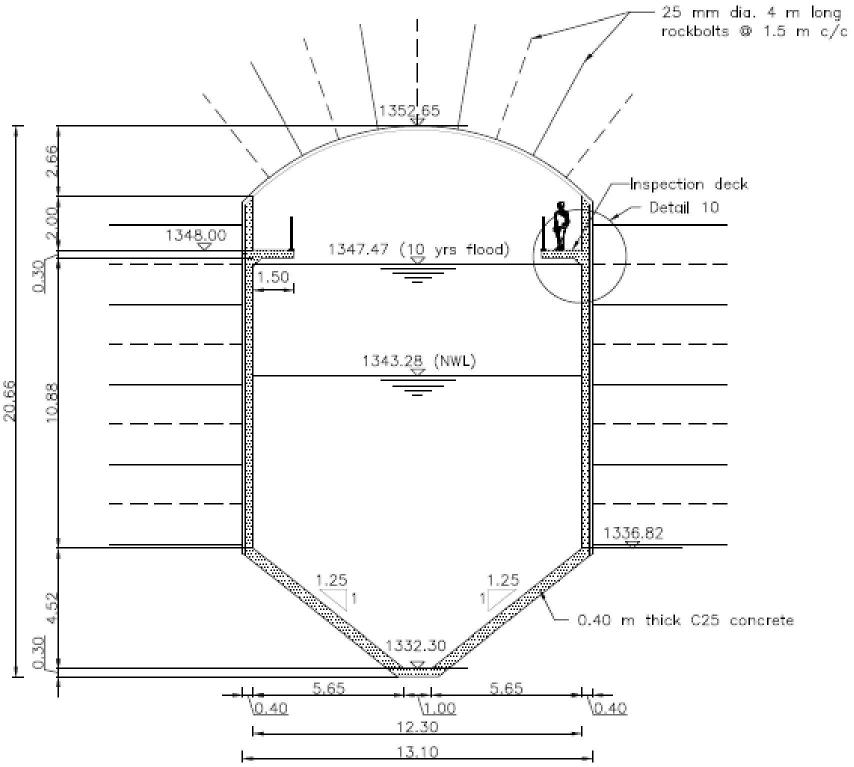


Figure 4-16 Section of Settling Basin at start of settling zone

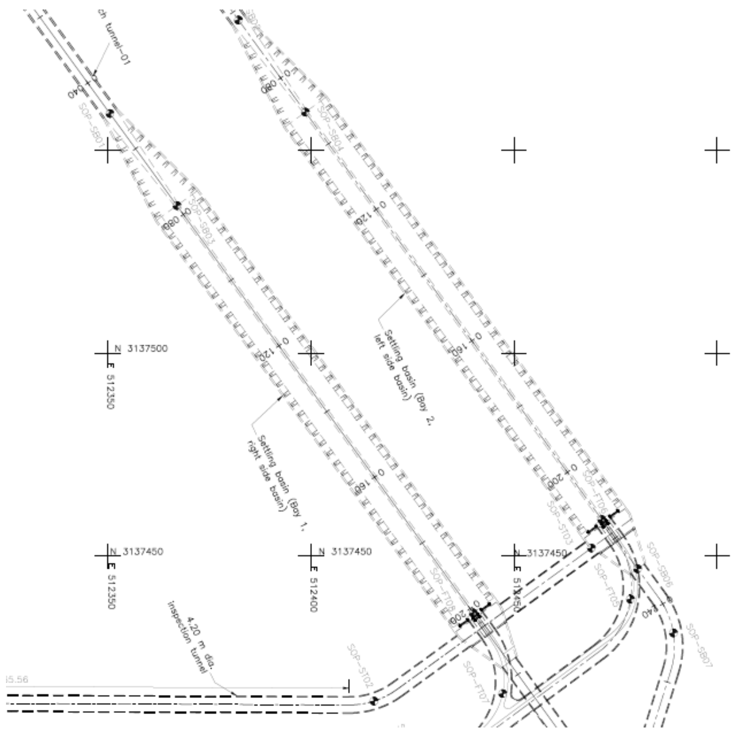


Figure 4-17 Plan of Settling basin

Figure 4-18 Settling basin cavern (Photo taken by author)

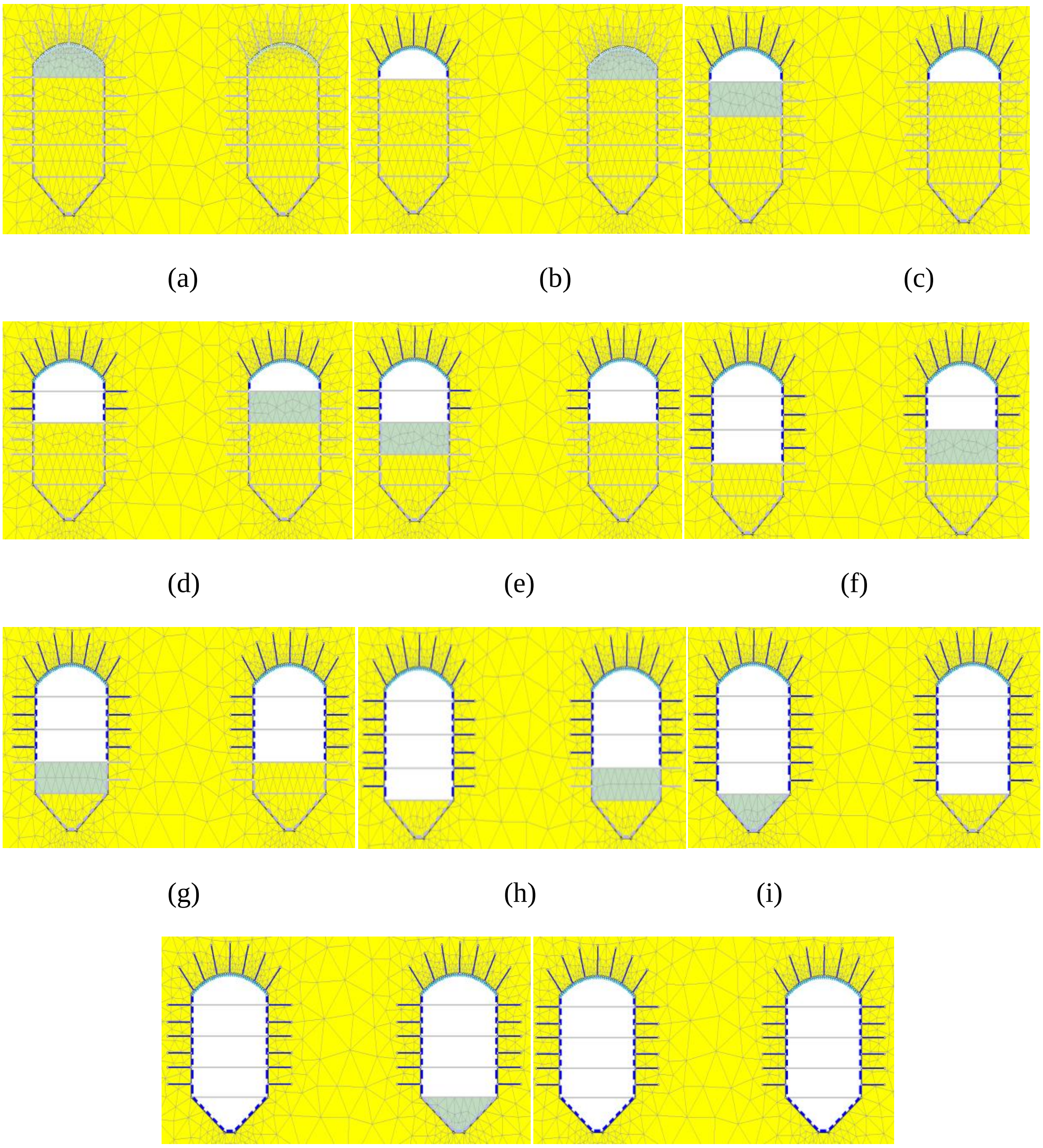
4.6.2 Numerical Analysis of Settling Basin Cavern

The 2-D numerical analysis of the sections of settling basin cavern is done using RS2 software from Rocscience. It is a two dimensional elastic-plastic finite element program for estimating the stress and displacement around the underground openings.

The analysis done is plane-strain. Plain strain analysis is where two principle in-situ stressesare in the plane of excavation whereas the third principle stress 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 (Gautam, n.d.). For our study, 3 sections have been taken representing three different rock mass qualities.

4.6.3 Process of Modeling:

Excavation of the twin caverns were simulated by dividing the caverns into subdivisions of benches of height 4m. During construction, there is always a time gap between excavation and installation of support. During this time, the rock mass relaxes and some deformation always occurs. This has to be incorporated in the numerical model as well. In our model, the deformation of the excavation boundary was simulated by replacing the core with a material whose deformation modulus is 30% of the actual rock mass and initial loading is 'none' in the following stage. This is a method suggested by Usmani et al., 2015. In the stage after relaxation, the material was excavated and supports were installed. The process is as explained in the figures below:



(a) (b) (c)

(d) (e) (f)

(g) (h) (i)

(j) (k)

Figure 4-19 Sequence of excavation (a to k) of the twin caverns in RS2

The excavation steps are:

- Stage 1 : Allow relaxation in heading A1 by replacing the core with material having 30% deformation modulus and no initial loading
- Stage 2 : Fully excavate heading A1 and install support, : Allow relaxation in heading A2 by replacing the core with material having 30% deformation modulus and no initial loading

The above mentioned steps are repeated for the remaining benches (B1 to B8).

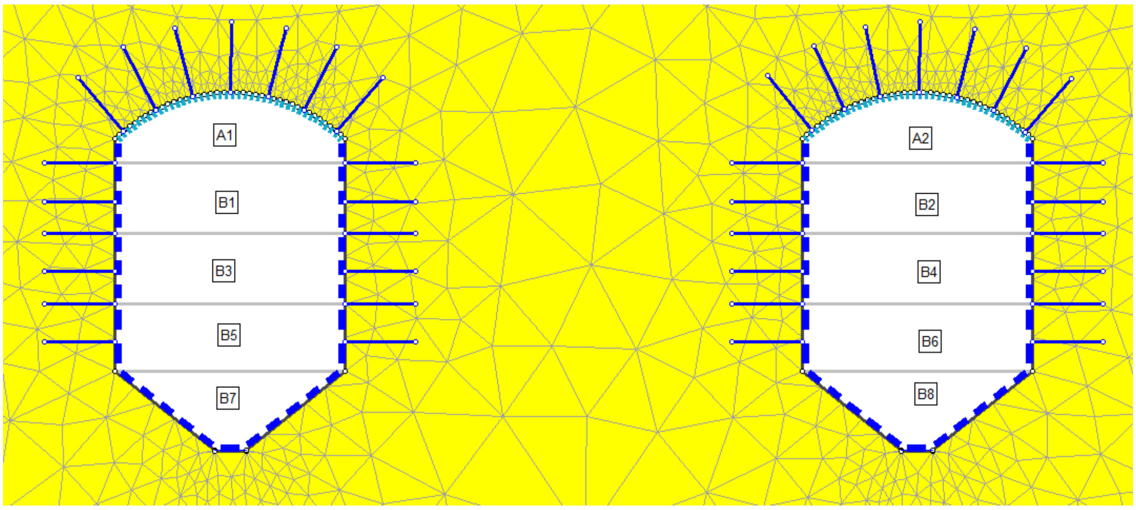


Figure 4-20 Final stage of the model with all supports installed

According to the data we were provided, the supports used in the settling basin caverns were the following:

- 25 mm diameter rock bolts at c/c spacing of 1.5m in walls and roof
- 150 mm thick shotcrete in the roof
- 100 mm thick shotcrete in the walls
- 300 mm thick C25 concrete in the wall
- 400 mm thick C25 concrete in the sloping part

The supports in RS2 are basically of 2 types: Bolts and Liners. The bolts were modeled as "fully-bonded" with the following parameters: diameter = 25mm, bolt modulus 200GPa and tensile capacity of 0.1 MN. The shotcrete and concrete supports were defined as reinforced concrete liner support. RS2 formulates these as beam elements. For concrete, the properties such as thickness and strength are entered. These beam elements are formulated as Timoshenko beam elements. The liners are treated as elastic in nature and their support capacity plots are studied to determine whether they lie within the Factor of Safety (FOS) envelopes. The following are the results of modeling using liners.

The input parameters for analysis of the settling basin cavern are tabulated below:

Table 4-9 Input parameters for numerical model of 3 sections of settling basin cavern of SMHEP

Parameter	Chainage 0+82.9 m	Chainage 0+104.55m	Chainage 0+190.5 m
UCS (MPa)	78	78	78
Peak GSI	55	47	30
Residual GSI	28	25	30
m_i	28	28	28
D	0	0	0
ν	0.34	0.34	0.36
σ_v (MPa)	4.77	5.146	5.56
σ_h (MPa)	8.48	8.646	9.114
E_{rm} (MPa)	16719	10431	3332

Table 4-10 Hoek brown Parameters for the 3 sections of settling basin cavern of SMHEP

Parameter	Chainage 0+82.9 m	Chainage 0+104.55m	Chainage 0+190.5 m
m_b (Peak)	5.613	2.244	2.298
m_b(Residual)	2.065	1.932	2.298
s (Peak)	0.067	0.0002	0.0004
s (Residual)	0.003	4.54e-5	0.0004
a (Peak)	0.504	0.507	0.522
a (Residual)	0.526	0.531	0.522

4.6.4 Results of Modelling

Chainage 0 + 82.90 m:

The section at chainage 82.90 had a Q value of 1.250. Initially, the modeling was done without support to determine deformation. The rock mass was modeled as a plastic material that follows the Generalized Hoek-Brown failure criteria. The deformation of the boundaries after complete excavation of both the bays are shown below :

There are a total of 376 yielded finite elements.

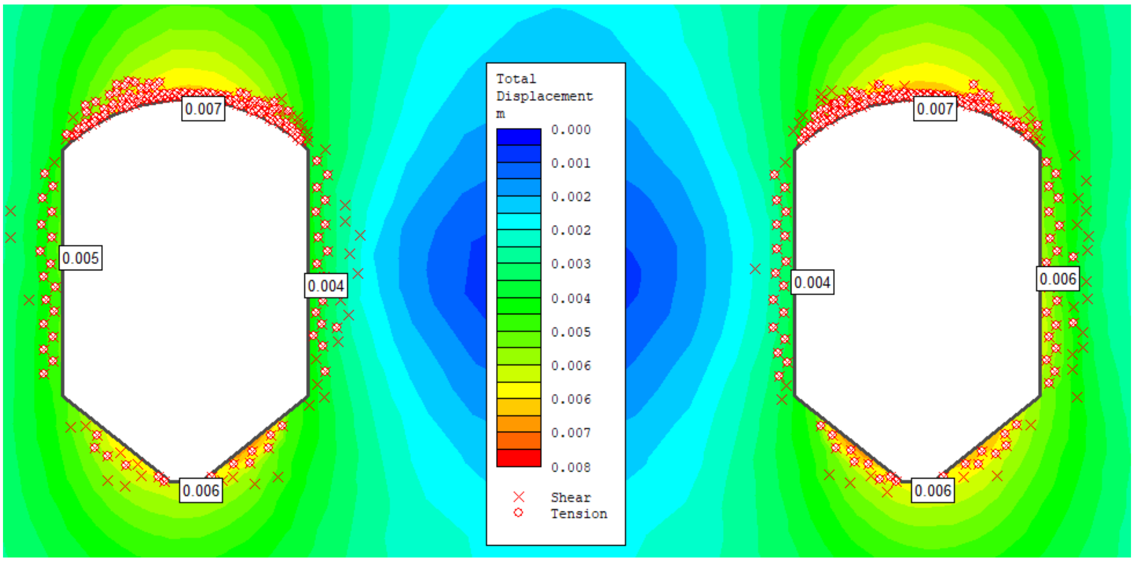


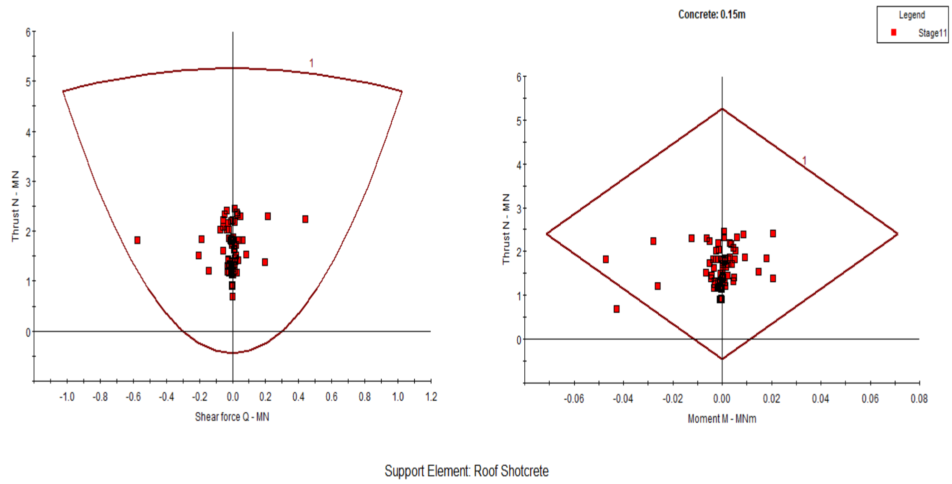
Figure 4-21 Deformation values around the cavern boundaries along with yielded elements for chainage 0 + 82.90

The results of modeling after installing the defined supports are given below. As stated above, the liners are modeled as elastic beam elements.

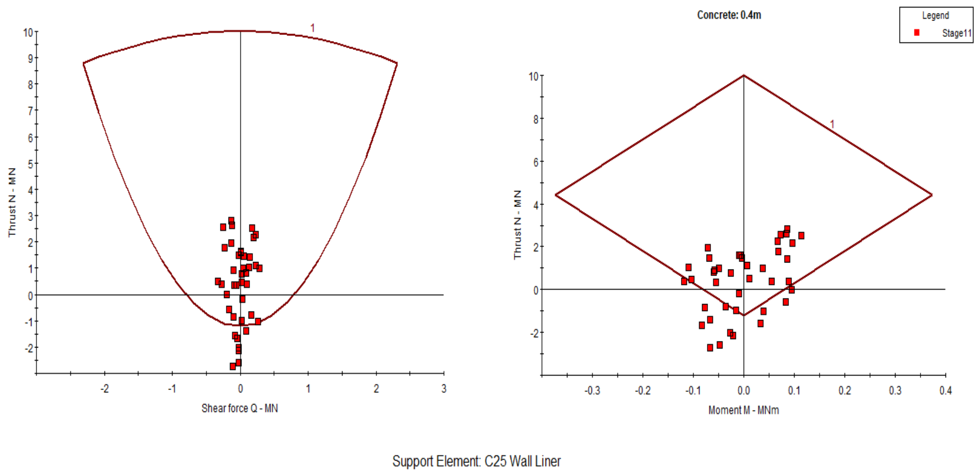
Table 4-11 Deformation of cavern with defined support installed

Location	Left Bay	Right Bay
Roof	7 mm	6 mm
Wall	5 mm	6 mm
Base	6 mm	6 mm

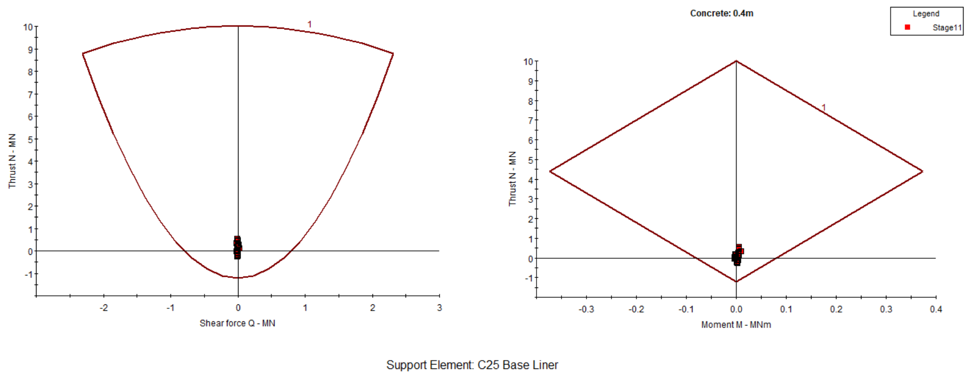
The support capacity plots will determine where the support liners are adequate or not. The points lying outside the graphs are below FOS 1. The Support Capacity Plots of the different liner elements are shown in the figures below.



(a)



(b)



(c)

Figure 4-22 Support Capacity Plots for (a)Roof Shotcrete (b)Wall Shotcrete (c)Base Concrete (d) Wall Concrete

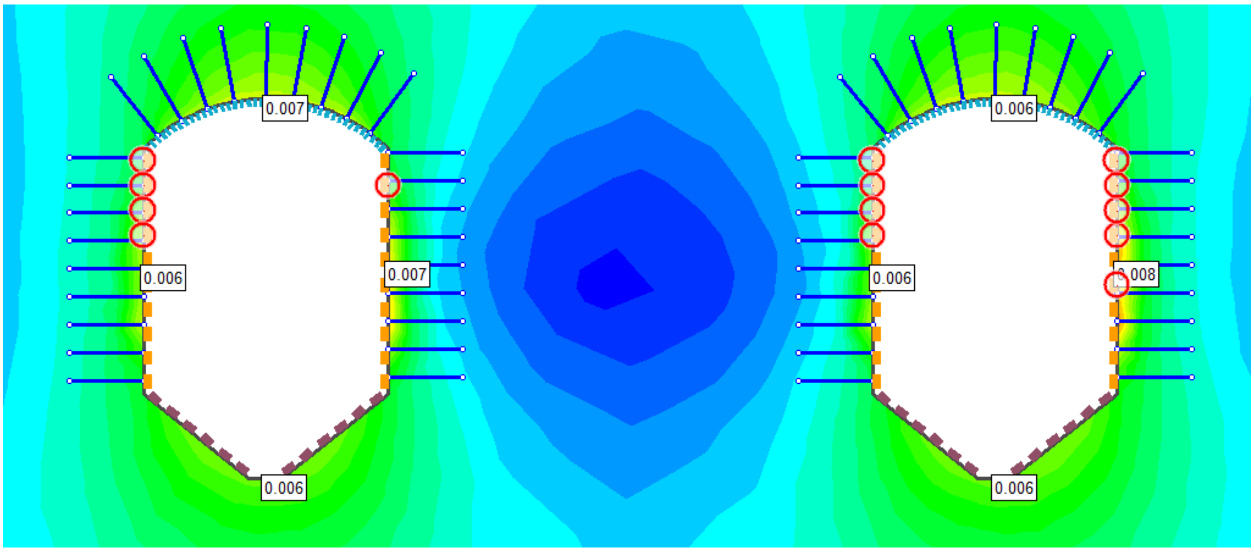


Figure 4-23 Liner elements which lie outside the FOS envelope

These liner elements which are outside the safety envelope need to be re-designed.

Chainage 0 + 104.55 m:

Similar to the previous section, the following results were obtained for the section at 0 + 104.55 m

There are a total of 467 yielded finite elements.

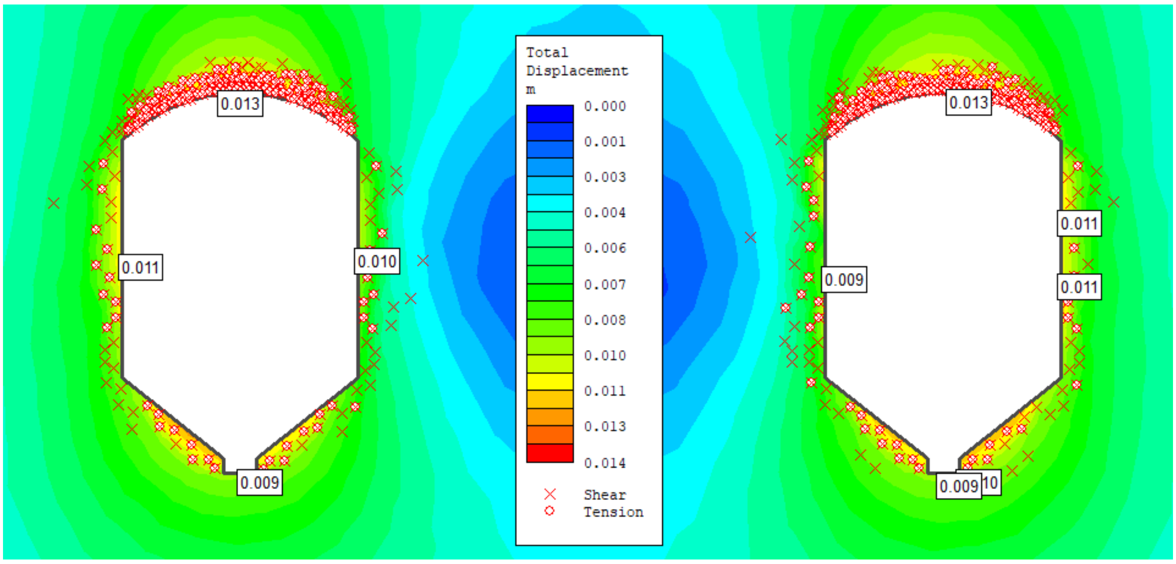


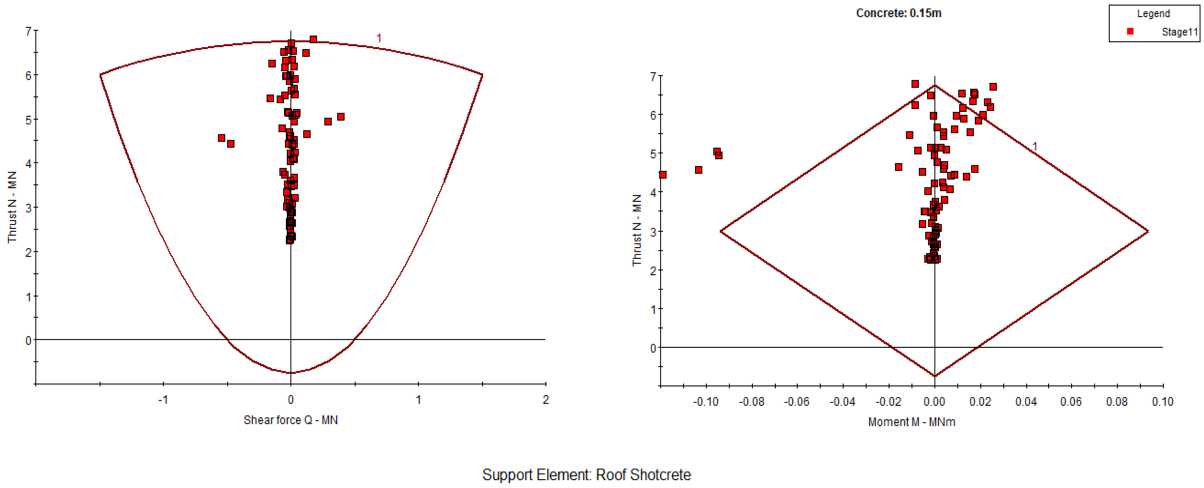
Figure 4-24 Deformation values around the cavern boundaries along with yielded elements for chainage 0 + 104.55 m

The results of after installing the defined supports are given below.

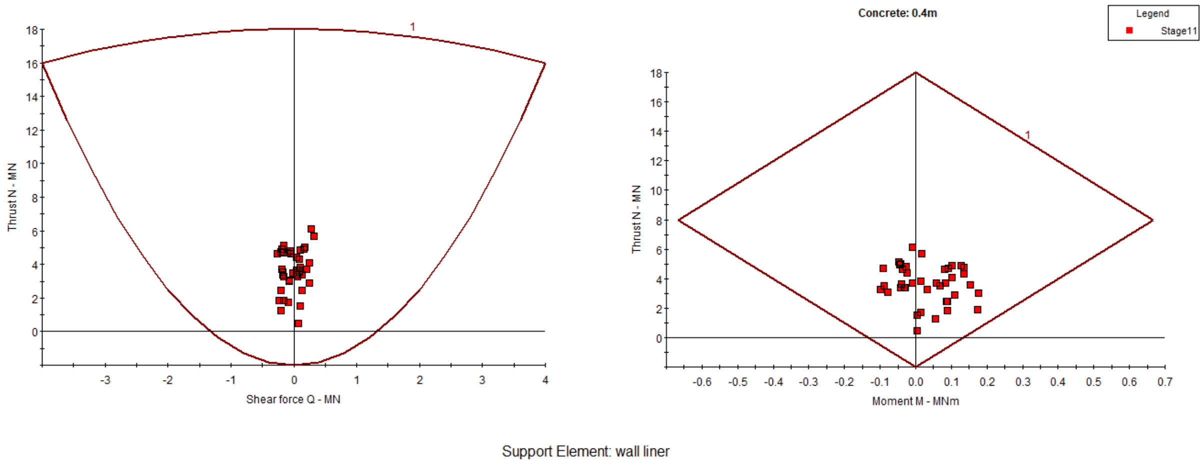
Table 4-12 Deformation of cavern with defined support installed

Location	Left Bay	Right Bay
Roof	10 mm	10 mm
Wall	11 mm	10 mm
Base	9 mm	9 mm

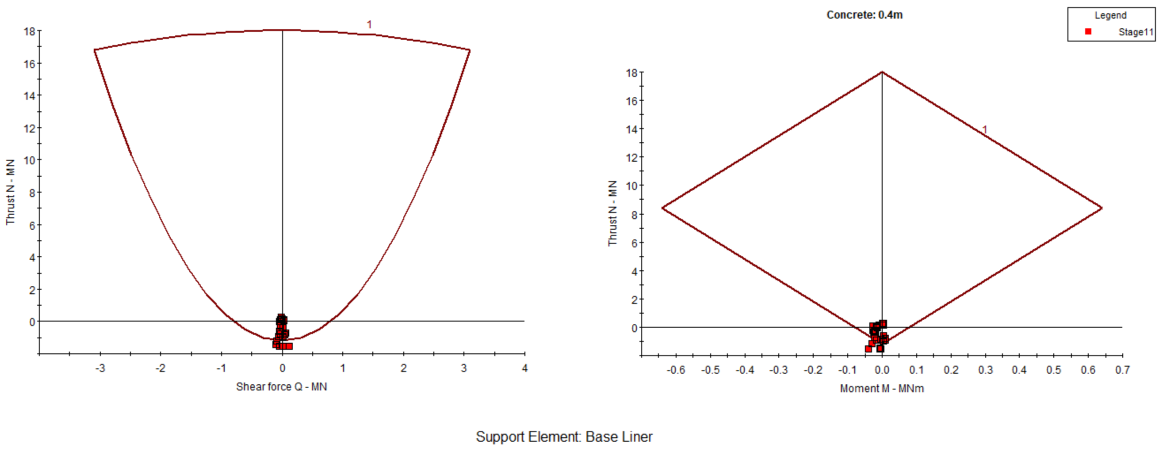
The support capacity plots for the defined supports are:



(a)

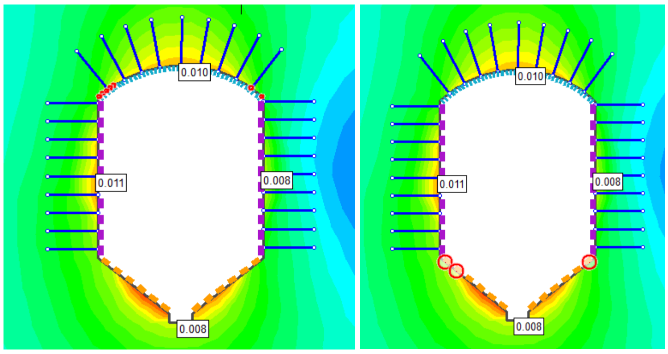


(b)



(c)

Figure 4-25 Support Capacity Plots for (a)Roof Shotcrete (b)Wall Concrete (c)Base Concrete



(a) (b)

Figure 4-26 Liner elements which lie outside the FOS envelope for the left bay (a)Roof Shotcrete (b)Base Concrete

These liner elements which are outside the safety envelope need to be re-designed. Modification of these supports is necessary and discussed below.

Chainage 0 + 190.5 m:

The results of modeling for this section are as shown below

There are a total of 461 yielded finite elements.

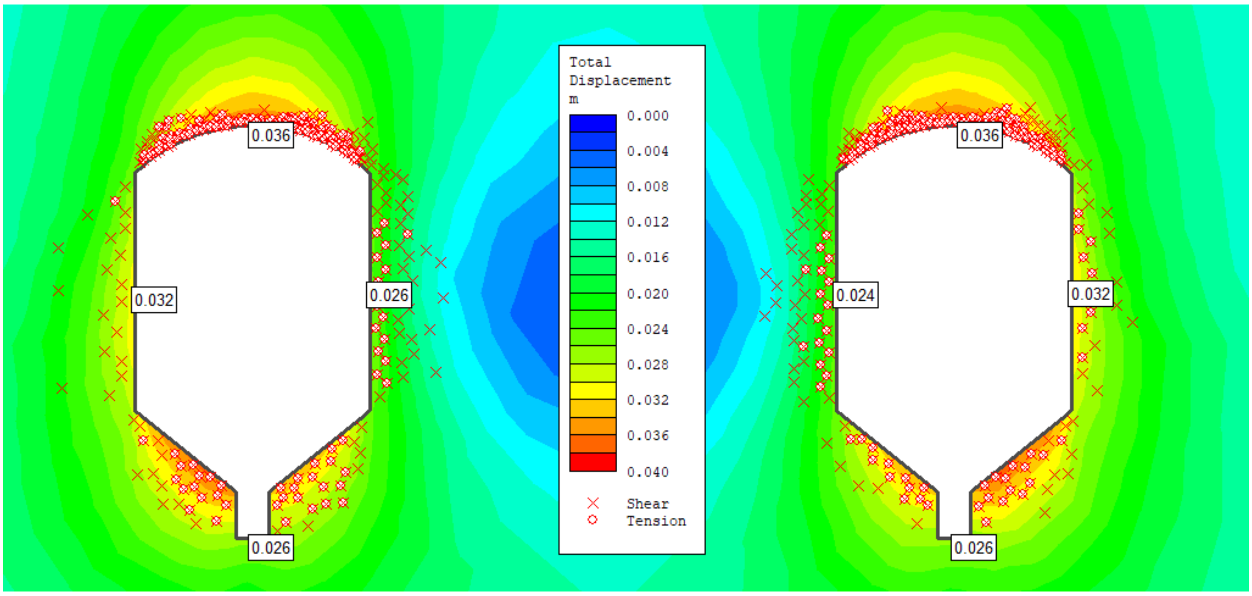


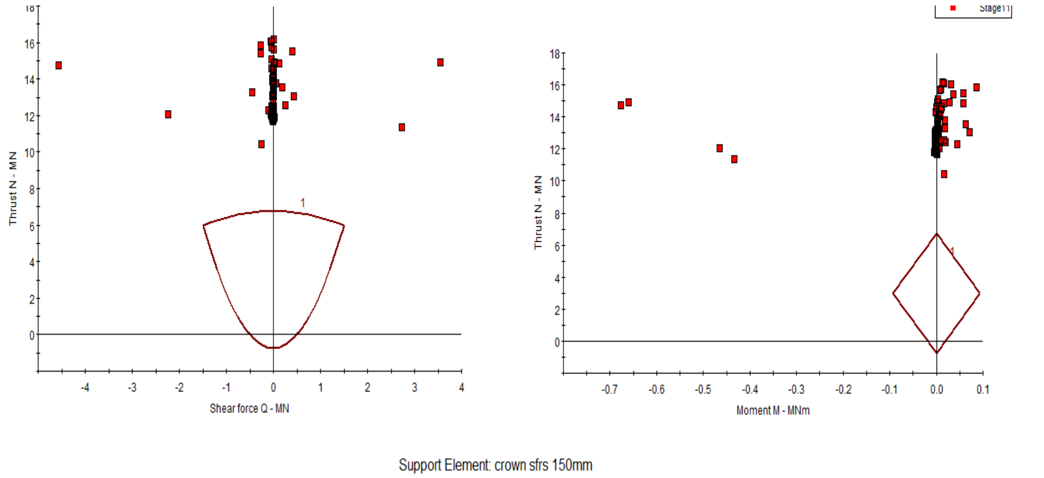
Figure 4-27 Deformation values around the cavern boundaries along with yielded elements for chainage 0 + 190.55 m

After installing the defined supports, the results are given below

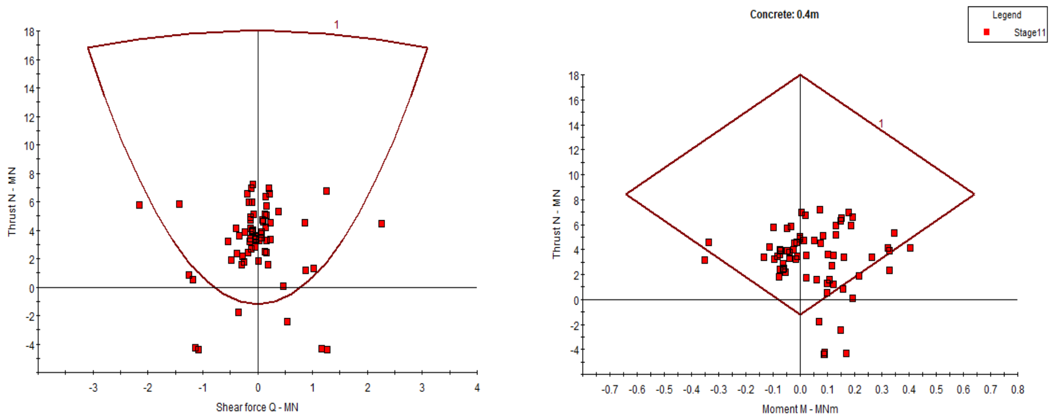
Table 4-13 Deformation of cavern with defined support installed

Location	Left Bay	Right Bay
Roof	32 mm	32 mm
Wall	30 mm	32 mm
Base	28 mm	28 mm

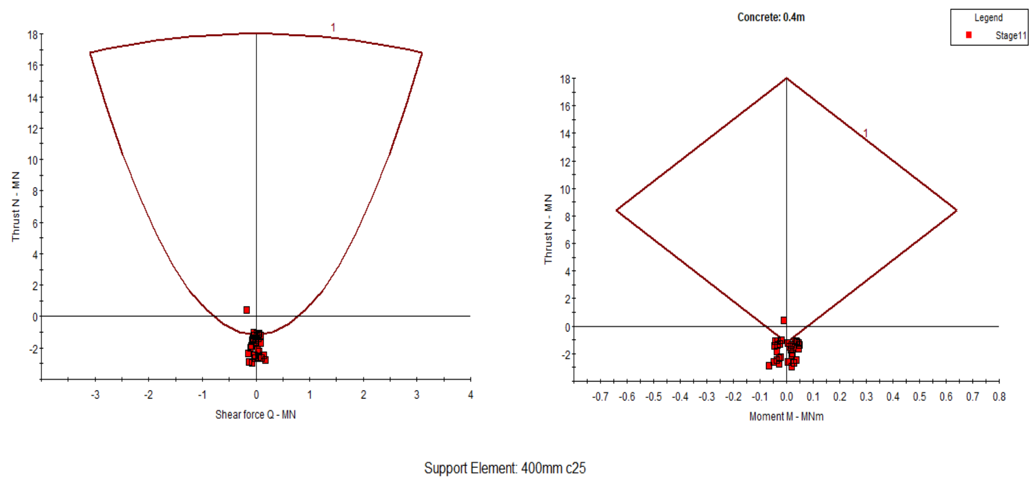
The Support Capacity Plots of the different liner elements are shown in the figures below.



(a)

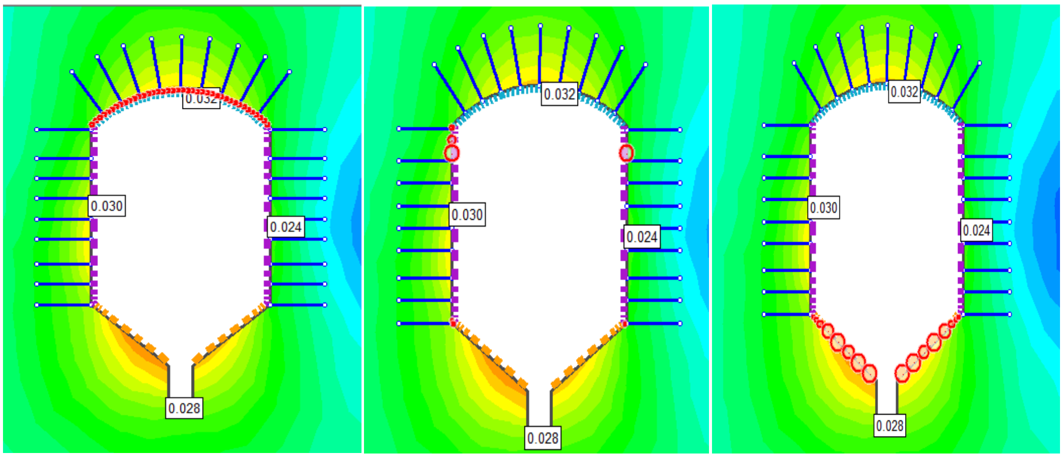


(b)



(c)

Figure 4-28 Support Capacity Plots for (a)Roof Shotcrete (b)Wall Concrete (c)Base Concrete



(a) (b) (c)

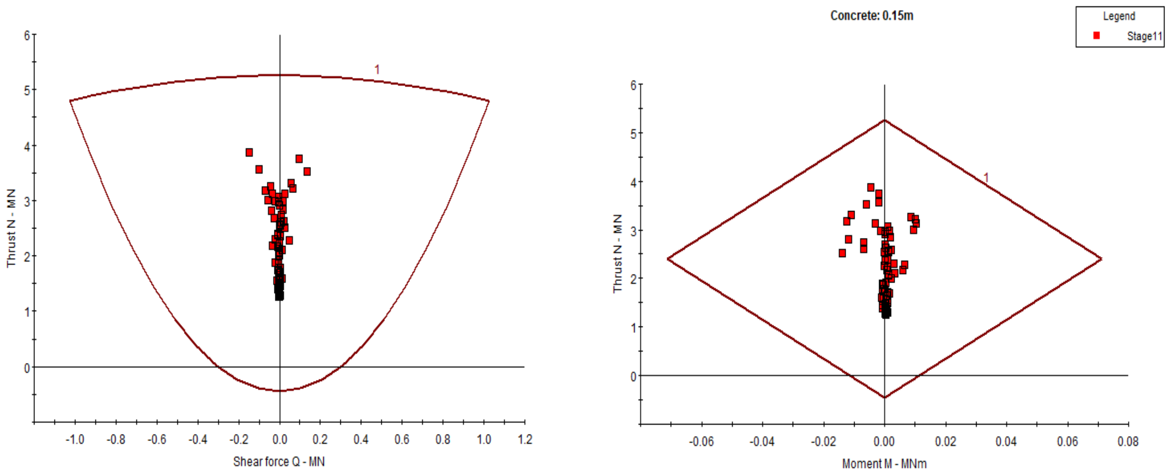
Figure 4-29 Liner elements which lie outside the FOS envelope for the left bay (a)Roof Shotcrete (b)wall concrete (c)Base Concrete

4.7 Modification of Supports

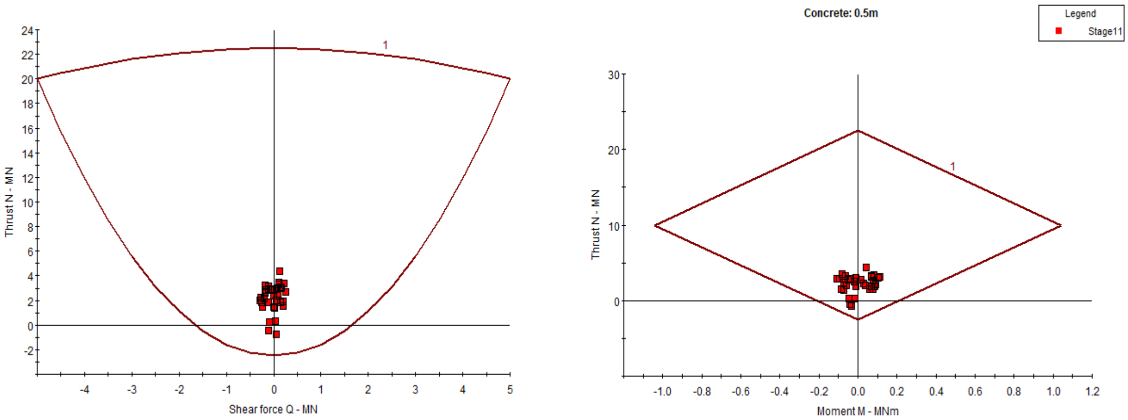
Yielding of supports is usually due to inadequacy or through high concentration of stress around the corners. Usually it's the high concentration of stress that causes liners around the corner to fail. This concentration of stress can be decreased to some level by smoothing the corner edges.

Chainage 0 + 82.90 m :

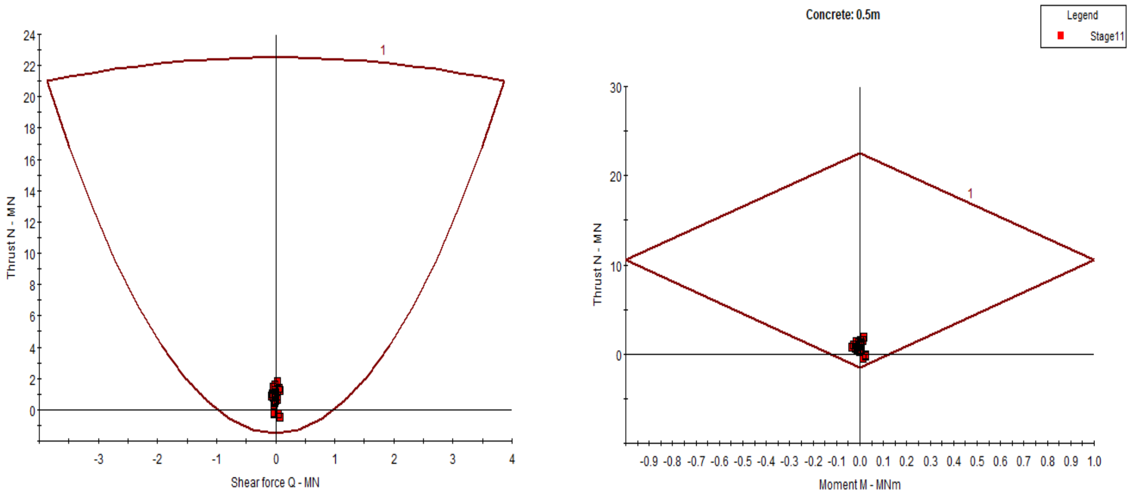
The corner edges have been curved and increasing the wall liner and the base liner by 10cm is enough to bring all the elements within the FOS envelop. The plots are as shown below:



(a)



(b)



(c)

Figure 4-30 Support Capacity Plots for the increased Supports (a) Roof Shotcrete (b) Wall concrete (c) Base concrete

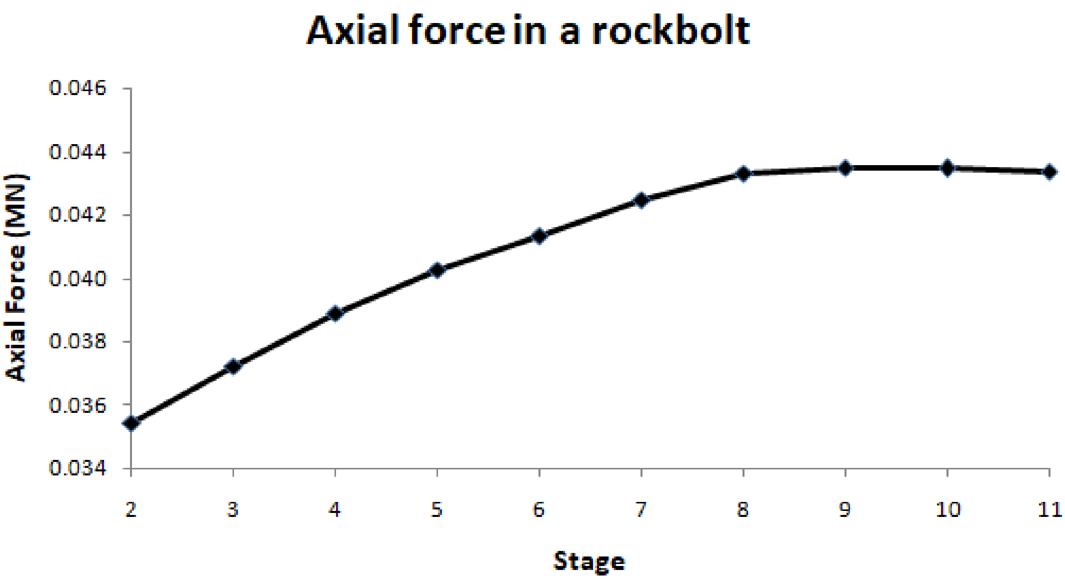


Figure 4-31 Axial force developed in a rockbolt at the crown for chainage 0 + 89.2 m

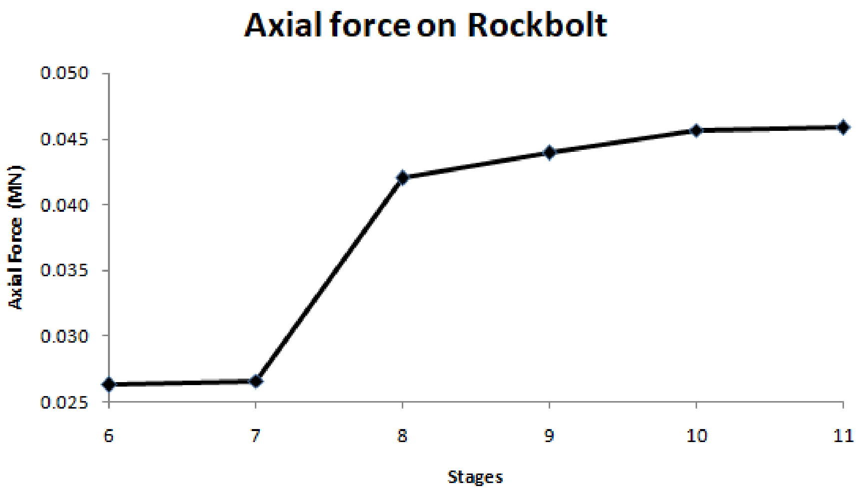


Figure 4-32 Axial Force developed in a rockbolt in the wall for chainage 0 + 89.2 m

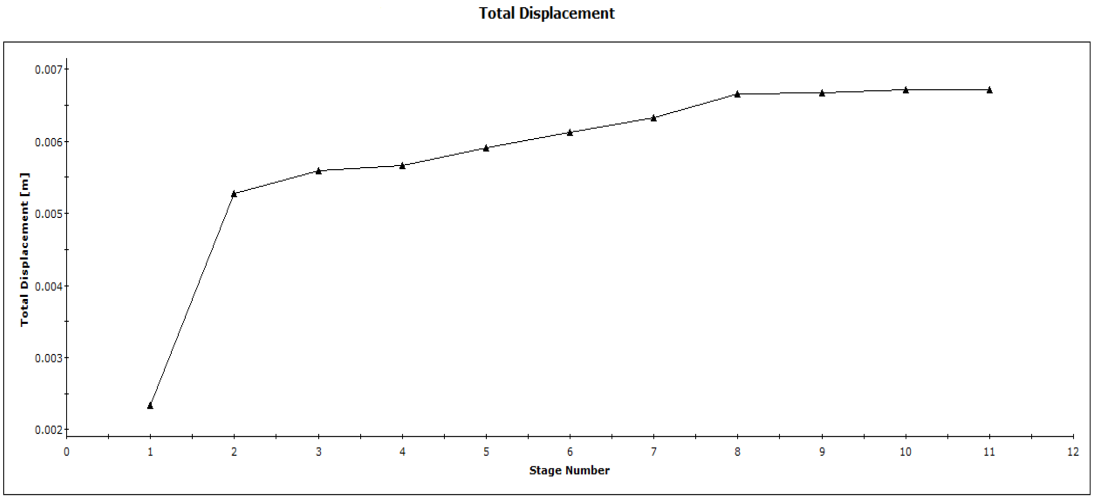


Figure 4-33 Displacement of the crown vs stages

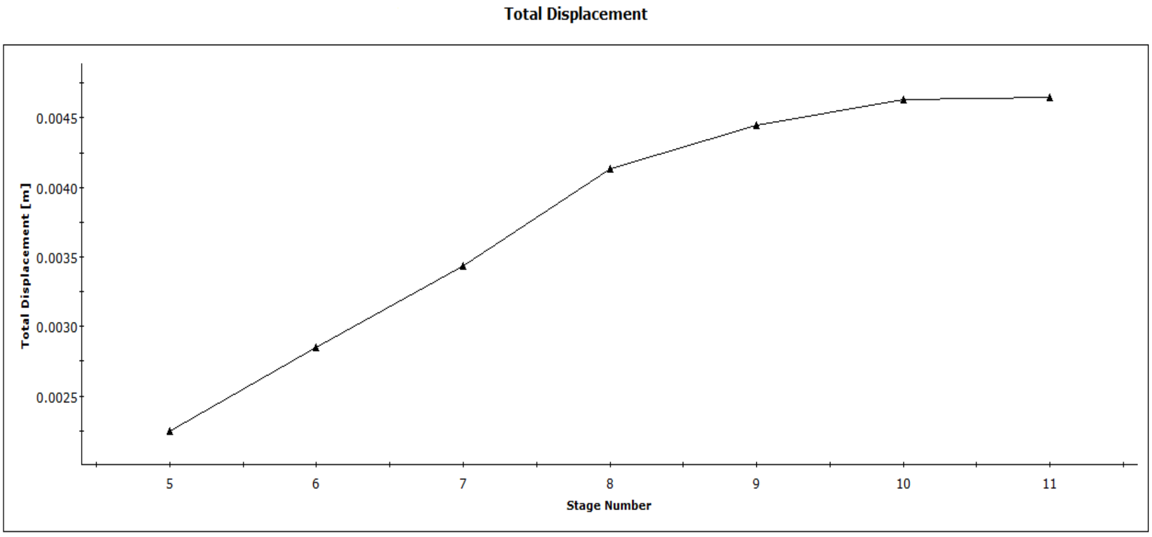


Figure 4-34 Displacement of the Walls vs Stage

Figure 4-31 shows the graph of axial force developed in a rockbolt at the crown. At the stage of installation (stage 1), 0.0354 MN Axial force is developed and gradually, more force is exerted as excavation proceeds. The bolt sustains maximum axial force of 0.0433MN in stage 8 and it continues till the final stage. At the wall, during stage 6, 0.026 MN is exerted while the final value of axial force is 0.045 MN as the cavern is totally excavated. **Figure 4-33** and **Figure 4-34** show the displacement of crown and wall respectively as the cavern is excavated in benches. The crown doesn't displace as much when subsequent benches are excavated. The wall, however deformas subsequent benches are excavated. A maximum deformation of 4.5 mm occurs in the wall

Similarly, for the other sections the following are the modifications to the supports and corresponding results:

Chainage		0 + 104.55 m	0 + 190.55 m
Roof Liner		500 mm at corners	300 mm in the crown and 400 mm at the corner after providing curved corners
Wall Liner		no modification	500 mm
Base Liner		650 mm at corner and 500mm	600 mm after providing curved corners
Maximum deformation	of crown	8 mm	32 mm
	of wall	11 mm	32 mm
Maximum axial force in rockbolts	at crown	0.0365 MN	0.076 MN
	at wall	0.052 MN	0.075 MN

The graphs and figures for these sections can be found in Annex B.

4.8 Results and Discussion

The support systems recommended by the drawings are mostly sufficient for the chainage 0 + 89.10 m and 0 + 104.55 m and yielding is observed only at the corners. However, for chainage 0 + 190.5, most of the support liner including the entirety of the crown liner has yielded. This may mostly be attributed to the poor rock mass of that chainage (Q value of 0.083).

The support system has been made safe by increasing the liner thickness in general and also by providing a curve to the corner for of chainage 0 + 89.10m and 0 + 190.5m. Providing a curved corner has helped in reducing the thickness of liner required otherwise.

The deformations of the crown for all three sections occur only during the excavation stage of the crown and are stable during the excavation of the subsequent benches. However, the deformation of the wall of the cavern continues to increase in the stages when subsequent benches are excavated.

The axial force sustained by a rockbolt at the crown for all three sections seem to take load up to the stage where half the cavern has been excavated. The load taken by the rockbolt does not increase significantly after half the cavern has been excavated and remains almost constant. In the walls also, the load taken by the rockbolt reaches its maximum value at the stage where the bench is excavated and remains almost constant afterwards.

4.9 Conclusion

As seen from the modeling of sections, we can conclude that the support systems provided according to the drawings are not adequate. Numerical analysis of the sections has indicated that support liners need to be increased in order to be fully safe.

Also, the sharp corners of the junction between walls and roofs lead to an accumulation of stress. This is a cause of yielding of liners near corners. One way to mitigate this is by providing a curve to the corners. This change in geometry of the cavern mitigates the stress accumulation in corners.

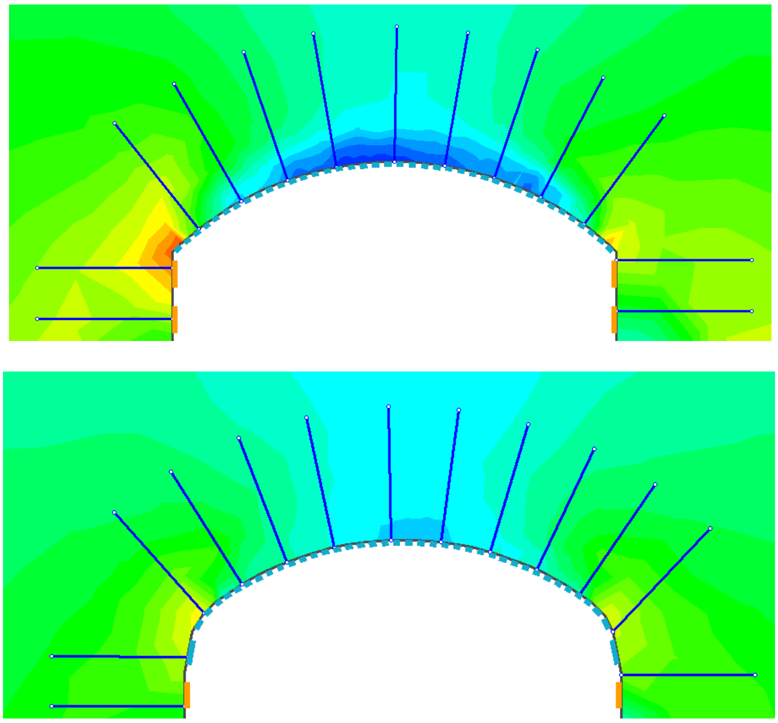


Figure 4-35 Stress around the cavern for sharp corner (top) and for rounded corner (bottom) for chainage 82.90m

However, since some input parameters are based on previous literature and not directly measured, there may have been some inaccuracies in analysis. Also, since data pertaining to discontinuities were not available, these have not been considered in our analysis. Moreover, the tectonic stress, which is highly site specific, has been taken as 6 MPa referring to Gautam, 2012. However, it needs to be measured in site by 3D stress measurements or obtained from back calculations, both of which were not available to us. Also, the effect of groundwater hasn't been explored.

Chapter 5 Structural Analysis and Design of Powerhouse

The structural analysis was done after the dimensioning of the powerhouse was conducted. It includes three parts i.e.,

- Preliminary Design
- Structural Analysis
- Detail Design

Control building is of 1 storey which consists of an office room, store room and extra rooms while Powerhouse is a 5 story building with 20m height which consists of machine hall room and service bay.

5.1 Data Collection:

The functional planning of powerhouse building is based on architectural drawings collected.

Other necessary data used for this project are listed below:

- M20 and M25 concrete have been used.
- Fe500 steel has been used.
- Density of plain concrete is taken as 24 kN/m^3 as per IS 875:1987(part 1)
- Density of Reinforced concrete is taken as 25 kN/m^3 (as per IS 875:1987 part 1)
- Density of Brick work is taken as 20.4 kN/m^3(as per IS 875:1987 part 1)
- Unit weight of imposed load is taken as per IS 875(Part 2):1987.
- Other necessary data required has been suitably assumed.

5.2 Estimation of load

5.2.1 General

Loads are primary consideration in any structural design because they define the nature and magnitude of hazards or external forces that a building must resist to provide reasonable performance (i.e. safety and serviceability) throughout the structure's useful life. The anticipated loads are influenced by a building's intended use (occupancy and function), configuration (shape and size) and location (climate and site conditions). Ultimately, the type and magnitude of the design loads affect critical decisions such has the Material selection, construction details, and architectural configuration. Thus, to optimize the value (i.e. performance versus economy) of the finished product, it is essential to apply design loads realistically. The loads have been estimated as per the IS code.

5.2.2 Dead Loads

This is the permanent of the stationary load like self-weight of the structural elements. This include the following

- Self-weight

- Weight of the finished structure part.
- Weight of partition walls etc.

Dead loads are based upon the unit weights of elements, which are established taking in account materials specified for construction.

The dead loads have been calculated on the basis of unit weights of materials given in IS 875: 1987 (Part I). Unless more accurate calculations are warranted, the unit weights of plain concrete and reinforced concrete made with sand and gravel or crushed natural stone aggregate may be taken as 24 KN/m^3 and 25 KN/m^3 respectively.

5.2.3 Imposed Loads

It is the load assumed to be produced by the intended use or occupancy of a building, including the weight of movable partitions, distributed, concentrated loads, load due to impact and vibration, and dust load but excluding wind, seismic, snow and other loads due to temperature changes, creep, shrinkage, differential settlement, etc.

The magnitude of imposed load depends upon the type of occupancy of the building chosen from code IS 875:1987(Part 2) and IS 4247 (Part I): 1993 for various occupancies.

5.2.4 Seismic Load

Seismic motions consist of horizontal and vertical ground motions, with the vertical motion usually having a much smaller magnitude. The factor of safety provided against gravity loads usually can accommodate additional forces due to vertical acceleration due to earthquakes. So, the horizontal motion of the ground causes the most significant effect on the structure by shaking the foundation back and forth.

Earthquake or seismic load on a structure depends upon the size of the structure, maximum earthquake intensity or string ground motion and the local soil, the stiffness, design and construction pattern, and its orientation in relation to the incident seismic waves. Since the probable maximum earthquake occurrences are not so frequent, buildings are designed for such earthquakes to ensure that they remain elastic and the building is prone to least damage. Seismic load can be calculated taking the view of acceleration response of the ground to the super structure. According to the severity of earthquake intensity they are divided into 5 zones.

- Zone I
- Zone II
- Zone III
- Zone IV
- Zone V

Seismic load will be calculated using seismic coefficient method as specified in IS 1893(part 1):2002

5.2.5 Load combinations

Different load cases and load combination cases are considered to obtain most critical element stresses in the structure in the course of analysis.
There are all together four load cases considered for the structural analysis and are mentioned as below:
Dead Load (DL)
Live load (LL)
Earthquake Loads in X-direction (EQx)
Earthquake Loads in Y-direction (EQy)

Following load combination was adopted as per IS 1893(Part I):2002 Cl.No.6.3.1.2

Table 5-1 Load Combinations

S.N.	Load Combination	SAP Notation
1	1.5(DL+ LL)	UDCON2
2	1.2(DL + LL + EQx)	UDCON3
3	1.2(DL+ LL - EQx)	UDCON4
4	1.2(DL + LL + EQy)	UDCON5
5	1.2(DL + LL - EQy)	UDCON6
6	1.5(DL + EQx)	UDCON7
7	1.5(DL - EQx)	UDCON8
8	1.5(DL + EQy)	UDCON9
9	1.5(DL - EQy)	UDCON10
10	(0.9 DL + 1.5 EQx)	UDCON11
11	(0.9 DL - 1.5 EQx)	UDCON12
12	(0.9 DL + 1.5 EQy)	UDCON13
13	(0.9 DL - 1.5 EQy)	UDCON14

5.2.6 Wind Load

The wind pressure intensity at any height of structure depends on the velocity and density of air, shape and height of the structure, topography of the surrounding, ground surface, angle of wind and geographic location.

When calculating the wind load on individual structural elements such as roofs and walls, and individual cladding units and their fittings, it is essential to take account of the pressure difference between opposite faces of such elements or units. For clad structures, it is, therefore, necessary to know the internal pressures as well as the external pressure. Then the wind load, F, acting in a direction normal to the individual structural element is:

$$F = (Cpe - Cpi).\ A.\ P_d$$

Where,

- Cpe = external pressure coefficient,
- Cpi = internal pressure coefficient,
- A = surface area of structural element or cladding unit, and P_d = design wind pressure

5.2.7 Seismic Weight Analysis:
Calculation of Fundamental Natural Period of Vibration

According to IS 1893:2002 Clause 7.6,

The approximate fundamental natural period of vibration (T_a), in seconds of a moment-resisting frame building with brick infill panels may be estimated by the empirical expressions:

$T_a = 0.075*h^{0.75}$

Where,

h= Height of building, in m.

Building	**Height**	**T_a**
Machine hall	24.5	0.82
Erection bay	24.5	0.82

Base Shear

The total horizontal base shear acting on the building is obtained using the following expression,

$$V_b = A_h \text{ x } W_i$$

Here, A_h is the design horizontal seismic coefficient for a structure that is determined by the following expression,

$$A_h = (Z/2)*(I/R)*(S_a/g) \qquad \text{(\textit{IS 1893:2002 (Part I) Cl 6.4.2})}$$

Provided that for any structure with $T \leq 0.1$s, the value of A_h will not be taken less than Z/2 whatever be the value of I/R,

Where,

Z= Zone factor for the Maximum Considered Earthquake (MCE) and service life of structure in a zone. The factor 2 in the denominator of Z is used so as to reduce the Maximum Considered Earthquake (MCE) zone factor to the factor for Design Basic Earthquake (DBE).

Zone Factor, Z

Seismic Zone	II	III	IV	V
Seismic Intensity	Low	Moderate	Severe	Very Severe
Z	0.1	0.16	0.24	0.36

Zone Factor	Z	0.36	For zone 5(Table 2) of IS1893(Part 1):2002
Importance Factor	I	1.5	From Table 6 (cl.6.4.2) of IS1893(Part 1):2002
Response Reduction Factor	R	5	Table 7 of IS1893(Part 1):2002

Natural period of vibration	Ta	0.825	Cl.7.6.1 of IS1893(Part 1):2002
Spectral acceleration coefficient	Sa/g	2.50	Cl.6.4.4 of IS1893(Part 1):2002
Design horizontal seismic coefficient	A_h	0.139	Cl.6.4.2 of IS1893(Part 1):2002

I= Importance factor, depending upon the functional use of the structures, characterized by hazardous consequences of its failure, post-earthquake functional need, historical value, or economic importance.

I=1.5 for important services and community buildings, such as hospitals, schools, monumental structures, emergency building like telephone exchange, television station, radio station, fire station buildings, large community halls and power stations.

I=1 for all other buildings.

R= Response reduction factor, depending on the perceived seismic damage performances of the structure, characterized by ductile and brittle deformations. However the ratio (I/R) shall not be greater than 1.

R=5 for Special RCC Moment Resisting Frame (SMRF)

S_a/g= Average response acceleration coefficient based on appropriate natural periods and damping of the structure.

For medium soil sites

S_a/g= 1+15 T {$0.0 \leq T \leq 0.10$}
=2.50 {$0.10 \leq T \leq 0.55$}
=1.36/T {$0.55 \leq T \leq 4.0$}

Here, to find A_h

Z = 0.36 (For seismic zone V)
R = 5 (Response reduction factor)
I = 1 (Importance factor)
S_a/g = 1.47 (Spectral acceleration)
g = acceleration due to gravity

Thus,

$A_h = (Z/2)*(I/R)*(S_a/g) = \frac{0.36}{2} * \frac{1}{5} * 1.47592 = 0.053$

Base Shear V_b =0.053 *41129.18

= 2184.9 kN

Lateral Load Distribution

The design base shear V_b computed can be distributed along the height of the building as per following expressions:

$Q_i = (V_b*h_i^2*W_i) / (\sum W_i*h_i^2)$

Where,

i range from 1 to n

n= Number of storey in the building at which the mass is located.

Q_i= Design lateral force at each floor.

h_i= Height of floor i measured from base.

W_i= Seismic weight of each floor.

SEISMIC WEIGHT CALCULATION:

ERECTION BAY										
Floor No	Height	Corbel	Gantry Girder	Beam	Column	Wall	Truss	DL	LL	DL+50%LL
1(Roof)	2.5	0	0	92.5	109.37	425.5	196	823.37	52.5	849.62
2nd	4.75	80	1652.63	222	831.25	808.45	0	3594.33	0	3594.33
3rd	4.25	0	0	222	743.75	723.35	0	1689.1	0	1689.1
Machine Hall										
Floor No	Height	Corbel	Gantry Girder	Beam	Column	Wall	Truss	DL	LL	DL+50%LL
1(Roof)	2.5	0	0	160	203.125	736	142.25	1241.375	52.5	1293.88
2nd	4.75	200	3600	384	1543.75	1398.4	0	7126.15	0	7126.15
3rd	4.25	0	0	384	1381.25	1251.2	0	3016.45	0	3016.45

Erection Bay		
Floor	height (m)	weight (kN)
Weight of floor 1(roof)	2.5	849.62
Weight of floor 2	4.75	3594.33
Weight of floor 3	4.25	1689.1
TOTAL	11.5	6133.05
Horizontal Shear coefficient (Ah)	0.0659	
Base Shear (kN)	710.96	

Floor	Height	Weight	wh*h^2	factor	Lateral Force	Shear Force
1 (roof)	11.5	849.62	112362.25	0.193	145.00	145.00
2	9	4443.95	359959.95	0.617	464.52	609.53
3	4.25	6133.05	110778.22	0.190	142.96	752.48
Total		11426.62	583100.41		752.48	

Machine Hall						
Floor				height (m)		weight (kN)
1(roof)				2.5		1293.88
Weight of floor 2				4.75		7126.15
Weight of floor 3				4.25		3016.45
TOTAL				11.5		11436.48
Horizontal Shear coefficient (Ah)				0.065854		
Base Shear (kN)				1392.83		
Floor	Height	Weight	wh*h^2	factor	Lateral Force	Shear Force
1 (roof)	11.5	1293.88	171115.63	0.161	224.91	224.91
2	9	8420.03	682022.43	0.644	896.42	1121.32
3	4.25	11436.48	206571.42	0.195	271.51	1392.83
Total		21150.39	1059709.48		1392.83	

Storey Drift Calculation:

The storey drift of the building considering earthquake force in X and Y direction are given below:

Storey Drift calculation of Machine Hall/Erection Bay							
Storey	Storey Height(m)	Along X-drection		Along Y-direction		Allowable Drift(=0.004*h) IS1893:2002 Clause7.11.1	Remarks
		Displacement	Storey Drift	Displacement	Storey Drift		
1^{st}	0	0	0	0	0	0	OK
2^{nd}	5.8	0.0000759	0.0000759	0.0000023	0.0000023	0.0232	OK
3^{rd}	4	0.0015	0.001424	0.0001	0.0000977	0.016	OK
4^{th}	4.5	0.0038	0.0023	0.0003	0.0002	0.018	OK
5^{th}	5	0.0087	0.0049	0.0006	0.0003	0.02	OK

For seismic joints:

Maximum displacement in Erection bay, $\Delta 1 = 0.002$

Maximum displacement in Machine Hall,$\Delta 2 = 0.002$

Seismic joint required= $(\Delta 1 + \Delta 2)$*R as provided by IS 1893(Paet1):2002 Clause 7.11.3

Where,R= Response Reduction Factor = 5

So, seismic joint= 10 cm

5.3 Preliminary Design

Approximate sizes of structural members such as slab, column, beam etc. are required before detail analysis of structure. Preliminary design is carried out to estimate the approximate size of the structural member. Preliminary design of the structural members is based on the IS code provisions for slab, beam, column for deflection control. Only gravity loads are considered during preliminary design considering that gravity loads are required to resist the lateral loads acting on the structure.

The calculation part for preliminary design can be seen in Annex A.

Table 5-2 Preliminary design details

Description	Width (mm)	Depth (mm)
Control Building slab		125
Control Building beam	250	400
Control Building column	400	400
Powerhouse beam	400	600
Powerhouse column	800	800
Shear wall		Thickness 650

5.4 Structural Design

The software SAP2000 v.20 is used for the analysis of the structure. The design moments and axial forces in the columns and shear and moments in the beams is found out with the help of the software. The size of the beams, columns and slabs are kept according to the architectural drawing of the powerhouse. Using the available dimensions, a model is created in SAP2000 and analysis is carried out for various load cases as provided by the code. The analysis is carried out for linear static case and the values for lateral loads are provided manually to the software

After preliminary design of components and roof design, a model of powerhouse was created in SAP2000. All the loads acting on the components were added and load cases and combinations were applied according to the code.

The loads applied to the components were roof truss load, crane load, gantry girder load, and earthquake forces in X- and Y-direction.

5.5 SAP Analysis

Input

The grid is formed according to plan of power house and control room, material we use is M25, Fe 500 for the concrete and steel respectively

Load Pattern

In load pattern, different cases of dead load, live load and earthquake load acting on the powerhouse have been considered.

Mass Source

Dead load has multiplying factor of 1.
According to IS: 456:2000 :

- For Live Load < 3 kN/m^2 factor 0.25
- For Live Load > 3 kN/m^2 factor 0.5

Restraint the base joints

The base of the column are restrained

Diaphragm the joints

The diaphragm constraints are defined to the joints at slab level so that all these joints move together with the slab in the same direction.

For each floor diaphragm are formed except at the position where columns are restrained.

Meshing the area
Meshing of the area is done to transfer the slab load uniformly to the beam so that slab and beam deflect in a same pattern.

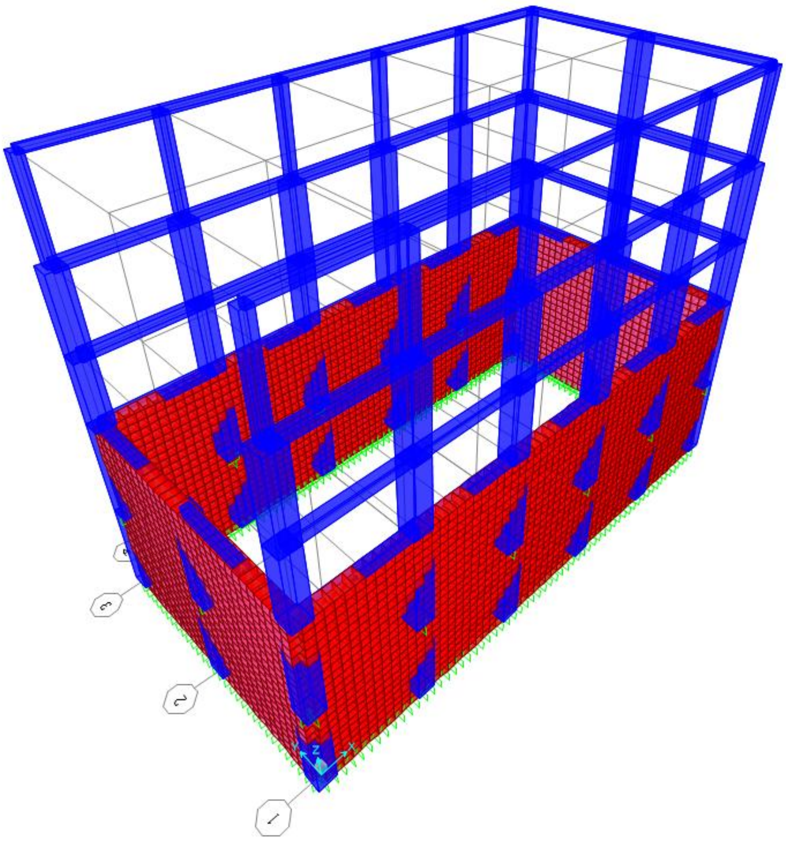


Figure 5-1 3D-View of machine Hall

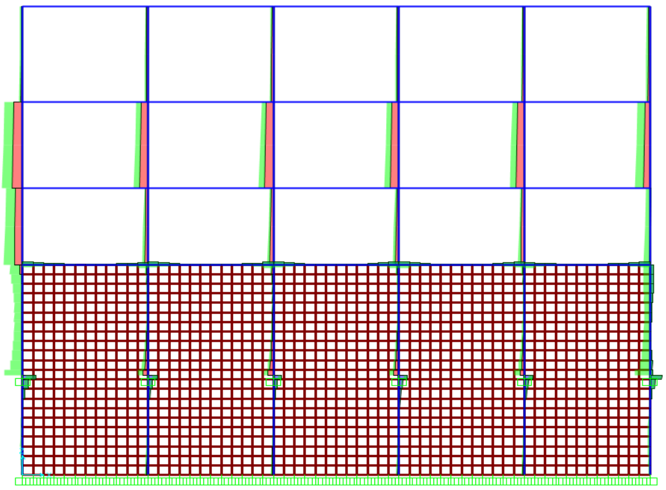


Figure 5-2 Axial Force in X-Z plane

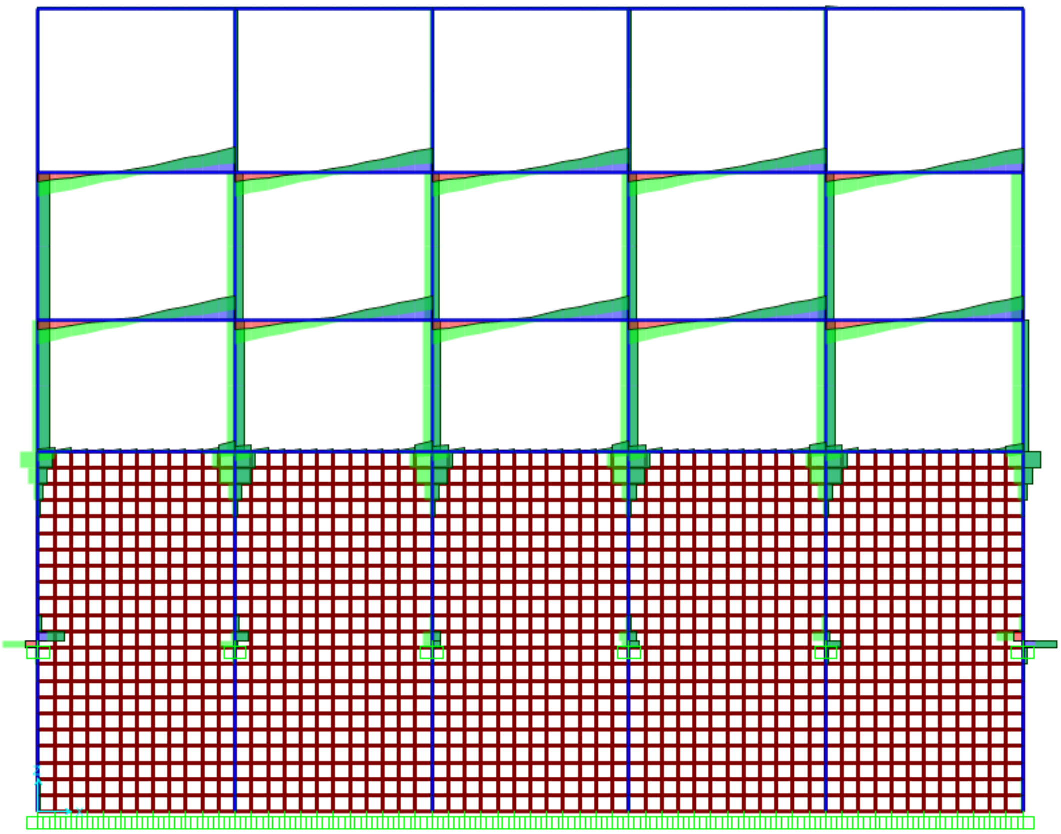


Figure 5-3 Shear Force in XZ plane

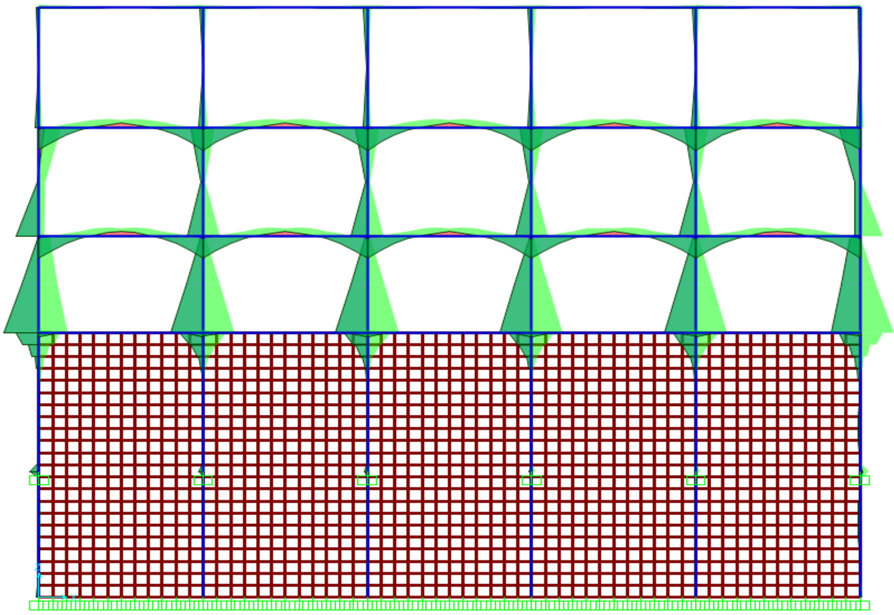


Figure 5-4 BMD in XZ plane

5.6 Detail Design

The detail design of structural members was carried out using limit state method.

The following structural components were designed:

1. Design of Slab
2. Design of Beam
3. Design of Column
4. Design of Staircase
5. Design of Gantry Girder
6. Design of Mat Foundation
7. Design of Corbel
8. Design of Shear Wall

5.6.1 Design of Super Structure:

5.6.1.1 Slab:

Slabs are plate elements forming floors and roofs of buildings and carrying distributed loads primarily by flexure. Inclined slabs may be used as ramps for multistory car parks. A staircase can be considered to be an inclined slab. A slab may be supported by beams or walls. Moreover, a slab is simply supported or continuous over one or more supports and is classified according to the manner of support:

(a) One way slabs spanning in one direction, i.e. supported on two opposite edges
(b) Two way slabs spanning in both directions, i.e. supported on four edges
(c) Circular slabs

(d) Flat slabs resting directly on columns with no beams.

Slabs are designed by using the same theories of bending and shear as are used for beams. The following methods of analysis are available:

(a) Elastic analysis- idealization into strips or beams
(b) Semi empirical coefficients as given in the Code, and
(c) Yield line theory.

A slab is essentially a beam or a flexure member. The minimum span should not be less than four times the overall depth of slab. Slabs are usually much thinner than beams. Slabs are analysed and designed as having a unit width, i.e. one-meter wide beam. Shear stresses are usually very low and shear reinforcement is not provided in slabs. It is preferred to increase the depth of a slab and hence reduce the shear rather than provide shear reinforcement.

There are two types of slabs

(a) One-way slabs: One-way slabs are those in which the length is more than twice the breadth. A one-way slab can be simply supported or continuous. S continuous one-way slab can be analysed in a manner similar to that for continuous beam. In long narrow slabs, where the length is greater than the twice the breadth, the two-way action effectively reduces to one-way action in the direction of the short span although the end beams do carry same slab load.
(b) Two-way slabs: When slabs are supported on four sides, two way spanning action occurs. Such slabs may be simply supported or continuous on any or all sides. The deflections and bending moments in a two-way slab are considerably reduced as compared to those in a one-way slab. Thus, a thinner slab can carry the same load when supported on all four edges.

5.6.1.2 Beams

Beams are structural elements that are capable of withstanding loads basically by resisting against bending. The bending force developed in the beam as a result of self-weight, external loads and span is known as bending moment. Depending upon the end conditions and materials, beams can be categorized into various types like fixed ended beams, simply supported beams, propped cantilever beams, propped cantilever beams, cantilever beams, etc.

Reinforced concrete beams can be categorized into the following types:

1. Singly Reinforced Beams
2. Doubly Reinforced Beams
3. Singly or Doubly Reinforced Beams

In case of singly reinforced and simply supported beams, reinforcements are placed at the bottom of the beam where most of the bending takes place. On contrary, reinforcements are placed at the top in case of cantilever beams.

In doubly reinforced beams, reinforcements are placed at both compression and tension regions. It is when the depth of section is restricted due to functional and aesthetic purposes that steel at the compression zone also need to be provided.

There are two basic types of work: analysis of section and design of section. In analysis of section, the moment of resistance is supposed to be determined putting into consideration the cross section and reinforcement details. In the design of sections, the cross section and amount of reinforcement are supposed to be determined, keeping in consideration the factored design loads.

Beams are designed in accordance to limit state method. Limit state of collapse in flexure is the basis on which the design takes place. Limit states of shear, torsion and serviceability are all checked for beam designs. The grade of concrete used for beam designs is M25 and the grade of steel is Fe500.

5.6.1.3 Columns

A column may be defined as a member carrying direct axial load which causes compressive stresses of such magnitude that these stresses largely control its design. A column or strut is a compression member, the effective length of which exceeds three times the least lateral dimension. Depending upon structural requirement, columns maybe of various shapes, i.e. circular, rectangular, square, hexagonal, etc. A square column has been designed in our case**.**

On the basis of whether slenderness effects are considered insignificant or not, the column may be classified as either a short column or a long column respectively. A short column generally fails by direct compression whereas a long column fails by buckling. Slenderness expressed in terms of the slenderness ratio, which is the ratio of the effective length to the least lateral dimension of the column.

Based on the nature of loading, columns may be classified as either axially loaded, uniaxially loaded or biaxially loaded columns. Axially loaded columns are under pure axial compression whereas uniaxially and biaxially loaded columns have eccentric loading in one or both directions respectively. Columns in framed structures under seismic loads are generally biaxially loaded and hence designed accordingly.

5.6.2 Design of Sub-Structure

5.6.2.1 Shear Wall

Shear wall is constructed to retain the earth and to prevent moisture from seeping into the building. Since the basement wall is supported by the mat foundation, the stability is ensured, and the design of the basement wall is limited to the safe design of vertical stem.

Shear walls are exterior walls of underground structures (tunnels and other earth sheltered buildings) or retaining walls must resist lateral earth pressure as well as additional pressure due to other type of loading. Shear walls carry lateral earth pressure generally as vertical slabs supported by floor framing at the basement level and upper floor level. The axial forces in the floor structures are, in turn, either resisted by shear walls or balanced by the lateral earth pressure coming from the opposite side of the building.

Although Shear walls act as vertical slabs supported by the horizontal floor framing, keep in mind that during the early construction stage when the upper floor has not yet been built the wall may have to be designed as a cantilever.

5.6.2.2 Machine Foundation

Machine foundations require a special consideration because they transmit dynamic loads to soil in addition to static loads due to weight of foundation, machine and accessories. The dynamic load due to operation of the machine is generally small compared to the static weight of machine and the supporting foundation. In a machine foundation the dynamic load is applied repetitively over a very long period of time but its magnitude is small and therefore the soil behavior is essentially elastic, or else deformation will increase with each cycle of loading and may become unacceptable. The amplitude of vibration of a machine at its operating frequency is the most important parameter to be determined in designing a machine foundation, in addition to the natural frequency of a machine foundation soil system. There are many types of machines that generate different periodic forces.

The following general requirements of machine foundations shall be satisfied and results checked prior to detailing the foundations.

- The foundation should be able to carry the superimposed loads without causing shear or crushing failure.
- The combined center of gravity of machine and foundation should, as far as possible, be in the same vertical line as the center of gravity of the base plane.
- No resonance should occur; hence the natural frequency of the foundation–soil system should be either too large or too small compared to the operating frequency of the machine. For low-speed machines, the natural frequency should be high.
- All rotating and reciprocating parts of a machine should be so well balanced as to minimize the unbalanced forces or moments.
- Where possible, the foundation should be planned in such a manner as to permit a subsequent alteration of natural frequency by changing base area or mass of the foundation as may subsequently be required.

Barrel Foundation

The barrel foundation is more common type in use and has the advantages of permitting a lower generator setting and more space outside the turbine pit. It is structurally indeterminate because of the equipment openings and recesses required as well as because of the discontinuity caused by the passage-way to the turbine pit.

The maximum barrel thickness is usually determined by the differences between the radii of the generator stator sole plate and the turbine pit. The height of the barrel and the lateral support afforded by intervening floor permit an intelligent estimate of maximum and minimum thickness.

The weight of generator is estimated as: $W = g\sqrt{k/N} - 85\ ton$ where g = 25 to 35, k is the capacity of generator in kVA and N= speed in rpm. 50% of this is the weight of rotor.

The torque in normal condition is given by $T = k/N$, where k = capacity of generator in kVA and N= speed in rpm. The short circuit torques is obtained by multiplying T by 5

5.7 Design Summary

- **Powerhouse beam:**
  Over all Depth:600mm
  Width:400mm
  Cover :35mm

Table 5-3 Rebar detailing beam along X-X grid of Power house (Beam Label:D3-A3)

S.N.	Section	Rebars Provided
1.	Left support of beam	Top: 2-20mm and 2-16mm rebars Bottom: 2-20mm rebars
2.	Mid span of beam	Top:2-20mm rebars Bottom: 2-20mm rebars
3.	Right support of beam	Top: 2- 20mm and 2-16mm rebars Bottom: 2-20mm rebars

Spacing of stirrups
At left end
2-Legged stirrups of 8mm Ø at 100mm spacingare provided.
At mid span
2-Legged stirrups at 8mm Ø200mm spacingare provided.
At right end
2-Legged stirrups at 8mm Ø100mm spacingare provided.
Anchorage Length: 1010.8mm

Table 5-4 Rebar detailing beam along Y-Y grid of Power house (Beam Label:A2-A3)

S.N.	Section	Rebars Provided
1.	Left support of beam	Top: 3-20mm and 2-16mm rebars Bottom: 2-20mm rebars
2.	Mid span of beam	Top:2-20mm rebars Bottom Rebars: 2-20mm rebars
3.	Right support of beam	Top: 2- 20mm and 2-16mm rebars Bottom: 2-20mm rebars

Spacing of stirrups
At left end
2-Legged stirrups of 8mm dia. at 100mm spacing are provided.
At mid span
2-Legged stirrups at 8mm dia.200mm spacing are provided.
At right end
2-Legged stirrups at 8mm dia.100mm spacing are provided.
Anchorage Length: 1010.8mm

Table 5-5 Rebar detailing beam along X-X grid of Power house (Beam Label:N6-O6)

S.N.	Section	Rebars Provided
1.	Left support of beam	Top: 2-16mm and 1-20mmØ rebars Bottom: 3-16mm rebars
2.	Mid span of beam	Top:2-16mm rebars Bottom: 3-16mm rebars

3.	Right support of beam	Top: 2- 20mm and 2-16mm rebars Bottom: 3-16mm rebars

Spacing of stirrups
At left end
2-Legged stirrups of 8mm dia. at 100mm spacing are provided.
At mid span
2-Legged stirrups at 8mm dia.180mm spacing are provided.
At right end
2-Legged stirrups at 8mm dia.100mm spacing are provided.
Anchorage Length: 1010.8mm

Table 5-6 Rebar detailing beam along Y-Y grid of Power house (Beam Label:L6-L7)

S.N.	Section	Rebars Provided
1.	Left support of beam	Top: 2-16mm rebars Bottom: 2-16mm rebars
2.	Mid span of beam	Top:2-16mm rebars Bottom: 2-16mm rebars
3.	Right support of beam	Top: 2- 16mm rebars Bottom: 2-16mm rebars

Spacing of stirrups
At left end
2-Legged stirrups of 8mm dia. at 100mm spacing are provided.
At mid span
2-Legged stirrups at 8mm dia.180mm spacing are provided.
At right end
2-Legged stirrups at 8mm dia.100mm spacing are provided.
Anchorage Length: 1010.8mm

- **Powerhouse column**
  Dimension: 800mm*800mm
  Cover: 50mm

Table 5-7 Reinforcement Bars for Powerhouse Column

S.N.	**Column Size (mm^2)**	**Level**	**Rebars**
1.	1000*1000	Below Gantry level	12-32 + 4*25 mm Rebars
2.	500*500	Above Gantry level	12-25mm Rebar

Table 5-8 Spacing of stirrups at zones above and below gantry girder:

S.N.	**Zone**	**Spacing of Stirrups**
1.	Tension	8mm Ø @ 100mm c/c
2.	Beam Intersection	8mm Ø @ 150mm c/c
3.	Lapping	8mm Ø @ 100mm c/c
4.	Compression (Below Gantry Girder)	8mm Ø @ 250mm c/c
5.	Compression(Above Gantry Girder)	8mm Ø @ 150mm c/c

Lapping distance: 1000mm.
Confinement length below gantry level: 1300mm
Confinement length above gantry level: 1200mm

Table 5-9 Reinforcement Bars for Control Room Column

S.N.	Column Size (mm^2)	Rebars
1.	400*400	8-16mm ∅Rebars

Table 5-10 Spacing of stirrups at various zones

S.N.	Zone	Spacing of Stirrups
1.	Tension	8mm ∅ @ 100mm c/c
2.	Beam Intersection	8mm ∅ @ 150mm c/c
3.	Lapping	8mm ∅ @ 100mm c/c
4.	Compression	8mm ∅ @ 200mm c/c

Lapping distance: 1000mm.
Confinement length: 1100mm

- **Gantry Girder**
  Built-up section: IS WB 600 @133.7 kg/m and IS MC 400@ 35.8kg/m
  Thickness of weld: 7mm

- **Corbel**
  Dimension of bearing plate: 300mm*250mm
  Depth of front face of corbel: 1000mm
  Depth of vertical inclined face: 700mm
  Cover: 50 mm

Table 5-11 Reinforcement details of corbel

S.N.	Reinforcement Type	Rebars
1.	Tension Reinforcement	8-25mm ∅ rebars
2.	Shear Reinforcement	10-12mm ∅ 2 legged stirrup at upper $\frac{2}{3}$ depth of corbel

- **Shear wall**
  Thickness: 650mm
  Clear Cover: 40mm
  Overall depth of wall, D: 650 mm

Table 5-12 Reinforcement detail for Shear Wall

S.N.	Face	Horizontal Reinforcement	Vertical Reinforcement
1.	Inner	12mm-Φ bar @ 250 mm c/c	16mm- Φbar @ 75 mm c/c
2.	Outer	16mm-Φ bar @ 200 mm c/c	20mm-Φ bar @100 mm c/c

- **Mat foundation**
  Overall depth: 1400mm
  Cover: 50mm

Table 5-13 Reinforcement detail of mat foundation

S.N.	Axis	Rebars
1.	X- Axis	Top: 16mm Ø rebars @ 100mm c/c Bottom: 16mm Ø rebars @ 200mm c/c
2.	Y- Axis	Top: 16mm Ø rebars @ 100mm c/c Bottom: 16mm Ø rebars @ 200mm c/c

- **Powerhouse Slab**

Overall depth: 155mm

Table 5-14 Reinforcement detail of powerhouse slab

S.N.	Section	Span	Reinforcement
1.	Support	Short	10mm bars @ 150mm c/c
		Long	10mmϕ bars @ 200mm c/c
2.	Mid Span	Short	10mmϕbars@200mmc/c
		Long	10mmϕ bars @ 200mm c/c
3.	Edge	Short	10mmϕbars@ 250mmc/c
		Long	10mmϕ bars @ 150mm c/c

- **Control Room Slab**

Overall depth: 125mm

Table 5-15 Reinforcement detail of control room slab

S.N.	Section	Span	Reinforcement
1.	Support	Short	8mm bars @ 250mm c/c
		Long	8mmϕ bars @ 250mm c/c
2.	Mid Span	Short	8mmϕbars@ 200mmc/c
		Long	8mmϕ bars @ 300mm c/c
3.	Edge	Short	8mmϕbars@ 200mmc/c
		Long	8mmϕ bars @ 150mm c/c

- **Barrel Foundation:**

Table 5-16 Reinforcement detail of Barrel Foundation

S.N.	Components	Reinforcement
1.	Barrel	28mm-Φ bar @ 300 mm c/c at inner and outer periphery
2.	Rotor	25mm-Φ bar @ 160 mm c/c at both faces
3.	Rotor pedestal (Sole plate)	6 no. of 16mm
4.	Longitudinal	25mm-Φ bar @ 250 mm c/c

Chapter 6 Conclusion

The project has studied the following items: ventilation tunnel of surge shaft cavern, cavern of the underground settling basin and the powerhouse. The study has applied the empirical, analytical and numerical methods for the sections of ventilation tunnel. Accordingly, the results were obtained and are available in their respective chapter. The case of tunneling in weak and disintegrated rock mass also has been done taking a case of the headrace tunnel. For it, the effect of forepoling has been studied by 2D numerical study. The settling basin cavern has been studied too. The defined support system has been studied under numerical investigation and modified support too has been recommended.

In the case of the powerhouse, we have carried out the structural design on the basis of architectural drawings provided by SMHEP. Alongside, the cost estimation for the powerhouse was also carried out.

The conclusion to our project may be explained in 2 separate sub headings:

6.1 Underground Structure

1. The squeezing assessment by empirical method reveal that none the of the section of the ventilation tunnel will undergo squeezing.

2. Comparison of the Q-system and RMR system by analytical method (CCM) shows that RMR support is stiffer and can withstand more stress. However, the Q supports are capable of deforming more before yielding.

3. Although deformation for the RMR guidelines is slightly lesser than Q support, the factor of safety for the Q supports is actually higher. This may be explained by the fact that the Q supports act over a wider range of deformation as evident by the lesser slope of the SCC and the fact that the support is activated after more deformation has occurred which means that ground has relaxed more and pressure is lesser.

4. Rock mass classifications do not recommend the support for the invert but modeling of sections recommend that invert liners be installed.

5. There is a huge concentration of stresses in the sharp corners of the tunnel sections. This requires to very thick linings of concrete which are uneconomical. A way to reduce liner thickness in corners is by rounding off the corners.

6. For tunneling in weak rock, the support requirements are very high which lead to uneconomic support systems and decrease in the tunnel cross-section. In addition to this, deformations are very high in such conditions. To mitigate this squeezing behavior the installation of forepole umbrella is very effective as evident by the analysis done in section.

7. Numerical analysis of the settling basin caverns reveals that there is large concentration of stresses at the sharp corners of the basin. This requires thicker lining of concrete to withstand it. However, the stress concentration can also be reduced by

providing a curve at the corners instead of sharp corners. These options may be used in conjunction to provide stable support for the caverns.

8. The stresses in the rock pillar between two settling basin caverns are well below the global compressive strength of the rock mass.

9. The modeling procedure recommended by Usmani et. al. 2015 seemed applicable in our case as Usmani et al 2015 had verified the process by instrumentation in their site.

6.2 Hydropower Powerhouse

In the case of the powerhouse, we have carried out the structural design and analysis of powerhouse on the basis of architectural drawings provided by SMHEP. The preliminary design of powerhouse was performed which was structurally analyzed and followed by the detailed design of the components of powerhouse. The dimension of all the structural components of the powerhouse, the effects of the loads on the structural components was analyzed and the detail of reinforcement requirement for the designed structural components has been perfomed. Alongside, the cost estimation for the powerhouse was also carried out.

This project helped us gain knowledge about the practical application of our field and made us familiar with the software we will be using during our professional career. One of the outcomes of the analysis we performed is it gave the knowledge about the maximum deflection at the top of the different powerhouse building components so that we were aware about how much space should be provided between them during construction in order to pursue their safety. This project covers the design with structural safety and estimation of the cost and materials for the project.

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Annex A: Structural Design of Powerhouse and Control unit

Preliminary Design

Design of Generator Floor Slab

References	Steps	Calculations	Remarks
IS 456:2000 Clause 23.2	**1.**	Taking M25 grade of concrete, Taking largest slab of dimension = 7.6*5m L/d_{min} = 26* Modification factor For modification factor, f_s= 0.58*fy*(Area of steel reqd.)/(Area of steel provided) Assuming area of steel required equal to area of steel provided.	
IS 456:2000 Clause 23.1(c) Fig. 4	**2.**	f_s = 0.58*500 = 290 N/mm^2 Size of slab = 7.6*5 sq. m Assuming percentage of steel to be 0.3%, MF = 1.4 L/d =26*1.4 = 32.5 d = 5000/(25*1.4) = 129.36	
	3.	Take, 130 mm Check: Self-weight = 25*0.15 = 3.75 kN/m^2 Imposed load = 7 kN/m^2 Floor finish = 1.1 kN/m^2 Total load = 14.85 kN/m^2 Adopt B = 1000mm Total design load (w_u)= 1.5* 14.1*1 = 22.275 kN/m $$\text{Mu} = \frac{\text{wl}^2}{8} = 69.60\ kN/m$$ $$\text{d} = \sqrt{\frac{M_u}{0.138\ f_{ck}\, b}} = 126.3\ mm$$ Adopt effective depth, d= 130 mm & effective cover = 25 mm	**Effective depth, d= 130 mm** **Effective cover d'= 25 mm** **Total depth D =155 mm**

Design of Control Unit Slab

References	Steps	Calculations	Remarks
IS 456:2000 Clause 23.2	**1.**	Taking M25 grade of concrete, L/d_{min} = 26* Modification factor For modification factor,	

IS 456:2000 Clause 23.2.1(c), fig 4	**2.** **3.**	f_s= 0.58*f_y*(area of steel required / area of steel provided) Assuming area of steel required equal to area of steel provided. f_s = 0.58*500 = 290 N/mm^2 Size of slab = 5*3.4 sq. m Assuming percentage of steel to be 0.3%, MF = 1.4 L/d =26*1.4 = 36.4 d = 5000/36.4 mm = 95 Take, 100mm Check: Self-weight = 25*0.15 = 3.75 kN/m^2 Imposed load = .75 kN/m^2 Floor finish = 1.1 kN/m^2 Total load = 17 kN/m^2 Adopt B = 1000mm Total design load (w_u)= 1.5* 5.6*1 = 8.4 kN/m $\text{Mu} = \frac{\text{wl}^2}{8} = 12.138\ kN/m$ $\text{d} = \sqrt{\frac{M_u}{0.138\ f_{ck}\, b}} = 90.23\ mm$ Adopt effective depth, d= 100 mm & effective cover = 20 mm	**Effective depth, d= 100 mm** **Effective cover d'= 25 mm** **Total depth D =125 mm**

Design of Control Room Beam

References	Steps	Calculations	Remarks
IS 456:2000 Clause 23.2.1	**1.**	For fixed beam, Span/ d_{min} = (12- 15) Span of beam = 6.5 m d_{min}= 6500/(13 to15) D=400 mm Now, B= D/2 to D/3 = 250 mm	**Beam of 250 mm *400 mm**

Design of Erection Bay Beam

References	Steps	Calculations	Remarks
IS 456:2000 Clause 23.2.1	**1.**	For fixed beam, Span/ d_{min} = (12- 15) Span of beam = 6.5 m d_{min}= 7500/(13 to15) D= 600 mm Now, B= D/2 to D/3 = 400 mm	**Beam of 400 mm *600 mm**

Design of Machine Hall Column

References	Steps	Calculations	Remarks
Powerhouse guidelines by DOED	**1.**	Wt. from crane = 500 kN Load from gantry = 5kN Total load on rail runway = 230 kN Load from gantry on each column = 12.8 kN	
	2.	Load from beam on column Self-wt. of beam = 4.8 kN/m Total load from beam = 33.6 kN Totalload= (500+5+230++12.8+4.8 +121.9* 10^{-6} A_g) P_{uz}=1.2*3* Total load A_g = 0.8* 0.8	
	3.	Check for deflection $e_{min} = \frac{L}{500} + \frac{D}{30} = 26.31 > 20\ , So, okay$	**So, taking a column of size 800 mm * 800 mm**

Design of Shear Wall

References	Steps	Calculations	Remarks
	1.	Height of shear wall, H = 6 m Concrete grade, fck = 25 MPa Unit weight of sand and gravel (γ) = (14.7-22.6) kN/m3 Adopt γ = 20 kN/m3 Angle of plane of failure (ϕ) = 30° Surcharge = 20kN/m2 Active pressure coefficient (Ka) = $\frac{1-\sin\emptyset}{1+\sin\emptyset}$ = 0.333	
	2. a.	Calculation of active pressure, At top of wall, Due to over burden, Pa = Ka. γ. H1 = 0.333 x 20 x 0 = 0	

<table>
<tr>
<td>IS 456:2000</td>
<td>b.

3.</td>
<td>Due to surcharge, Pa = Ka *q
= 0.333*20
= 6.6kN/m2
At bottom of wall,
Due to overburden, Pa = Ka. γ. H2
= 0.333x 20 x 6
= 40 kN/m2
Due to surcharge, Pa = Ka *q
= 0.333*20
= 6.6kN/m2

Active pressure force = Area of active pressure diagram
<table>
<tr><td>Area</td><td>P</td><td>H from bottom</td></tr>
<tr><td>1.</td><td>0.5*6*40 = 120</td><td>6/3 = 2 m</td></tr>
<tr><td>2.</td><td>6.6*6 = 39.6</td><td>6/2 = 3 m</td></tr>
</table>
Let b =1 m
Max B.M. = 120*2+ 39.6* 3 = 358.8 kNm
Factor of Safety =1.5
Factored moment, Mu = 1.5 x 358.8 = 538.2 kNm
From thickness of wall with moment consideration is given by:
$d=\sqrt{\frac{\mathrm{Mu}}{0.138*\mathrm{fck}*\mathrm{b}}} = \sqrt{\frac{538.2}{0.138*25*1000}} = 400\ mm$
Adopt
Effective depth (d) = 400 mm
Effective cover = 50mm
Overall depth (D) = 400+50= 450mm
Take D = 450 mm</td>
<td></td>
</tr>
</table>

Design of Roof Truss

Materials and their specifications

Grade of steel tubular pipe used		As per IS 1161-2014
CGI Sheet	0.056 kN/m^2	As per IS 875-1987 (Part I)
CGI sheet	0.078 kN/m^2	40% increased due to lagging of CGI

Dead Load

Truss Spacing	4.6 m
Spacing of purlins at middle nodes	1.54
Spacing of purlins at end nodes	1.54
Roof Slope	12.5º

Live Load

Description	Value	Units	Remarks
Minimum imposed load for roof where access not provided	0.75	kN/m^2	IS 875:1987 (Part 3)
Live load as per IS 875 for roof slope >10º	0.70	kN/m^2	slope >10º X= 0.75-0.02*(12.5º-10) IS 875: 1987(Part 3)

Wind load on Roof Truss

Wind load has been calculated as per IS: 875 – 1987 (Part-3)

Where,

$$P_z = 0.6\, V_z^2$$

$V_z = K1 * K2 * K3 * V_b$

Vb =basic wind speed in m/s

k1 =Probability factor (or risk coefficient)

k2 =Terrain, height and structure size factor

k3 =Topography factor

Terms	Values	Remarks
Calculation of Probability Factor (Risk Coefficient), K1		
Basic Wind Speed	47 m/s	For Nepal
Life Span	100 years	Assumed
Risk Coefficient(K1)	1.07	IS 875-1987(Part 3), Table 1

Calculation of Terrain, height and structure size factor, K2		
Width of Building(B)	16m	
Length of Building(L)	47m	
Height of Building (H)	17m	
Class of Building	B	Since largest dimension of structure is within 20m to 50 m
Terrain Category	2	PH structure is in the river bank which has scattered obstruction with height less than 10m
K2	1	IS:875-1987(Part3), Table 2
Calculation of Topography Factor, K3		
K3	1	

So,

Design wind velocity Vz= 50.29 m/s
Design wind pressure Pz = 1517.45 N/m^2
Height to width ratio h/w = 1.06

Windward	Leeward	¼ of L	½ of L	
0	0	90	90	Wind Angle
EF	GH	EG	FH	Face
-0.8	-0.6	-1.0	-0.6	C_{pe}
C_{pi} (Assuming medium permeability)		+0.2	-0.2	

C_{pe}- C_{pi}	-1.0	-0.8	-1.2	-0.8
C_{pe} + C_{pi}	-0.6	-0.4	-0.8	-0.4

Wind load intensity $= (C_{pe} - C_{pi}) * P_z$

-1.517	-1.214	-1.821	-1.214	kN/m^2
Upward	Upward	Upward	Upward	

-0.910	-0.607	-1.214	-0.607	kN/m^2
Upward	Upward	Upward	Upward	

Design of Erection Bay Slab

Reference	Step	Calculation	Output
Slab ID S(A-B)(1-2) Two Adajacent Edge Discontinuos)	**1.**	**Clear Spans** L_x = 7m L_y = 7.5m Provided Thickness (d) = 130 mm Effective Cover = 25 mm Overall Depth = 130 + 25 = 155mm $l_x = L_x + d$ = 7.130m $l_y = L_y + d$ = 7.630m **Check** l_y/l_x = 7.730/5.130= 1.07<2 (Two way Slab)	**Overall depth, D =155mm**
Table 2 IS 4247 (Part I) : 1993	**2.**	**Design Load** Self-weight of slab = 25*0.155 = 3.875kN/m^2 Imposed Load on control bay floor = 10 kN/m^2 Floor Finish = 1.1 kN/m^2 Considering 1000 mm strip of slab along shorter span Dead Load = 4.975 kN/m^2 Live Load = 10 kN/m^2 Design Load (w_u) = 1.5(D.L. + L.L.) = 22.46kN/m^2	
IS 456: 2000 Table 26 **IS 456:2000 AnnexD-1.1**	**3.**	**Moment Calculation** Negative bending moment coefficient at continuous edge Short span coefficient, $-\alpha_x$ = 0.050 Long span coefficient, $-\alpha_y$ = 0.047 Positive bending moment coefficient at mid span $+\alpha_x$ = 0.038 $+\alpha_y$ = 0.035 **For short span** Mid span moment (M_{x1}) = $\alpha_x * w_u * l_x^2$ = 0.038* 22.46* 7.13^2 = 33.10 kN-m Support moment, (M_{x2}) = $\alpha_x * w_u * l_x^2$ = 0.050* 22.46*7.13^2 = 44.38 kN-m **For longer span** Mid span moment (M_{x1}) = $\alpha_y * w_u * l_x^2$ = 0.035 * 22.46* 7.13^2 = 20.68 kN-m Support moment, (M_{x2}) = $\alpha_y * w_u * l_x^2$	

		$= 0.047 * 22.46 * 7.13^2$ $= 27.78$ kN-m		
	4.	**Check for depth from moment considerations** Depth of slab $d = \sqrt{\frac{Mmax}{0.138 * fck * b}} = \sqrt{\frac{44.38 * 10^6}{0.138 * 25 * 1000}} = 113$ mm Provided effective depth = 130 mm > 113 mm	**Okay**	
IS 456:2000 Annex D-1.2	5.	**Length of middle and edge strips** Along short edge, Width of middle strips = ¾ th of the span = ¾ * 5130 ≈ 3848mm Width of edge strips = 1/8 th of the span = 1/8 * 5130 ≈ 642mm Along long edge, Width of middle strips = ¾ th of the span = ¾ * 7730 ≈ 5798mm Width of edge strips = 1/8 th of the span = 1/8 * 7730 ≈ 967mm		
IS456:2000 CL26.5.2.1	6.	**Calculation of minimum area of steel** Min A_{st} = 0.12% of bD = 0.0012 * 1000 * 155 = 186mm^2		
Annex G-1.1 SP 16:1980 Table 96 Annex G-1.1	7.	**Reinforcement in middle strips** i. Steel along short span $M_{uxl} = 0.87 * f_y * A_{st} * \left(d - \frac{Ast * fy}{fck * b}\right)$ $33.10 * 10^6 = 0.87 * 500 * A_{st}\left(130 - \frac{Ast * 500}{1000 * 25}\right)$ $A_{st\ required} = 509.35\ mm^2$ Providing 12 mm ϕ bars @ 220 mm c/c $A_{st\ provided} = 541.07\ mm^2$	**Providing 12 mm ϕ bars @ 220 mm c/c** **$A_{st\ provided}$ = 514.078 mm^2**	
SP 16:1980 Table 96		ii. Steel along long span $M_{uyl} = 0.87 * f_y * A_{st} * \left(d - \frac{Ast * fy}{fck * b}\right)$ $20.68 * 10^6 = 0.87 * 500 * A_{st}\left(130 - \frac{Ast * 500}{1000 * 25}\right)$ $A_{st\ required} = 388.97mm^2$ Providing 12 mm ϕ bars @ 300 mm c/c $A_{st\ provided} = 392.69mm^2$	**Providing 12mm ϕ bars @ 300 mm c/c** **$A_{st\ provided}$ = 392.69 mm^2**	
	8.	**Reinforcement in supports** Length of support reinforcement = 7730/5= 1546mm Taking 1000mm	**Providing 12mm ϕ bars @ 120**	

IS456:2000 **CL26.5.2.1**		Steel along short span $M_{ux2} = 0.87 * f_y * A_{st} * \left(d - \frac{Ast*fy}{fck*b}\right)$ $44.38*10^6 = 0.87*500* A_{st}\left(130 - \frac{Ast*500}{1000*25}\right)$ $A_{st\ required} = 913.04\ mm^2$ Providing 12 mm ϕ bars @120 mm c/c $A_{st\ provided} = 942.47\ mm^2$ i. Area of steel along long span $M_{uy2} = 0.87 * f_y * A_{st} * \left(d - \frac{Ast*fy}{fck*b}\right)$ $27.78*10^6 = 0.87*500*A_{st}*\left(130 - \frac{Ast*500}{1000*25}\right)$ $A_{st\ required} = 535.33\ mm^2$ Providing 12 mm ϕ bars @220 mm c/c $A_{st\ provided} = 541.07\ mm^2$	**mm c/c** **$A_{st\ provided}$ = 942.47 mm^2** **Providing 12mm ϕ bars @ 220 mm c/c** **$A_{st\ provided}$ = 541.65 mm^2**
	9.	**Reinforcement in edge strips** Min A_{st} = 0.12% of bD = 0.0012 * 1000 * 155 = 186 mm^2 Providing 12mm ϕ bars @ 150mm c/c, 3nos. along long span and 12mm ϕ bars @ 250mm c/c, 3nos. along the short span	**Providing 12mm ϕ bars @ 150 mm c/c in long span and 12mm ϕ bars @ 250 mm c/c in short span**
IS 456:2000 **Cl 40.1** **IS 456:2000** **Table 19** **IS 456:2000** **Table 20** **IS 456:2000** **Cl 40.2.1.1**	**10.**	**Check for shear** For short span Maximum shear, $V_{max} = \frac{wu*lx}{2} = \frac{22.46*5.130}{2}$ = 57.60 kN Nominal shear stress $\tau_v = \frac{Vmax}{b*d} = \frac{57.60*10^3}{1000*130}$ = 0.44 N/mm^2 Percentage tensile stress = $\frac{100*Ast}{b*d} = \frac{100*392}{130*1000}$ = 0.3% Maximum shear stress for concrete grade M25 = 3.1N/mm^2 Design shear strength of concrete for M25 concrete grade, τ_C = 0.38N/mm^2 For 155mm overall depth of slab k = 1.29 Design shear strength(τ') = τ_C.k = 0.49 $N/mm^2 > \tau_V$	**Okay**
	11.	**Check for deflection** Along short span, The basic value for continuous slab ; L/d = 26 $f_s = 0.58 * f_y * \frac{Ast\ required}{Ast\ provided}$	**Safe**

IS456:2000 **Cl 23.2.1** **Fig. 4** **IS456:2000** **CL 26.2.3.3**	 **12.**	$= 0.58 * 500 * \frac{913}{1047.2}$ $= 252.8$ Modification factor = 1.6 $(L/d)_{allowable} = 26*1.6 = 41.6$ $(L/d)_{actual} = \frac{5130}{130} = 39.46$ $(L/d)_{actual} < (L/d)_{allowable}$ **Check for development length** MOR = $0.87 * f_y * A_{st} *(d- \frac{Ast*fy}{b*fck})$ $= 0.87 * 500 * 1047.2*(105- \frac{1047.2*500}{1000*25})$ =38.29 kN-m V_u = 57.60 kN-m Now, Anchorage value of bars, Lo = greater of 12 ϕ and effective depth=260mm $1.3\frac{M}{V} + Lo = 1.3\frac{38.29*10^6}{57.60*10^3} + 125 = 989.18mm$ And Development length of bars $Ld = \frac{0.87*fy*\phi}{4\tau_{bd}} = \frac{0.87*10*500}{4*1.6*1.4} = 485.49mm < 989.18mm$	 **Okay**

Design Summary

Overall depth = 155mm

- Reinforcement in the short span (at support) = 12mmϕ bars @ 120mm c/c
- Reinforcement in the long span (at support) =12mmϕ bars @ 220mm c/c
- Reinforcement in the short span(at mid span)=12mmϕbars@220mmc/c
- Reinforcement in the long span (at mid span) = 12mmϕ bars @ 300mm c/c
- Reinforcement along the short span (at edge) =12mmϕ bars @ 250mm c/c
- Reinforcement along the long span(at edge)=12mmϕbars@ 150mmc/c

Design of Control Room Slab

Slab ID **S(L-M)(5-6)** **(One Short Edge Discontinuous)**	**1.**	**Clear Spans** L_x = 3.4m L_y = 5m Provided Thickness (d) = 100 mm Effective Cover = 25 mm Overall Depth = 100 + 25 = 125m $l_x = L_x + d$ = 3.5m $l_y = L_y + d$ = 5.1m **Check** l_y/l_x = 5.1/3.5= 1.45<2 (Two way Slab)	**Overall depth, D =125mm**
Table 2 **IS 4247** **(Part I) : 1993**	**2.**	**Design Load** Self-weight of slab = 25*0.125 = 3.125kN/m^2 Live Load on roof = 2 kN/m^2 Floor Finish = 1.1 kN/m^2 Considering 1000 mm strip of slab along shorter span Dead Load = 4.225 kN/m^2 Live Load = 2 kN/m^2 Design Load (w_u) = 1.5(D.L. + L.L.) = 9.34 kN/m	
IS 456: 2000 **Table 26** **IS 456:2000** **AnnexD-1.1**	**3.**	**Moment Calculation** Negative bending moment coefficient at continuous edge Short span coefficient, $-\alpha_x$ = 0.0425 Long span coefficient, $-\alpha_y$ = 0.037 Positive bending moment coefficient at mid span $+\alpha_x$ = 0.056 $+\alpha_y$ = 0.028 **For short span** Mid span moment (M_{x1}) = $\alpha_x*w_u*l_x^2$ = 0.056 * 9.34* 3.5^2 = 6.4 kN-m Support moment, (M_{x2}) = $\alpha_x*w_u*l_x^2$ = 0.0425 * 9.34* 3.50^2 = 4.86 kN-m **For longer span** Mid span moment (M_{x1}) = $\alpha_y*w_u*l_x^2$ = 0.028* 9.34* 3.50^2 = 3.20 kN-m Support moment, (M_{x2}) = $\alpha_y*w_u*l_x^2$ = 0.037 *9.34* 3.50^2 = 4.23kN-m	
	4.	**Check for depth from moment considerations** Depth of slab	

		d = $\sqrt{\frac{Mmax}{0.138*fck*b}} = \sqrt{\frac{6.4*10^6}{0.138*25*1000}}$ = 43 mm Provided effective depth = 100 mm > 43 mm	**Okay**
IS 456:2000 **Annex D-1.2**	**5.**	**Length of middle and edge strips** Along short edge, Width of middle strips = ¾ th of the span = ¾ * 3500 ≈ 2625mm Width of edge strips = 1/8 th of the span = 1/8 * 3500 ≈ 450mm Along long edge, Width of middle strips = ¾ th of the span = ¾ * 5100 ≈ 3825mm Width of edge strips = 1/8 th of the span = 1/8 * 5100 ≈ 630mm	
IS456:2000 **CL26.5.2.1**	**6.**	**Calculation of minimum area of steel** Min A_{st} = 0.12% of bD = 0.0012 * 1000 * 125 = 150 mm^2	
Annex G-1.1 **SP 16:1980** **Table 96** **Annex G-1.1** **SP 16:1980** **Table 96**	**7.**	**Reinforcement in middle strips** i. Steel along short span $M_{uxl} = 0.87 * f_y * A_{st} * \left(d - \frac{Ast*fy}{fck*b}\right)$ $6.41*10^6 = 0.87*500* A_{st}\left(100 - \frac{Ast*500}{1000*25}\right)$ $A_{st\ required}$ = 151.97mm^2 Providing 10 mm ϕ bars @ 200 mm c/c $A_{st\ provided}$ = 392.69 mm^2 ii. Steel along long span $M_{uyl} = 0.87 * f_y * A_{st} * \left(d - \frac{Ast*fy}{fck*b}\right)$ $3.20*10^6 = 0.87*500* A_{st}\left(100 - \frac{Ast*500}{1000*25}\right)$ $A_{st\ required}$ = 74.67 mm^2 Providing 10mm ϕ bars @ 300 mm c/c $A_{st\ provided}$ = 167.55 mm^2	**Providing 10 mm ϕ bars @ 200 mm c/c** **$A_{st\ provided}$ = 392.69 mm^2** **Providing 10mm ϕ bars @ 300 mm c/c** **$A_{st\ provided}$ = 167.55 mm^2**
	8.	**Reinforcement in supports** Length of support reinforcement = 4775/5= 955mm Taking 1000 mm i. Steel along short span $M_{uxl} = 0.87 * f_y * A_{st} * \left(d - \frac{Ast*fy}{fck*b}\right)$ $4.86*10^6 = 0.87*500* A_{st}\left(100 - \frac{Ast*500}{1000*25}\right)$	**Providing 10mm ϕ bars @ 250 mm c/c** **$A_{st\ provided}$ = 201.062 mm^2**

IS456:2000 **CL26.5.2.1**		$A_{st\ required}$ = 114.38mm^2 Providing 10mm ϕ bars @ 250 mm c/c $A_{st\ provided}$ = 201.062 mm^2 ii. Area of steel along long span $M_{uy2} = 0.87 * f_y * A_{st} * \left(d - \frac{Ast*fy}{fck*b}\right)$ $4.23*10^6 = 0.87*500*A_{st}*\left(100 - \frac{Ast*500}{1000*25}\right)$ $A_{st\ required}$ = 99.20 mm^2 Providing 10 mm ϕ bars @250 mm c/c $A_{st\ provided}$ = 201.062 mm^2	**Providing 10mm ϕ bars @ 250 mm c/c** **$A_{st\ provided}$ =201.062 mm^2**
IS 456:2000 **Cl 40.1**	**9.**	**Reinforcement in edge strips** Min A_{st} = 0.12% of bD = 0.0012 * 1000 * 125 = 150 mm^2 Providing 8mm ϕ bars @ 150mm c/c, 3nos. along long span and 8mm ϕ bars @ 200mm c/c, 3 nos. along the short span	**Providing 10mm ϕ bars @ 150 mm c/c in long span and 10mm ϕ bars @ 200 mm c/c in short span**
IS 456:2000 **Table 19** **IS 456:2000** **Table 20** **IS 456:2000** **Cl 40.2.1.1**	**10.**	**Check for shear** For short span Maximum shear, $V_{max} = \frac{wu*lx}{2} = \frac{9.34*3.50}{2}$ = 16.345 kN Nominal shear stress $\tau_v = \frac{Vmax}{b*d} = \frac{16.345*10^3}{1000*100}$ = 0.17 N/mm^2 Percentage tensile stress = $\frac{100*Ast}{b*d} = \frac{100*125.65}{100*1000}$ = 0.125 Maximum shear stress for concrete grade M25 = 3.1N/mm^2 Design shear strength of concrete for M25 concrete grade, τ_C = 0.29N/mm^2 For 125mm overall depth of slab k = 1.3 Design shear strength(τ') = τ_C.k = 0.377 N/mm^2>τ_V	**Okay**
	11.	**Check for deflection** Along short span, The basic value for continuous slab ; L/d = 26	

IS456:2000 **Cl 23.2.1** **Fig. 4** **IS456:2000** **CL 26.2.3.3**	**12.**	$f_s = 0.58 * f_y * \frac{Ast\ required}{Ast\ provided}$ $= 0.58 * 500 * \frac{151.97}{201.062}$ = 219.19 Modification factor = 1.8 $(L/d)_{allowable}$ = 26*1.8 = 46.8 $(L/d)_{actual} = \frac{3500}{100}$ = 35 $(L/d)_{actual} < (L/d)_{allowable}$ **Check for development length** MOR = $0.87 * f_y * A_{st} *(d- \frac{Ast*fy}{b*fck})$ $= 0.87 * 500 * 201.062*(100- \frac{251*500}{1000*25})$ =8.39 kN-m V_u = 16.345 kN-m Now, Anchorage value of bars, Lo = greater of 12 ϕ and effective depth=100mm $1.3\frac{M}{V} + Lo = 1.3\frac{8.39*10^6}{16.345*10^3}$+100=767.3mm And Development length of bars $Ld = \frac{0.87*fy*\phi}{4\tau_{bd}} = \frac{0.87*10*500}{4*1.6*1.4}$=485.49mm<767.3mm	**Safe** **Okay**

Design Summary

Overall depth = 125mm

- Reinforcement in the short span (at support) = 10mmϕ bars @ 250mm c/c
- Reinforcement in the long span (at support) =10mmϕ bars @ 250mm c/c
- Reinforcement in the short span(at mid span)=10mmϕbars@200mmc/c
- Reinforcement in the long span (at mid span) = 10mmϕ bars @ 300mm c/c
- Reinforcement along short span (at edge) =10mmϕ bars @ 200mm c/c
- Reinforcement along long span(at edge)=10mmϕbars@ 150mmc/c

Design of Beams

Detail Design of Powerhouse Beams

Beam Label: B3-C3 (Along X-X grid of second floor)

References	Steps	Calculations	Remarks
	1.	**Available Data:** Characteristic strength of Concrete(fck)= 25MPa Grade of Steel(fy)= 500 MPa Overall Depth of Beam (D)= 600mm Width of Beam (B)= 400mm Nominal cover= 35mm Taking 20mm Dia rebar, Effective Depth (d)= 600-25-20/2 = 565mm	**D= 600mm** **B= 400mm** **d= 565mm**
IS 13920: 2016 Cl.6.1		**Check for Axial Stress** Axial stress (A)= $\frac{P}{A}$= $\frac{0}{0.40*0.565}$= 0 <f_{ck}	**P/A<0.08 f_{ck} (OK)**
IS 13920: 2016 Cl. 6.1.1	**2.**	**Check for member size:** Depth of Beam (D)= 600 B/D= 400/600= 0.667 > 0.3 (OK)	**B/D> 0.3 (OK)**
IS 13920: 2016 Cl.6.1.2		Width of Beam (B)= 400mm > 200mm	**B> 200 (OK)**
IS 13920: 2016 Cl.6.1.3		Clear Length(L)= 7m D/L= 0.6/7= 0.086 (D< $\frac{1}{4} * L$) (OK)	**D< $\frac{1}{4} * L$ (OK)**
	3.	**From Numerical Modelling (SAP2000):** Maximum Hogging Moment at left end: -251.79 kNm Maximum Sagging Moment at mid span: 74.98 kNm Maximum Hogging moment at right end: -246.22 kNm	
IS 13920: 2016 Cl.6.2.1b	**4.**	**Check for Limiting Longitudinal Reinforcement:**	A_{st}**min= 542** mm^2

		Minimum Reinforcement A_{st} min= 0.24 $\frac{\sqrt{f_{ck}}}{f_y}$ * 400*565 = 542.4 mm^2	
IS 13920: 1993 Cl.6.2.2		Maximum Reinforcement, A_{st} = 0.025bd = 0.025*400*600 =5650 mm^2	**A_{st}max= 5650mm^2**
IS 456: 2000 Cl. 41.4.2.	**5.**	**Design for Flexure** M_{lim} =0.36σ_{ck} x_m b(d-0.42x_m) = 0.36*25*0.46*565*400(565-0.42*0.46*565) =426.504 kNm	**M_{lim}= 426.504 kNm**
		<u>At left support of beam</u> Maximum hogging moment (M_u), obtained from modelling= -229.79 kNm Since, $M_{lim} > M_u$, it is a singly reinforced section.	**Singly Reinforced Section**
IS 456: 2000 Annex G, G.1.1 c)		We have, $A_{st} = \frac{M_u}{0.87*d(1-\frac{A_{st}*fy}{b*d*fck})}$ Also,	
From SP 16: 1981 Table 3		$\frac{M_u}{bd^2}$=$\frac{251.79*10^6}{400*565^2}$ = 1.972 P= 0.505%	
		A_{st}=$\frac{P*bd}{100}$= $\frac{0.505*400*565}{100}$ On solving this, we get, Area of Steel (A_{st})= 1141.3 mm^2 Provide 4-20mm bars at top. Area of steel provided at top= 1256.6 mm^2 Provide 2-20mm bottom bars. Area of bottom bars provided= 628.3 mm^2	**Provide 4-20mm Bars at top. Also, provide 2-20mm Bars at bottom.**
		<u>At mid-span of the beam</u> Maximum sagging moment (M_u)= 74.98	**Singly**

From SP 16: 1981 **Table 3**		kNm Since, $M_{lim} > M_u$, it is a singly reinforced section. We have, M_u= 0.87$\sigma_y A_{st}$ (d-0.42x) A_{st}= $\frac{M_u}{0.87*d(1-\frac{A_{st}*fy}{b*d*fck})}$ $\frac{M_u}{bd^2}$=$\frac{74.98*10^6}{400*565^2}$ = 0.587 P= 0.139% A_{st}=$\frac{P*bd}{100}$= $\frac{0.139*400*565}{100}$ On solving this, we get, Area of Steel (A_{st})= 314.14mm^2 Provide 2-20mm rebars at bottom. Area of steel provided at bottom= 628.3 mm^2 Provide 2-20mm rebars at top. Area of steel provided at top= 628.3 mm^2	**Reinforced Section** **Provide 2-20mm at bottom and 2-20mm bars at top.**
From SP 16: 1981 **Table 3**	**6.**	**<u>At right support of beam</u>** Maximum hogging moment (M_u)= -246.22 kNm Since, $M_{lim} > M_u$, it is a singly reinforced section. We have, M_u= 0.87$\sigma_y A_{st}$ (d-0.42x) $\frac{M_u}{bd^2}$=$\frac{246.22*10^6}{400*565^2}$ = 1.93 P= 0.492% A_{st}=$\frac{P*bd}{100}$= $\frac{0.492*400*565}{100}$ On solving this, we get, Area of Steel (A_{st})= 1111.92mm^2 Provide 4-20mm Bars at top. Area of steel provided at top= 1256.6 mm^2	**Singly Reinforced Section** **Provide 4-20mm Bars at top. Also, provide 2-20mm rebars at bottom.**

IS 13920:2016 cl.6.2.1a)		Provide 2-20mm rebars at bottom. Area of steel provided at bottom= 628.3 mm^2	
IS 13920:2016 cl.6.2.1b) and cl.6.2.2		**Check for Reinforcement** Number of bars at top throughout the section = 2 no. of 20mm ≥ 12mm Number of bars at bottom throughout the section = 2 number of 20mm ≥12 mm (OK)	
IS 13920:2016 cl.6.2.3		A_{st} provided at any section is greater than $A_{st\ min}$ and less than $A_{st\ max.}$ (OK)	
IS 13920:2016 cl.6.2.5		Left support Half of negative steel = 628.3 mm^2 Positive steel = 628.3 $mm^2 \geq$ 628.3 mm^2 Mid-span Half of negative steel = 314.15 mm^2 Positive steel = 628.3 $mm^2 >$ 314.15 mm^2 Right support Half of negative steel = 628.3 mm^2 Positive steel = 628.3 $mm^2 \geq$ 628.3 mm^2 (OK)	
IS 456:2000 Cl. 23.2.1	**7**	Anchorage length at external joint = L_d (in tension) + 10φ - 8φ Development length (L_d)= $\frac{0.87*fy*\varphi}{4\tau bd}$ = 48.5 φ L_0= L_d+ 10Ø(FOS) - 8Ø (8Øfor 90° bend) = 970.982 + 2*20 = 1010.982mm Anchorage length= 1010.9mm for 20mm bar.	

IS 456:2000 Cl. 26.2.1	8	**Check for Deflection** Effective length (L)= 7+0.565= 7.565m d=0.565m L/d= 7.565/0.565= 13.389m $\propto$ = 26 (For Continuous) β= 1 (For span up to 10m) γ: f_s= 0.58*f_y * $\frac{Area\ of\ cross\ section\ of\ steel\ required}{Area\ of\ cross\ section\ of\ steel\ provided}$ Therefore, γ= 0.9 (***From Fig. 4 IS 456:2000)*** δ= 1.04 ***(From Fig. 5 IS 456:2000)*** Λ= 1 ***(From Fig. 6 IS 456:2000)*** Therefore, $\propto\ \beta\ \gamma\ \delta\Lambda$ = 24.336m > 13.389m **(OK)**	**Safe**
IS 456:20000 Table 20 IS 456:2000 Table 19 IS 456:2000 Table 20 IS 456:2000 Table 19 IS 456:2000 Table 20	9	**Check for Development Length** $L_d = \frac{0.87 f_y \emptyset}{4\tau_{bd}} = \frac{0.87*500*20}{4*1.4*1.6}$ = 970.982 mm M=0.87*f_y *A_{st}* d* (1-$\frac{A_{st}*f_y}{b*d*f_{ck}}$)= 0.87*500*1030.412*565* (1-$\frac{1030.412*500}{400*565*25}$)= 230.156 kNm L_d= 1.3*$\frac{M}{V_u}$+ L_0= 1.3* $\frac{230.156}{206.7}$ + 0 = 1451.34mm > 970.982mm. **(OK)** **Design for Shear Reinforcement:** We have, Tensile steel provided at left end=0.556% σ_{ck} =25 τ_{cmax} = 3.1 N/mm^2 Permissible shear strength of concrete, τ_c=0.51 N/mm^2 Design shear strength of concrete $\tau_c bd$= (0.51*400*565)/1000= 114.79 kN Tensile steel provided at right end=0.556% σ_{ck} =25 τ_{cmax} = 3.1 N/mm^2 Permissible shear strength of concrete, τ_c=0.51 N/mm^2	**Safe**

IS 456:2000 Table 19		Design shear strength of concrete $\tau_c bd$= (0.51*400*565)/1000= 114.79 kN Tensile steel provided at mid span= 0.262% σ_{ck} =25 τ_{cmax} = 3.1 N/mm^2 Permissible shear strength of concrete, τ_c= 0.366 N/mm^2 Design shear strength of concrete $\tau_c bd$= (0.385*400*565)/1000= 87.01 kN **Spacing of Stirrups**	
IS 456: 2000 Cl. 40.4 **IS 13920:1993 Cl. 6.3.5**		At left end, Factored Shear Force from analysisV_u= 206.7 kN Nominal Shear Stress (τ_u)=0.915 N/mm^2 Design Shear Stress (τ_c)=0.453 N/mm^2 As, $\tau_u > \tau_c < \tau_{cmax}$ Therefore, provide proper shear reinforcement. Here, V_{us}= ($\tau_u - \tau_c$)*b*d= (0.914-0.55) *400*565 = 104322 kN. Taking 2-legged, 8mm Ø stirrups. Area of stirrup (A_{sv}) = 2*50.27 mm^2 = 100.54 mm^2 $S_v = \frac{0.87\ \sigma_y\ A_{sv}\ d}{V_{us}} = \frac{0.87*415*100.54*565}{104322} =$ 196.57mm We know, $S_v$should be $\leq \frac{d}{4}$ and $8d_b$ i.e. $\leq \frac{565}{4}$ = 141.25 mm and 8*20= 160mm Hence, take S_v = 100mm. Provide 2-Legged stirrups at 100mm spacing.	**Provide 2-Legged 8mm stirrups at 100mm spacing.**
IS 456: 2000 Cl. 26.5.1.6		At mid span, Factored Shear Force from analysis,V_u= 40.5 kN Nominal Shear Stress (τ_u)=0.179 N/mm^2 Design Shear Stress (τ_c)= 0.385 N/mm^2 As, $\tau_u < \tau_c < \tau_{cmax}$ Therefore, provide nominal shear reinforcement.	

IS 13920:1993 Cl. 6.3.5 **IS 456: 2000 Cl. 40.4** **IS 13920:1993 Cl. 6.3.5**		Here, Taking 2-legged, 8mm Ø stirrups. Area of stirrup (A_{sv}) = 2*50.27 mm^2 = 100.54 mm^2 $$\frac{A_{sv}}{bs_v} \geq \frac{0.4}{0.87\ f_y}$$ $$s_v \leq \frac{A_{sv} * 0.87\ f_y}{0.4 * b}$$ Taking 2-legged, 8mm Ø stirrups. Area of stirrup (A_{sv}) = 2*50.27 mm^2 = 100.54 mm^2 $s_v \leq \frac{100.54*0.87*415}{400*0.4} \leq$ 226.85mm We know, $S_v$should be $\leq \frac{d}{2}$ i.e. $\leq \frac{565}{2}$ = 282.5 mm. Hence, take S_v = 200 mm. Provide 2-Legged stirrups at 200mm spacing. <u>At right end</u>, Factored Shear Force from analysisV_u= 206.7 kN Nominal Shear Stress (τ_u)=0.915 N/mm^2 Design Shear Stress (τ_c)= 0.453 N/mm^2 As, τ_u>τ_c<τ_{cmax} Therefore, provide proper shear reinforcement. Here, V_{us}= $(\tau_u - \tau_c)$*b*d= (0.914-0.55) *400*565 = 104322 kN. Taking 2-legged, 8mm Ø stirrups. Area of stirrup (A_{sv}) = 2*50.27 mm^2 = 100.54 mm^2 S_v= $\frac{0.87\ \sigma_y\ A_{sv}\ d}{V_{us}} = \frac{0.87*415*\ 100.54*565}{104322}$ = 196.57mm We know, $S_v$should be $\leq \frac{d}{4}$ and $8d_b$	**Provide 2-Legged 8mm stirrups at 200mm spacing (c/c).** **Provide 2-Legged 8mm stirrups at 100mm spacing(c/c).**

		i.e. $\leq \frac{565}{4}$ = 141.25 mm and 8*20= 160mm Hence, take S_v = 100mm. Provide 2-Legged stirrups at 100mm spacing.	

Design Summary:

Width of beam= 400mm
Overall depth= 600mm
Cover= 35mm

- Length of Beam=7 m
- Reinforcement provided at top of the beam at left end= 4-20mm ∅ rebars.
- Reinforcement provided at bottom of the beam at left end= 2-20mm ∅ rebars.
- Reinforcement provided at top of the beam at mid span= 2-20mm ∅ rebars.
- Reinforcement provided at top of the beam at mid span= 2-20mm ∅ rebars.
- Reinforcement provided at top of the beam at right end= 4-20mm ∅ rebars.
- Reinforcement provided at bottom of the beam at right end= 2-20mm ∅ rebars.
- Shear reinforcement at supports= 2 Legged, 8mm stirrup @ 100mm c/c.
- Shear reinforcement at mid span=2 Legged, 8mm stirrup @ 200mm c/c.
- Anchorage length= 1010mm

Detail Design of Control Room Beams
Beam Label: N6-O6 (Along X-X grid of Control Room)

References	Steps	Calculations	Remarks
	1.	**Available Data:** Characteristic strength of Concrete(fck)= 25MPa Grade of Steel(fy)= 500 MPa Overall Depth of Beam (D)= 400mm Width of Beam (B)= 250mm Nominal cover= 35mm Taking 20mm Dia rebar, Effective Depth (d)= 400-25-20/2 = 365mm	**D= 400mm** **B= 250mm** **d= 365mm**
IS 13920: 2016 Cl.6.1	**2.**	**Check for Axial Stress**	**P/A<0.08 f_{ck}**

		Axial stress (A)= $\frac{P}{A}$= $\frac{0}{0.25*0.365}$= 0 <f_{ck}	**(OK)**
IS 13920: 2016 Cl. 6.1.1 **IS 13920: 2016 Cl.6.1.2** **IS 13920: 2016 Cl.6.1.3**	**3.**	**Check for member size:** Depth of Beam (D)= 400 B/D= 250/400= 0.625 > 0.3 (OK) Width of Beam (B)= 250mm > 200mm Clear Length(L)= 6.5m D/L= 0.4/6.5= 0.0615 (D< $\frac{1}{4} * L$) (OK)	**B/D> 0.3 (OK)** **B> 200 (OK)** **D<** $\boldsymbol{\frac{1}{4} * L}$ **(OK)**
	4.	**From Numerical Modelling (SAP2000):** Maximum Hogging Moment at left end: -60.521kNm Maximum Sagging Moment at mid span: 71 kNm Maximum Hogging moment at right end: -104.896 kNm	
IS 13920: 2016 Cl.6.2.1b **IS 13920: 1993 Cl.6.2.2** **IS 456: 2000 Cl. 41.4.2.**	**5.**	**Check for Limiting Longitudinal Reinforcement:** Minimum Reinforcement A_{st}min= 0.24 $\frac{\sqrt{f_{ck}}}{f_y} *$ 250*365 = 219 mm^2 Maximum Reinforcement, A_{st}= 0.025bd = 0.025*250*365 =2281.25 mm^2	$\boldsymbol{A_{st}}$**min= 219** $\boldsymbol{mm^2}$ $\boldsymbol{A_{st}}$**max= 2281.25**$\boldsymbol{mm^2}$ $\boldsymbol{M_{lim}}$**= 426.504 kNm**

	6.	**Design for Flexure** M_{lim} = 0.36$\sigma_{ck}x_m$b(d-0.42x_m) = 0.36*25*0.46*56*400(565-0.42*0.46*565) = 111.247 kNm	**Singly Reinforced Section**
IS 456: 2000 Annex G, G.1.1 c) **From SP 16: 1981 Table 3**	**7.**	**<u>At left support of beam</u>** Maximum hogging moment(M_u), obtained from modelling= -60.521 kNm Since, $M_{lim} > M_u$, it is a singly reinforced section. We have, $A_{st}=\frac{M_u}{0.87*d(1-\frac{A_{st}*fy}{b*d*fck})}$ Also, $\frac{M_u}{bd^2}=\frac{60.521*10^6}{250*365}$ = 1.817 P= 0.46% $A_{st}=\frac{P*bd}{100}=\frac{0.46*250*365}{100}$ On solving this, we get, Area of Steel (A_{st})= 420.44mm^2 Provide 2-16mm and 1-20mm rebars at top.Area of steel provided at top= 716.62 mm^2 Provide 3-16mm rebars at bottom. Area of rebars provided at bottom=603.168 mm^2	**Provide 2-16mm and 1-20mm rebars at top. Also, provide 3-16mm rebars at the bottom.**
From SP 16: 1981 Table 3		**<u>At mid-span of the beam</u>** Maximum sagging moment(M_u)= 70.821 kNm Since, $M_{lim} > M_u$, it is a singly reinforced section. We have, $A_{st}=\frac{M_u}{0.87*d(1-\frac{A_{st}*fy}{b*d*fck})}$ $\frac{M_u}{bd^2}=\frac{70.821*10^6}{250*365^2}$ = 2.126	**Singly Reinforced Section** **Provide 3-16mm rebars at bottom and 2-16mm rebars at**

<table>
<tr><td></td><td></td><td>P= 0.55%

$A_{st}=\frac{P*bd}{100}=\frac{0.55*250*365}{100}$
On solving this, we get,
Area of Steel (A_{st})= 501.875mm^2

Provide 3-16mm rebars at bottom.
Area of steel provided at bottom= 603.168 mm^2

Provide 2-16mm rebars at top.
Area of steel provided at top= 402.112 mm^2</td><td>top.</td></tr>
<tr><td>From SP 16: 1981
Table 3

IS 13920:2016 cl.6.2.1a)</td><td></td><td><u>At right support of beam</u>

Maximum hogging moment(M_u)= -104.89 kNm
Since, $M_{lim} > M_u$, it is a singly reinforced section.

We have,
M_u= 0.87$\sigma_y A_{st}$ (d-0.42x)

$\frac{M_u}{bd^2}=\frac{104.89*10^6}{250*365^2}$ = 3.15

P= 0.880%

$A_{st}=\frac{P*bd}{100}=\frac{0.880*250*365}{100}$
On solving this, we get,
Area of Steel (A_{st})= 807.5678mm^2

Provide 2-20mm and 2-16mm rebars at top.
Area of steel provided at top= 1030.412 mm^2

Provide 3-16mm rebars at bottom.
Area of steel provided at bottom=603.168 mm^2</td><td>Provide 2-20mmand 2-16mm rebars at top. Also, provide 3-16mm rebars at bottom.</td></tr>
<tr><td>IS 13920:2016 cl.6.2.1b) and cl.6.2.2</td><td>8.</td><td>Check for Reinforcement
Number of bars at top throughout the section = 2 no. of 16mm ≥ 12mm
Number of bars at bottom throughout the section = 3 number of 16mm ≥12 mm (OK)</td><td></td></tr>
</table>

IS 13920:2016 cl.6.2.3 **IS 13920:2016 cl.6.2.5** **IS 456:2000 Cl. 23.2.1** **IS 456:2000 Cl. 26.2.1**	 **9.** **10.**	A_{st}provided at any section is greater than $A_{st\ min}$and less than $A_{st\ max}$. (OK) Left support Half of negative steel = 358.31 mm^2 Positive steel = 603.168 mm^2> 358.31 mm^2 Mid-span Half of negative steel = 301.584 mm^2 Positive steel = 603.168 mm^2>301.584 mm^2 Right support Half of negative steel = 515.206 mm^2 Positive steel = 603.168 mm^2>= 515.206 mm^2 (OK) Anchorage length at external joint = L_d (in tension) + 10φ - 8φ Development length (L_d)= $\frac{0.87*fy*\varphi}{4\tau bd}$ = 48.5 φ L_0= L_d+ 10∅ - 8∅ (8∅for 90° bend) = 970.982 + 2*20 = 1010.982 Anchorage length= 1010.9mm for 20mm bar **Check for Deflection** Effective length (L)= 6.5+0.365= 6.865m d=0.365m L/d= 6.865/0.565= 18.808m ∝ = 26 (For Continuous) β= 1 (For span up to 10m) γ: f_s= 0.58*f_y * $\frac{Area\ of\ cross\ section\ of\ steel\ required}{Area\ of\ cross\ section\ of\ steel\ provided}$ Therefore, γ= 0.82 (***From Fig. 4 IS 456:2000)*** δ= 1.17 ***(From Fig. 5 IS 456:2000)*** Λ= 1 ***(From Fig. 6 IS 456:2000)*** Therefore, ∝ $\beta\ \gamma\ \delta$Λ = 24.944m >18.808m **(OK)** **Check for Development Length**	 **Safe**

		$L_d = \frac{0.87 f_y \emptyset}{4\tau_{bd}} = \frac{0.87*500*20}{4*1.4*1.6}$ = 970.982 mm M=0.87*f_y *A_{st}* d* (1-$\frac{A_{st}*f_y}{b*d*f_{ck}}$)= 0.87*500*1030.412*365* (1-$\frac{1030.412*500}{250*365*25}$)= 126.65 kNm L_d= 1.3*$\frac{M}{V_u}$+ L_0= 1.3* $\frac{126.65}{138.953}$ + 0 = 1184.9mm > 970.982mm. **(OK)**	**Safe**
IS 456:20000 **Table 20** **IS 456:2000** **Table 19** **IS 456:2000** **Table 20** **IS 456:2000** **Table 19** **IS 456:2000** **Table 20** **IS 456:2000** **Table 19**	**11.**	**Check for Shear Reinforcement:** We have, Tensile steel provided at left end=0.785% σ_{ck} =25 τ_{cmax} = 3.1 N/mm^2 Permissible shear strength of concrete, τ_c=0.6 N/mm^2 Design shear strength of concrete $\tau_c bd$= (0.6*250*365)/1000= 55.08 kN Tensile steel provided at right end=1.129% σ_{ck} =25 τ_{cmax} = 3.1 N/mm^2 Permissible shear strength of concrete, τ_c=0.67 N/mm^2 Design shear strength of concrete $\tau_c bd$= (0.67*250*365)/1000= 61.14 kN Tensile steel provided at mid span= 0.688% σ_{ck} =25 τ_{cmax} = 3.1 N/mm^2 Permissible shear strength of concrete, τ_c= 0.55 N/mm^2 Design shear strength of concrete $\tau_c bd$= (0.55*250*365)/1000= 50.18 kN	
	12.	**Spacing of Stirrups** <u>At left end</u>, Factored Shear Force from analysisV_u= 117.15 kN Nominal Shear Stress (τ_u)=1.284 N/mm^2 Design Shear Stress (τ_c)= 0.55 N/mm^2 As, τ_u>τ_c<τ_{cmax} Therefore, provide proper shear reinforcement. Here, V_{us}= ($\tau_u - \tau_c$)*b*d= (1.284-0.55)*250*365	

IS 456: 2000 **Cl. 40.4** **IS 13920:1993** **Cl. 6.3.5**		= 66962.5 kN. Taking 2-legged, 8mm Ø stirrups. Area of stirrup (A_{sv}) = 2*50.27 mm^2 = 100.54 mm^2 $S_v = \frac{0.87\ \sigma_y\ A_{sv}\ d}{V_{us}} = \frac{0.87*415*100.54*365}{104322} =$ 197.864mm We know, $S_v$should be ≤ $\frac{d}{4}$ and 8d_b i.e. ≤ $\frac{365}{4}$ = 91.25 mm and 8*20= 160mm Hence, take S_v = 100mm. Provide 2-Legged stirrups at 100mm spacing.	**Provide 2-Legged 8mm stirrups at 100mm spacing.**
IS 456: 2000 **Cl. 26.5.1.6**		<u>At mid span</u>, Factored Shear Force from analysis,V_u= 15.145 kN Nominal Shear Stress (τ_u)=0.166 N/mm^2 Design Shear Stress (τ_c)= 0.55 N/mm^2 As, $\tau_u < \tau_c < \tau_{cmax}$ Therefore, provide minimum shear reinforcement. Here, Taking 2-legged, 8mm Ø stirrups. Area of stirrup (A_{sv}) = 2*50.27 mm^2 = 100.54 mm^2 $$\frac{A_{sv}}{bs_v} \geq \frac{0.4}{0.87\ f_y}$$ $$s_v \leq \frac{A_{sv} * 0.87\ f_y}{0.4 * b}$$ Taking 2-legged, 8mm Ø stirrups. Area of stirrup (A_{sv}) = 2*50.27 mm^2 = 100.54 mm^2	
IS 13920:1993 **Cl. 6.3.5**		$s_v \leq \frac{100.54*0.87*415}{250*0.4} \leq$ 362.95mm We know, $S_v$should be ≤ $\frac{d}{2}$ i.e. ≤ $\frac{365}{2}$ = 182.5 mm.	**Provide 2-Legged 8mm stirrups at 180mm spacing (c/c).**

IS 456: 2000 **Cl. 40.4** **IS 13920:1993** **Cl. 6.3.5**		Hence, take S_v = 180 mm. Provide 2-Legged stirrups at 180mm spacing. <u>At right end</u>, Factored Shear Force from analysisV_u= 138.952 kN Nominal Shear Stress (τ_u)=1.523 N/mm^2 Design Shear Stress (τ_c)= 0.67 N/mm^2 As, τ_u>τ_c<τ_{cmax} Therefore, provide proper shear reinforcement. Here, V_{us}= ($\tau_u - \tau_c$)*b*d= (1.523-0.67)*250*365 = 77814.5 kN. Taking 2-legged, 8mm Ø stirrups. Area of stirrup (A_{sv}) = 2*50.27 mm^2 = 100.54 mm^2 $S_v = \frac{0.87\ \sigma_y\ A_{sv}\ d}{V_{us}} = \frac{0.87*415*100.54*365}{77814.5} =$ 170.25mm We know, $S_v$should be ≤ $\frac{d}{4}$ and 8d_b i.e. ≤ $\frac{365}{4}$ = 91.25 mm and 8*20= 160mm Hence, take S_v = 100mm. Provide 2-Legged stirrups at 100mm spacing.	**Provide 2-Legged 8mm stirrups at 100mm spacing(c/c).**

Design Summary:

Width of beam= 250mm
Overall depth= 400mm
Cover= 35mm
Length of Beam = 6.5m

- Reinforcement provided at top of the beam at left end= 2-16mm Ø rebars and 1-20mm Ø rebars.
- Reinforcement provided at bottom of the beam at left end= 3-16mm Ø rebars.
- Reinforcement provided at top of the beam at mid span= 2-16mm Ø rebars.
- Reinforcement provided at bottom of the beam at mid span= 3-16mm Ø rebars.
- Reinforcement provided at top of the beam at right end= 2-20mm Ø rebars and 2-16mm Ø rebars.
- Reinforcement provided at bottom of the beam at right end= 3-16mm Ø rebars.

- Shear reinforcement at supports= 2 Legged, 8mm stirrup @ 100mm c/c.
- Shear reinforcement at midspan=2 Legged, 8mm stirrup @ 180mm c/c.
- Anchorage length= 1010m

Design of Column:

Column C2 below the Gantry Girder:

Reference	Steps	Calculations	Remarks
	1.	**Known data** From Preliminary design of column Depth (D) = 1000 mm Width (b) = 1000 mm Depth of beam (d) = 600 mm Storey height (h) = 4000 mm Unsupported length of column (L) = 3400 mm From Sap2000 model analysis of the structure Axial load (Pu) = 2193.52 KN Moment (M_{ux}) = 74.090 KN-m Moment (M_{uy}) = 59.736 KN-m Assumptions: Effective Cover (d’) = 50 mm	
IS 456:2000 Table 28	**2.**	Effective length of compression member ($Leff$) = 0.65 * L = 2210 mm	
IS 456:2000 Clause 25.1.2	**3.**	**Check for Slenderness ratio** $\frac{Leff}{D} = \frac{Leff}{b} = \frac{2210}{800}$ = 2.76 < 12	**To be designed as short column**
IS 13920:2016 Clause 7.1	**4.**	**Check for Axial stress** Axial stress = $\frac{Pu}{Ag} = \frac{2193.52*10^3}{1000*1000}$ = 2.25 N/mm^2 0.08*f_{ck} = 0.08*25 = 2 N/mm^2	**Axial stress > 0.08*f_{ck} Thus, OK**
IS 13920:2016 Clause 7.2	**5.**	**Check for member size** D=b= 1000mm > 300mm Aspect ratio = 1000/1000 = 1 > 0.45	**OK**

IS 456:2000 Clause 26.5.3.1	**6.**	**Limiting Longitudinal Reinforcement** Minimum Reinforcement = 0.8%*bD = 8000 mm^2 Maximum Reinforcement = 4% *bD = 40000 mm^2	
IS 456:2000 Clause 25.4	**7.**	**Check for minimum eccentricity** $e_{min} = \frac{L}{500} + \frac{D_{least}}{30}$ $e_{x,min} = e_{y,min}$ = 3400/800 + 1000/30 = 40.133 mm > 20mm	**Minimum eccentricity is taken as 40.133 mm. To be designed as bi-axially loaded column.**
	8.	**Design of Longitudinal Reinforcement** Moments form the Sap analysis M_{ux1} = 70.090 KNm M_{uy1}= 59.736 KNm Moments due to min. eccentricity $M_{ux2} = P_u * e_{x,min}$ =2193.52*40.133*10^{-3} = 88.033 KNm $M_{uy2} = P_u * e_{y,min}$ =2193.52*40.133*10^{-3} = 88.033 KNm Design moments Taking maximum of (Mx1,Mx2) and (My1,My2), M_{ux} = 88.033KNm M_{uy} = 88.033 KNm The reinforcement is to be designed equally on all four sides. d'/D = 50/1000 = 0.05 d'/b = 50/1000 = 0.05 Thus, Chart 48 of SP 16 is used for checking the trial areas for longitudinal reinforcement. $\frac{P_u}{f_{ck}bD} = \frac{2193.52*10^3}{25*1000*1000}$ = 0.088 Take p% = 1 $\frac{p}{f_{ck}} = \frac{1}{25}$ = 0.04	

IS 456:2000 Clause 39.6 **IS 456:2000 Clause 39.6**		From Chart 48 of SP 16; $\frac{M_{ux,i}}{f_{ck}bD^2} = \frac{M_{uy,i}}{f_{ck}Db^2}$ = 0.08 $M_{ux,i} = M_{uy,i}$ = 2000 KNm $P_{uz} = 0.45 f_{ck} * A_c + 0.75 f_y * A_{st}$ =0.45*25*0.99*1000^2+0.75*500*0.001*1000^2 = 14887.500 kN Then, $\frac{P_u}{P_{uz}} = \frac{2193.52}{14877.50}$= 0.147 As per Clause 39.6 of IS 456:2000 α_n=1.023 for 0.8 $\frac{P_u}{P_{uz}}$> 0.8 Now, $(\frac{Mux}{Mux,i})^{\alpha n}+(\frac{Muy}{Muy,i})^{\alpha n}$ = 0.116< 1 Thus, design ok. Area of longitudinal steels required = 1 % * 1000 * 1000 = 10000 mm^2 **So, provide 12-32⌀+ 4-25⌀ bars.** Therefore, area of steel provided (A_p) = 9817 mm^2	
IS 13920:2016 Clause 7.5 **IS 456:2000 Clause 40.2.2**	**9.**	**Design of Shear** The design shear for column must be larger of the followings: **I.** Factored shear force as per linear structure analysis; **Vu= 129.932 kN** Now, Area of steel provided (A_p) = 9817 mm^2 Design Axial load (P_u) = 2193.52 KN $\delta = 1 + \frac{3P_u}{A_g f_{ck}}$ but not exceeding 1.5 = 1.263 Thus, Adopt δ=1.26 $100\frac{A_s}{bd} = 100\frac{9817}{bd} = 0.982$ From Table 19 of IS 456:2000 Shear strength (τ_c) = 0.634 N/mm^2 $\tau_{c\delta}$ = 0.634*1.26 = 0.801 N/mm^2 Then, $V_{uc} = bD\tau_{c\delta}$ = 1000*1000*0.801*10^{-3} = 506.927 KN >V_d	

		Thus, minimum shear reinforcement is to be provided in the form of lateral ties.	
IS 456:2000As Clause 26.5.3.2 **IS 456:2000 Clause26.5.3.2** **IS 13920:2016 Clause 7.4.2** **IS 456:2000 Clause 26.5.3.2**	**10.**	**Design of Transverse Reinforcement** Diameter of lateral ties (⌀tr) shall not be less than i. (1/4) *20 = 5 ii. 6mm Thus, **Adopt ⌀tr as 8mm.** Pitch of lateral ties is least of i. 1000mm ii. 16*32 = 512mm iii. 300mm iv. 48*8 = 384 mm **The maximum spacing of lateral ties should be 250mm center to center.** Thus, provide 8mm lateral ties @25mm spacing Spacing between longitudinal bars= (1000 – 50*2)/3 = 300mm>75mm Thus, Additional ties to be provided. Spacing between corner longitudinal bars= 1000-2*50 = 900mm 48⌀tr = 48*8 = 384mm 900mm>48⌀tr Thus, longitudinal bars to be ties by close ties.	
IS 13920:2016 Clause 8.1	**11.**	**Design of Special Confinement** Spacing confinement should be provided up to the length (Lo) which is not less than i. 1000mm ii. 1/6 *2210 = 368.33mm iii. 450mm Thus, provide confining rebar for length Lo=1000mm	

IS 13920:2016 Clause 8.2 **IS 13920:2016 Clause 8.1**		Special confining reinforcement shall extend at least 300mm into the footing or mat. $$A_{sh} = 0.81 S_v h \frac{f_{ck}}{f_y} (\frac{A_g}{A_k} - 1)$$ Where, $A_k = Dk^2$ D_k = 1000-2*(50-(20/2))+2*8 = 941mm A_k = 941^2 = 885481mm^2 Ash = $\frac{\pi}{4}8^2$ = 50.265mm^2 h = $Dk/2$ = 941/2 = 470.5mm Then, 50.625=0.81*S_v*470.5*$\frac{25}{500}$ $(\frac{1000*1000}{941*941} - 1)$ S_v =91.785mm Spacing of special confinement should be least of i. (1/4) *1000 = 250mm ii. 6*20= 120mm iii. 100mm Thus, provide special confining reinforcement for length of 1000mm 8mm 5 legged hoops in both directions.	
IS 13920:2016 Clause 7.3.2.1	**12.**	**Splicing of Longitudinal Reinforcement** Not more than 50% of the longitudinal bars should be spliced at one section and lap splices should be provided only in the middle half of clear column height. $\frac{h_c}{4} = \frac{4000}{4}$ = 1000mm Provide 8mm diameter links@250mm c/c for a distance of 1000mm on either side of lap splice. **Provide 8mm diameter links@100mm c/c for a distance of 1000mm lap splice.**	
IS 456:2000 Clause 26.2.5.1	**13.**	Lap length of Longitudinal Bars Lap length should be greater of i. Development length $L_d = \frac{\emptyset\, \sigma_s}{4\tau_{bd}}$ $= \frac{32*0.87*500}{4*1.4*1.6*1.25}$ = 932.50mm ii. 24∅ = 24*32= 768mm Thus, adopt clear lap length as 932.50 mm	

Control Room Column (C31)

Reference	Steps	Calculations	Remarks
	1.	**Known data** From Preliminary design of column Depth (D) = 400 mm Width (b) = 400 mm Depth of beam (d) = 400 mm Storey height (h) = 4500 mm Unsupported length of column (L) = 4100 mm From Sap2000 model analysis of the structure Axial load (Pu) = 421.22 KN Moment (M_{ux}) =44.875KN-m Moment (M_{uy}) = 15.668 KN-m Assumptions: Effective Cover (d') = 50 mm	
IS 456:2000 Table 28	**2.**	Effective length of compression member ($Leff$) = 0.65 * L = 2665 mm	
IS 456:2000 Clause 25.1.2	**3.**	**Check for Slenderness ratio** $\frac{Leff}{D} = \frac{Leff}{b} = \frac{2665}{400}$ = 6.66 < 12	**To be designed as short column**
IS 13920:2016 Clause 7.1		**Check for Axial stress** Axial stress = $\frac{Pu}{Ag} = \frac{421.22*10^3}{400*400}$ = 2.632 N/mm^2 0.08*f_{ck} = 0.08*25 = 2 N/mm^2	**Axial stress > 0.08*f_{ck}. OK**
IS 13920:2016 Clause 7.2	**4.**	**Check for member size** D=b= 400mm > 300mm Aspect ratio = 400/400 = 1 > 0.45	**OK**
IS 456:2000 Clause 26.5.3.1	**5.**	**Limiting Longitudinal Reinforcement** Minimum Reinforcement = 0.8%*bD = 1280 mm^2 Maximum Reinforcement = 4% *bD = 6400mm^2	

IS 456:2000 Clause 25.4	**6.**	**Check for minimum eccentricity** $e_{min} = \frac{L}{500} + \frac{D_{least}}{30}$ $e_{x,min} = e_{y,min}$ = 4600/800 + 400/30 = 21.533 mm > 20mm	**Hence, minimum eccentricity is taken as 33.467 mm. To be designed as bi-axially loaded column.**
                                    IS 456:2000 Clause 39.6	**7.**	**Design of Longitudinal Reinforcement** Moments form the Sap analysis M_{ux1} = 44.875 KNm M_{uy1}= 15.668KNm Moments due to min. eccentricity $M_{ux2} = P_u * e_{x,min}$ = 221.18*22.533*10^{-3} = 9.070 KNm $M_{uy2} = P_u * e_{y,min}$ = 221.18*22.533*10^{-3} = 9.070 KNm Design moments Taking maximum of (Mx1, Mx2) and (My1, My2), M_{ux} = 44.875KNm M_{uy} = 15.668 KNm The reinforcement is to be designed equally on all four sides. d'/D = 50/400 = 0.125 d'/b = 50/400 = 0.125 Thus, Chart 48 of SP 16 is used for checking the trial areas for longitudinal reinforcement. $\frac{P_u}{f_{ck}bD} = \frac{421.22*10^3}{25*400*400}$ = 0.105 Take p% = 1% $\frac{p}{f_{ck}} = \frac{1}{25}$ = 0.04 From Chart 48 of SP 16; $\frac{M_{ux,i}}{f_{ck}bD^2} = \frac{M_{uy,i}}{f_{ck}Db^2}$ = 0.8 $M_{ux,i} = M_{uy,i}$ = 1280 KNm $P_{uz} = 0.45f_{ck} * A_c + 0.75f_y * A_{st}$	

IS 456:2000 Clause 39.6		$=0.45*25*0.975*400^2+0.75*500*0.025*400^2$ = 2382.00 Then, $\frac{P_u}{P_{uz}} = \frac{421.22}{2382.00}$ = 0.177 As per Clause 39.6 of IS 456:2000 α_n=1.023 for 0.8 $\frac{P_u}{P_{uz}}$> 0.8 Now, $(\frac{Mux}{Mux,i})^{\alpha n}+(\frac{Muy}{Muy,i})^{\alpha n}$ = 0.054< 1 Thus, design ok. Area of longitudinal steels required = 1 % * 400 * 400 = 1600 mm^2 So, provide 8-16⌀ bars. Therefore, area of steel provided (A_p) = 1608.495 mm^2	
IS 13920:2016 Clause 7.5 **IS 456:2000 Clause 40.2.2**	**8.**	**Design of Shear** The design shear for column must be larger of the followings: Factored shear force as per linear structure analysis; **Vu= 94.458 KN** Now, Area of steel provided (A_p) =1600 mm^2 Design Axial load (P_u) = 421.22 KN $\delta = 1+\frac{3P_u}{A_g f_{ck}}$ but not exceeding 1.5 = 1.316 Thus, Adopt δ=1.3 $100\frac{A_s}{bd} = 100\frac{5026}{bd} = 1.005$ From Table 19 of IS 456:2000 Shear strength (τ_c) = 0.863 N/mm^2 $\tau_{c\delta}$ = 0.581*1.3 = 1.136 N/mm^2 Then, $V_{uc} = bD\tau_{c\delta}$ $= 400*400*1.136*10^{-3}$ = 181.702 KN >V_d Thus, minimum shear reinforcement is to be provided in the form of lateral ties.	

	9.	**Design of Transverse Reinforcement**	
IS 456:2000As Clause 26.5.3.2		Diameter of lateral ties (⌀tr) shall not be less than iii. (1/4) *20 = 5 iv. 6mm Thus, Adopt ⌀tr as 8mm. As per Clause 26.5.1.6 of IS 456:2000 For minimum shear reinforcement considering 8mm ⌀ 4 legged lateral ties. Spacing of lateral ties; $S_v = \frac{0.87 f_y A_{sv}}{0.4b}$ $= \frac{0.87*400*3*50.265}{0.4*400}$ = 409.978 mm Where, A_{sv}=4* $\frac{\pi}{4}8^2$	
IS 456:2000 Clause26.5.3.2		Pitch of lateral ties is least of v. 400mm vi. 16*8 = 150mm vii. 300mm	
IS 13920:2016 Clause 7.4.2		The maximum spacing of lateral ties should be 150mm center to center. Thus, provide 8mm lateral ties @150mm spacing	
IS 456:2000 Clause 26.5.3.2		Spacing between longitudinal bars= (400 – 50*2)/3 = 100 >75mm Thus, Additional ties to be provided. Spacing between corner longitudinal bars= 400-2*50 = 300mm 48⌀tr = 48*8 = 384mm 300mm>48⌀tr Thus, longitudinal bars to be ties by close ties.	

IS 13920:2016 Clause 8.1 **IS 13920:2016 Clause 8.2** **IS 13920:2016 Clause 8.1**	**10.**	**Design of Special Confinement** Spacing confinement should be provided up to the length (Lo) which is not less than iv. 400mm v. 1/6 *2665 = 444mm vi. 450mm Thus, provide confining rebar for length Lo=450mm Spacing of special confinement should be least of iv. (1/4)*800 = 200mm v. 6*16= 100mm vi. 100mm Thus, provide special confining reinforcement for length of 1100mm 8mm 3 legged hoops in both directions.	
IS 13920:2016 Clause 7.3.2.1	**11.**	**Splicing of Longitudinal Reinforcement** Not more than 50% of the longitudinal bars should be spliced at one section and lap splices should be provided only in the middle half of clear column height. $\frac{h_c}{4} = \frac{4100}{4}$ = 1025mm Take 1200 mm Provide 8mm diameter links@150mm c/c for a distance of 1200mm on either side of lap splice. **Provide 8mm diameter links@100mm c/c for a distance of 800mm lap splice.**	
IS 456:2000 Clause 26.2.5.1	**12.**	Lap length of Longitudinal Bars Lap length should be greater of iii. Development length $L_d = \frac{\emptyset\, \sigma_s}{4\tau_{bd}}$ $= \frac{16*0.87*500}{4*1.4*1.6*1.25}$ = 721.429mm iv. 24∅ = 24*16 = 384mm Thus, adopt clear lap length as 776.8 mm	

Design Summary:

For column below the gantry girder, we use **12 no. of 32mm**and **4 no. of 25mm** bars.

For lateral ties,

- 8mm bars spaced at 100mm c/c for confinement length of 1300mm.
- 8mm bars spaced at 150mm c/c at column and beam intersection.
- 8mm bars spaced at 250mm c/c at compression zone.
- 8mm bars spaced at 100mm c/c at lapping zone.
- Lapping distance is 1000mm.

For column above the gantry girder we use **12 no. of 25mm** bars.

For lateral ties,

- 8mm bars spaced at 100mm c/c for confinement length of 1200mm.
- 8mm bars spaced at 150mm c/c at column and beam intersection.
- 8mm bars spaced at 250mm c/c at compression zone.
- 8mm bars spaced at 100mm c/c at lapping zone.
- Lapping distance is 1000mm.

For column of Control Unit, we use **8 no. of 16mm bars**

For lateral ties,

- 8mm bars spaced at 100mm c/c for confinement length of 1100mm.
- 8mm bars spaced at 150mm c/c at column and beam intersection.
- 8mm bars spaced at 200mm c/c at compression zone.
- 8mm bars spaced at 100mm c/c at lapping zone.
- Lapping distance is 800mm.

Design of Gantry Girder

Crane Capacity = 400 kN

Self-wt. of the crane girder excluding trolley = 200 kN

Self wt of trolley, electric motor, hook etc. = 40 kN

Approximate minimum approach of the crane hook to the gantry girder = 1.20 m

Wheel base = 3.5 m

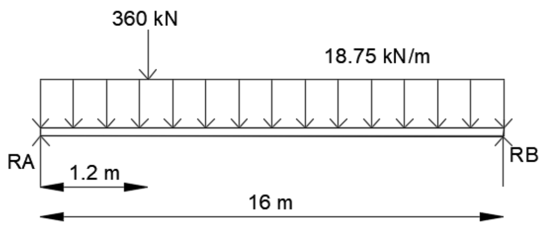
c/c distance between gantry rails = 16 m

c/c distance between columns(max) = 8 m

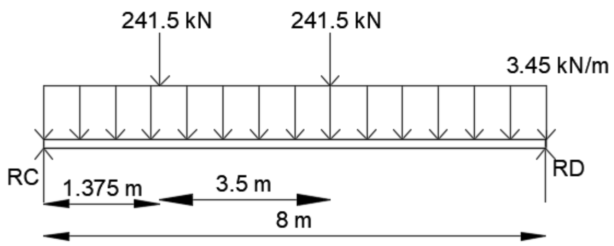
Self-wt. of rail section = 300 N/m

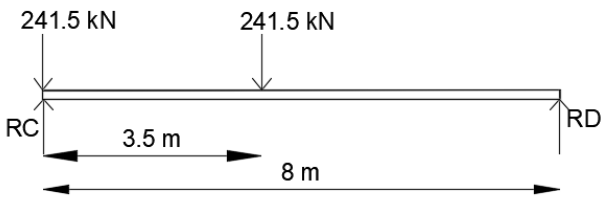
Diameter of crane wheels = 150 mm

Steel of grade Fe410

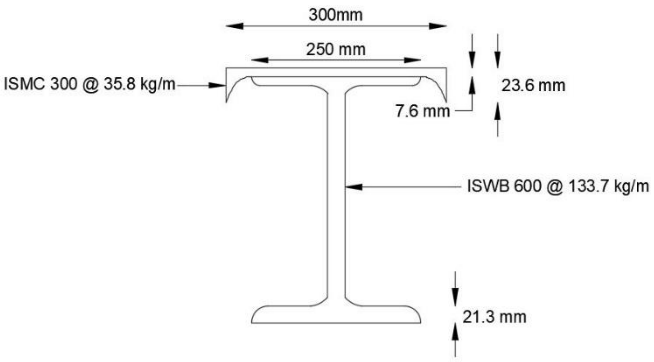
References	Steps	Calculations	Remarks
	1	**Maximum wheel load** Max conc. Load on crane =400+40 =440 kN Max factored load on crane =1.5*440 = 660 kN The crane will carry the self-wt. as UDL = 200/16 = 12.5 kN/m Factored UDL = 18.75 kN/m	
			
		Taking moment about B, $R_A * 16 = 360 * (16 - 1.2) + \frac{18.75 * 16^2}{2}$ $R_A = 705\ kN$ $R_A = 255\ kN$ Similarly, the reaction from the crane girder is distributed equally on the two wheels at the end of the crane girder.	**Maximum wheel load on each wheel of crane = 705/2 =352.5kN**
	2.	**Maximum bending moment** Assuming self-wt. of gantry girder to be 2kN.	

<table>
<tr>
<td></td>
<td></td>
<td>Total DL(w)= 2000+300 = 2.3 kN/m
Factored DL = 1.5*2.3 = 3.45 kN/m

The position of one-wheel load from the mid-point of span = wheel base/ 4
= 3.5/4 = 0.875 m
BM due to live load only:


Taking moment about D,
$$R_C * 8 = 352.5 * 6.625 + 352.5 * 3.125$$
$$R_c = 430 \text{ kN}$$
$$R_D = 275\ kN$$
Max BM due to live load = 275*3.125
= 859.375 kNm
BM due to impact = 0.10* 859.375
= 85.938 kNm
Total BM due to LL&impact = 945.31 kNm
BM due to DL $= \frac{wl^2}{8} = 27.6$kNm

</td>
<td>Maximum Bending moment = 972.91kNm</td>
</tr>
<tr>
<td></td>
<td>3.</td>
<td>Maximum Shear Force
Taking moment about D,
$$R_C * 8 = 352.5 * 8 + 352.5 * 4 = 528.75kN$$</td>
<td>Maximum shear force due to wheel load = 528.75 kN</td>
</tr>
<tr>
<td></td>
<td>4.</td>
<td>Lateral Forces
Lateral force transverse to the rails
= 5% of wt. of crab and weight lifted =0.05*440 = 22 kN
Factored lateral forces = 33 kN
Lateral forces on each wheel = 16.5 kN</td>
<td></td>
</tr>
<tr>
<td>IS 800 Annex H</td>
<td>5.</td>
<td>Preliminary trial section
Approximate section modulus required:
$$Z_{pz} = 1.1 * \left(\frac{M_Z}{f_y}\right) = 4280.8 * 10^3 mm^2$$
Taking ISWB 600 @133.7 kg/m and ISMC 300</td>
<td></td>
</tr>
</table>

@35.8 kg/m on its top flange.



	ISWB 600	ISMC 300
A	17000 mm^2	4630 mm^2
D	600	300
B	250	90
t	11.2	7.8
T	21.3	13.6
Ix	$1.06*10^9 mm^4$	$6.42*10^7 mm^4$
Iy	$5.3*10^7 mm^4$	$3.13*10^6 mm^4$

Centroid along Y-axis

$$\bar{Y} = \frac{17000 * 300 + 4630 * (600 + 7.8 - 23.5)}{17000 + 4630}$$

6.

$$= 360.86\ mm$$

Moment of inertia

$$I_{xx} = 1.06 * 10^9 + 17000 * 60.86^2 + 6.42 * 10^7 + 4630 * (600 + 7.8 - 360.86 - 23.5)^2 = 1.418 * 10^9 mm^2$$

$$Z_e = \frac{I_{xx}}{y} = 3.93 * 10^6\ mm^3$$

And,

$$I_{yy} = 5.3 * 10^7 + 3.13 * 10^6 = 5.613 * 10^7 mm^2$$

7.

Plastic modulus calculation

Distance of equal area from top surface of compound

		section is obtained by equating areas on two sides of equal area axis. Now, $4630 + 250 * 21.3 + y_1 * 11.2 = 250 * 21.3 + (600 - 2 * 21.3 - y_1) * 11.2$ Hence, y_1 = 72 mm (From lower surface of top flange) Z_p from area above plastic NA, = $8.07*10^5$ mm^3 Z_p from area below plastic NA, = $3.961*10^6$ mm^3 Therefore, Z_{Px}= $4.83*10^6$ mm^3 Z_{py}= $824.76*10^3$ mm^3	
IS 800 Table 2	**8.**	**Classification of Section** For I section, b/t_f= 125/21.3 =5.86 <8.4 d/t_w=(600-2*21.3)/11.2 = 49.76<84 For C section. b/t_f= 82.4/13.6 =6.05 <8.4 Hence, entire section is plastic.	**Entire section is plastic.**
	9.	**Check for moment** Bending strength: $M_d = \beta_b Z_p \left(\frac{f_y}{\gamma_{m0}}\right) < 1.2 Z_e \left(\frac{f_y}{\gamma_{m0}}\right)$ $M_{dx} = 1 * 4.83 * \frac{250}{1.1} = 1097.7\ kNm$ $< 1.2 * 3.93 * \frac{250}{1.1} = 1071.8$ kNm Bending strength, M_{dx} = 1097.7 kNm For compression flange, $M_d = \beta_b Z_p \left(\frac{f_y}{\gamma_{m0}}\right) < 1.2 Z_e \left(\frac{f_y}{\gamma_{m0}}\right)$ $M_{dy} = 1 * 8.25 * \frac{250}{1.1} = 187.44\ kNm$ $< 1.2 * 5.809 * \frac{250}{1.1} = 158.43$ kNm $M_{dy\ (flange)}$ = 158.43 kNm	**Safe**
	10.	**Check for local moment capacity** $\frac{M_{ux}}{M_{dx}} + \frac{M_{uy}}{M_{dy}} < 1$	

		$= \frac{676.16}{1097.7} + \frac{21.97}{158.43} = 0.754 < 1$	
IS 800 clause 8.4	**11.**	**Check for shear capacity** $V_d = V_n/V_{mo}$ $V_n = \frac{A_v f_Y w}{\sqrt{3}} = 1039.23\ kN$ $V_d = \frac{1039.23}{1.1} = 944.75\ kN > 377.34\ kN$ Also, $0.6V_d = 566.85 > 377.34\ kN$	**Safe against shear**
IS 800 Table 6	**12.**	**Deflection Check** $\delta = WL^3 * \frac{\frac{3a}{4L} - \frac{a}{L^3}}{6EI} = 10\ mm$ Permissible maximum deflection = L/500 = 16 mm > 10 mm	
	13.	**Design of Connections** Here, V= 377.34 kN A= 4630 mm^2 y= 360.86-7.6 = 353.26 mm I_x= 1.418*10^9 mm^4 The required shear capacity of the weld: $q = \frac{VAy}{I_Z} = 435.24\ \mathrm{N/mm}$ Let us provide 6 mm weld size to connect channel with flange of I-section. Strength of weld provided $= 0.7 * 6 * \frac{410}{1.5 * \sqrt{3}} = 662.8\ \mathrm{N/mm}$ . .	**Provide 6 mm size fillet weld for connection**

Design Summary:

IS WB 600 @ 133.7 kg/m and IS MC 400@ 35.8kg/m

Thickness of weld: 7mm

Design of Corbel

Grade of concrete= M25

Grade of steel =Fe500

Ultimate load = 660 kN

Shear span(a)= 350 mm

References	Steps	Calculations	Remarks
IS 456:2000 Cl. 40.2.3	**1**	**Calculation of load** Unit wt, of concrete = 25kN/m^3 Unit wt. of crane rail = 0.65 kN/m Max. C/C span of column = 8m Span of EOT crane = 16 m Wheel distance of crane = 1.5 m Wheel load including impact W1= 660 kN Beam wt. = (35.8+133.7) *8 =1356 kg = 13.56 kN Rail wt. = 0.44*8 = 3.52 kN Total vertical load on corbel = 678 Kn Factor of safety for crane and rail load = 1.05 Factor vertical load for beam = 1.5 So, total factored vertical load = 720kN	**Total vertical load = 720 kN**
	2	**Bearing Plate** Here, length of bearing plate = 300 mm Width of bearing plate = 250 mm Bearing stress of concrete = 0.45*f_{ck} = 11.25 MPa Bearing area required = $\frac{P_u}{\sigma_{br}} = 64000\ mm^2$	**Adopt a bearing plate of 300*250 mm**
	3	**Depth of corbel at support** For M25, $T_{c,max}$= 3.1 MPa Let, T_c = 80% of $T_{c,max}$= 2.48 MPa $d = \frac{P_u}{L * T_c} = 967.7$ mm Adopt, d= 1000 mm with cover of 50 mm Depth of corbel at face =700 mm Dist. of load from the face of corbel: (a)= 300 mm For, strut action a/d = 300/1000=0.3	**Depth of corbel >D/2** **a/d<0.6.(OK)**
	4.	**Lever arm** $\alpha = \frac{P_u}{0.86\, \sigma_{ck}\, bd} = 0.04$	

DOED guidelines **SP-34:1987 Cl. 7.7.2**	**5.** **6.** **7.**	β= 0.3 Then, $\left(\frac{\mathrm{z}}{\mathrm{d}}\right)^2-\left(\frac{\beta}{\alpha+\boldsymbol{\beta}}\right)\frac{z}{d}+\left(\frac{\alpha}{\alpha+\boldsymbol{\beta}}\right)\beta^2=0$ $\text{or,}\frac{\mathrm{z}}{\mathrm{d}}=0.73{,}0.03$ $z=730mm$ and, $\mathrm{z}=\mathrm{d}-0.42\mathrm{x}$ $\mathrm{x}=645\ \mathrm{mm}$ x/d = 645/1000= 0.645 >0.46. **Area of tension steel** $\mathrm{F_T}=\frac{\mathrm{a}}{\mathrm{z}}*\mathrm{p}=295.89\ \mathrm{kN}<0.5*P=360kN$ So, F_T= 360 kN Strain in tension steel: $\mathcal{E}_\mathrm{s}=\frac{\mathrm{d}-\mathrm{x}}{\mathrm{x}}*\mathcal{E}_{\mathrm{c,max}}$ $\mathcal{E}_\mathrm{s}=0.0019$ Stress in Fe500 at yield point: $\mathcal{E}_{sy}=0.87*\frac{\mathrm{f_y}}{\mathrm{E_s}}=0.00218$ $\mathcal{E}_\mathrm{s}<\mathcal{E}_{sy}$ Stress in Fe500 at break point: $\mathcal{E}_{sy}=0.87*\frac{\mathrm{f_y}}{\mathrm{E_s}}+0.002=0.00418$ $\mathcal{E}_\mathrm{s}<\mathcal{E}_{sy}$ Stress in tension reinforcement =369.6N/mm^2 Required area of tension steel = 975 mm^2 Percentage of tension steel = 0.11% **Calculation for Tension reinforcement** Dia. of main tension bar = 25 mm Number of bars = 8 Area of reinforcement = 3927 mm^2 % of steel = 0.49% **Area of shear steel** Min. area of shear reinforcement = 0.5A_{st} = 1963.5 mm^2 Provide 12mm 2 legged stirrups – 10 nos. in upper 2/3 depth.	**Compression rebar is required.** **OK** **OK** **Not OK (% of steel should be within 0.4-1.3%)** **OK** **Provide 8 no. of 25 mm bars**

IS 456:2000 Table 20 **IS 456:2000 Cl. 26.2.3.3**	 **8.** **9.**	Area provided= 2010.61 mm^2 Spacing = 2/3* 1000/10 = 70 mm **Shear capacity of section** For, 0.5% of steel, T_c=0.49 MPa Increased shear strength $T'c = \left(\frac{2d}{a}\right) T_c = 3.26 \text{ MPa} > 3.1 MPa$ Total shear resistance: $V_u = V_C + V_V = 3963\ kN > 760 kN$ **Development Length** $L_d = \phi * \left(\frac{0.87 f_y}{T_{bd}}\right) = 1215\ mm$	 **OK** **Development length of 1215 mm**

Design Summary:

- Bearing plate of 300*250 mm
- 8 nos. of 25 mm bar as tension bar
- 10 nos. of 12 mm 2 legged stirrups in 2/3 of depth 70 mm c/c.
- Development length of 1215 mm.

Design of Staircase

References	Steps	Calculations	Remarks
	1. 2. a. i.	 Floor height = 6000mm Riser = 200mm Tread = 250mm No of flight = 2 No of riser in a flight= 6000/ (2*200) =15 No of tread in a flight = 15-1 =14 Width of tread=600-2R = 200mm Width of stair=space given/2 – railing gap- 150= 1400mm Width of landing=width of stair=1300mm Going length = 3500 mm Effective length = 6500 mm From deflection criteria, Effective depth =195 mm, clear cover of 25 mm **Load calculation** **For Flight**	**No. of tread =14** **No. of riser = 15** **Rise = 200 mm** **Tread=250mm**

	ii.	Weight of steps per meter width of flight W_1 = Area of step*1*Density = 0.625 kN/m	
	iii.	Weight of steps per meter in plan W_2= Area of step *1*density/tread= 2.5 kN/m	
	iv.	Weight of waist slab per meter width W_3 = Density*1*(D*$(R^2+T^2)^{1/2}$) = 1.76 kN/m Weight of waist slab per meter length in plan W_4 = Density*1*(D*$(R^2+T^2)^{1/2}$) /T = 7 kN/m	
	b.	Floor finish = 1.1 kN/m Live load= 5kN/m Dead Load= W_2+W_4 =9.5 kN/m Total load = 15.6 kN/m Design load = 23.4 kN/m	
	c.	**For landing A** Floor finish = 1.1 kN/m Live load = 5 kN/m Total load = 13.1 kN/m Design load (W_u) = 19.65 kN/m	
	4.	**For landing B** The design load for upto 1100 mm will be same. But for distance of 150 mm from the wall, there will be no live load. So, Total factored load = 15.9 kN/m **Calculation of maximum design BM and SF** For landing A, R_A+R_B=123.8 KN $\Sigma M_A=0$ On calculation, R_B=60.71 KN R_A=63.09 KN Let point of zero SF occurs at distance x from A. SF at x = 0 So, x= 2.82 m Maximum BM = 63.09*2.82+19.65*.75*(2.82-0.75/2)-23.4*0.68^2/2 = 188.88 kN/m	
IS 456:2000 Cl. 40.1	5.	$M = 0.133 f_{ck} b d^2$ $d = \sqrt{\frac{M}{0.138 * b * \text{fck}}}$ d = 233mm = 235 mm with 25 mm cover **Design**	**Provide 16mm dia. main bars @ 140mm**

	6. 7. 8.	Area of steel: $$A_{st} = \frac{BM}{0.87 * \sigma_y * 45.45} = 2290\ mm^2$$ Provide 11 numbers of 16 mm bars @140 mm c/c spacing. For Distribution bar, Area of distribution bar = 0.12%of bD = 423mm^2 Providing 9 nos. 10 mm ∅ bars Spacing = 110 mm c/c **Check for shear** Nominal shear stress = $$\tau_v = \frac{V_u}{bD} = \frac{63.09 * 10^3}{1500 * 235} = 0.179\ N/mm^2$$ Percent tension stress=$\frac{100 A_{st}\,(provided\ )}{bD} =$ 0.63 $$\tau_c = 0.54$$ Shear strength of slab $\tau_c' = 1.1 * 0.54 = 0.594 > 0.179$ Check for Development length For 16 mm bar, **Development length:** $$L_D = \frac{\phi \sigma_s}{4\tau_{bd}} = 1243\ mm$$ **Design of landing slab A** Effective span = 1.4+0.20+1.4+0.235 =3.235 m Width = 1.5 m Factored load = 15.9 kN/m^2 Total load = 15.9* 1.5* 3.235 =77.155 kNm Reaction from one flight =50 kN Maximum BM = 71.64 kN/m^2 Maximum SF = 89 kN Effective depth = 235 mm Area of steel: At = 830 mm^2 So, use 7 nos. bar of 12 mm diameter in landing slab	**spacing c/c** **Provide 10mm dia. distribution bars @ 110mm spacing c/c** **Provide 7 nos. bar of 12 mm diameter 210 mm c/c.**

Design Summary

- No. of tread=14
- No. of riser = 15
- Rise = 200 mm
- Tread=250mm
- For Flight: use 16mm dia. main bars @ 140mm spacing c/c
- 10mm dia. distribution bars @ 110mm spacing c/c
- For landing: Use 7 nos. bar of 12 mm diameter 210 mm c/c.

Design of Shear Wall

References	Steps	Calculations	Remarks
	1	**Design Constants** Clear height between the floor (h) 5.8 m unit weight of soil, γ = 20 KN/m^2 Angle of internal friction of the soil, ɵ = 30^0 surcharge produced due to vehicular movement is Ws1 = 20 KN/m^2 Safe bearing capacity of soil, qs = 61.6 KN/m^2	
	2	**Moment calculation** Active pressure coefficient (Ka) = $\frac{1-sin\emptyset}{1+sin\emptyset}$=0.333 Lateral load due to soil pressure, Pa = Ka x γ x h^2/2 = 0.333x20x5.8^2/2 =112.02kN/m	**Pa= 112.02 kN/m**
		Lateral Load due to surcharge load, Ps = Ka x Ws x h =0.33*20*5.8 =38.28kN/m	**Ps= 38.28 kN/m**
		Characteristic Bending moment at the base of wall, since weight of wall gives insignificant moment, so this can be neglected in the design Mc = Pa x h/3 + Ps x h/2 =112.02*5.8/3+38.28*5.8//2 =327.584 kN-m Design moment, M = 1.5Mc=1.5*266.942=491.376 kN-m	
	3	**Approximate design of section** Let effective depth of wall = d	**M= 491.376 kN-**

IS456:2000 Cl.32. 3.4		BM = 0.136 $f$ckbd^2 491.376*10^6=0.136*25*1000*d^2 d = 380.16 mm Adopt d= 610 mm Let effective cover is 40mm Overall depth of wall, D = 610+40 = 650 mm Take D = 650 mm	**m** **D=650 mm d=610 mm**
IS456:2000 Cl.32.5. a	**4**	**Calculation of Vertical Steel Reinforcement** $A_{st}=\frac{bdfck}{2*fy}(1-\sqrt{1-\frac{6.6M}{fckb\,d^2}}$ A_{st}=2942.06mm^2 Min A_{st}=0.12% of b*D =0.0012*1000*650 =780 mm2 <A_{st} Max. Dia. of bar = D/30 = 650/30=22 mm <u>For Outer face vertical reinforcement,</u> Providing 20 mm- Φ bar, Area of 20mm bar =314.16 mm^2 No. of bars =9.36 = 10 Spacing of the bar @ 100 mm c/c A_{st} provided=3141.6 mm^2 P_t=3141.6*100/ (1000*610) =0.51% <u>For Inner face vertical reinforcement,</u> Providing 16 mm- Φ bar, Area of 16 mm bar =201.06 mm^2 No. of bars =14.63 = 15 Spacing of the bar = 75 mm c/c A_{st} provided=3015.9 mm^2 P_t=3015.9*100/ (1000*610) =0.49%	**Ast=2942.06 mm^2** **At outer face, Providing 10-20mm- Φ bar @ S=100mm Pt= 0.51%** **At inner face, Provide 14-16 mm - Φ bar @ 75 mm c/c**
IS456:2000 26.5.2.2 IS456:2000 Cl.31.6.2.1 **IS456:2000 Table-19**	**5**	**Check for Shear** The critical section for shear strength is taken at a distance of 'd' from the face of support. Thus, Critical section is at d = 0.24 m from the top of mat foundation. i.e at (5.8-0.24) = 5.56 m from the top edge of wall Shear force at critical section is Vu = 1.5x(Ka x Ws x Z + Ka x γ x Z^2/2) =1.5*(0.333*20*5.56+0.333*20*5.56^2/2) =209.95 kN Nominal shear stress, $\tau u=vu/bd$ =209.95*1000/ (1000*610) =0.34 N/mm2 Permissible shear stress τc=0.38 N/mm2 τc>τu Hence safe.	**Vu=176.92 kN** **τu=0.34 N/mm2** **τc=0.38 N/mm2**
IS456:2000 Cl.23.2. a	**6**	**Check for Deflection** Leff =5.8+0.61=6.41 m	

IS456:2000 **Cl.32.5. d**	**7**	Allowable deflection = Leff /250=26mm Actual deflection= $\frac{P_s+Leff^4}{8EI}+\frac{P_a+Leff^4}{30EI}$ =23.43 mm <26mm Which is less than allowable deflection, hence safe. **Calculation of Horizontal Reinforcement steel bar** Area of Horizontal Reinforcement=0.002Dh =0.002*650*5800 = 7540 mm^2 As the temperature change occurs at front face of basement wall, 2/3 of horizontal reinforcement is provided at front (outer) face and 1/3 of horizontal reinforcement is provided in inner face. Front face Horizontal Reinforcement steel, =2/3*7540 =5026.67 mm^2 Providing 16mm- Φ bar No of bars required, N=5026.67/201=25 nos Spacing = (h-clear cover at both sides- Φ)/(N-1) = (5800-40-16)/(25-1) =239.33mm Provide 16mm- Φ bar @ 200 mm. Max Spacing = 3d or 450mm. OK Inner face Horizontal Reinforcement steel, =1/3 * 7540 = 2513.33 mm^2 Providing 12mm- Φ bar No of bars required, N=2513.33/113=23 nos Spacing = (h-clear cover at both sides- Φ)/(N-1) = (5800-40-12)/ (23-1) =261.3 Provide 12mm- Φ bar @ 250 mm Max Spacing = 3d or 450mm. OK Hence spacing provided for Horizontal steel is OK.	**Providing 25 -16mm- Φ bar @ 200 mm at outer face.** **Providing 23 -12mm- Φ bar @ 250 mm at inner face.**

Detail Summary:

Overall depth of wall, D = 650 mm

- Clear cover is 40mm & Max bar used is 20mm-Φ
- Provide vertical reinforcement 20mm-Φ @100 mm c/c at the outer (front) face
- Provide nominal vertical reinforcement 16mm- Φ bar @ 75 mm c/c at the inner face
- Provide horizontal reinforcement of 16mm-Φ bar @ 200 mm c/c at outer face.

- Provide horizontal reinforcement of 12mm-Φ bar @ 250 mm c/c at inner face.

Design of Isolated Footing

References	Steps	Calculations	Remarks
	1.	**Soil Pressure** Axial load = 800 kN Assume, safe bearing capacity of soil (SBC) = 150 kN/m^2 For earthquake load, bearing capacity is increased by 50%. Bearing capacity of soil (q_0) = 225 kN/m^2 Area of foundation: A= 800/225 = 3.6 sq. m Thus, area of square footing is taken to be 2*2 sq. m	**$A_{required}$ = 3.6 sq. m** **$A_{provided}$ = 4 sq. m**
	2.	**Factored soil pressure** $\rho = * \frac{800}{2^2}$ = 192 kN/m^2 Area occupied by 15 footing= 60 sq.m 50% of plinth area = 120 sq. m So, isolated footing is required.	**OK**
	3.	**By one-way shear criteria,** Shear force: $V_u = \rho * B\left(\frac{L-a}{2} - d\right)$ $= 192 * 2\left(\frac{2-0.4}{2} - d\right) \ldots \ldots .(i)$ Also, Taking % of steel = 0.5 $V_u = \tau * B * d$ $= 0.49 * 10^3 * 2 * d \ldots \ldots \ldots (ii)$ Equating (i) and (ii), d= 0.225 m	
	4.	**By two-way shear** Shear force: $V_u = \rho\big(LB - (a+d)(b+d)\big)$ $= 192(4 - (0.4+d)^2) \ldots \ldots ..(i)$ Also, $V_u = \tau_v * 2d(0.4 + 0.4 + 2d)$ $= 2.5 * 10^3 * (0.8d + 2d^2) \ldots .(ii)$ Equating (i) and (ii), d= 0.30 m	**d = 300 mm** **Cover= 50 mm** **D= 350 mm**
	5.	**Bending moment criteria** $M_L = \rho * \left(\frac{L-a}{2}\right)^2 * \frac{1}{2} = 76.8\ kNm$ Then, BM = $0.138\sigma_{ck}bd^2$	

	6.	$$d = \sqrt{\frac{105.84 * 10^6}{0.138 * 25 * 2.5 * 10^3}} = 110mm$$ **Area of steel** $$BM = 0.87\sigma_y A_{st}\left\{d - \frac{\sigma_y A_t}{b\sigma_{ck}}\right\}$$ $$76.8 * 10^6 = 0.87 * 500 * A_{st}\left\{300 - \frac{500A_t}{2000 * 25}\right\}$$ A_{st} = 5898.14 mm^2 Minimum reinforcement reqd. =0.12% of BD = 720 mm^2 Minimum reinforcement required <Ast Let's provide 28 mm bars. No. of bars = 58898.14 / 615.75 = 10 bars	**OK** **Provide 10 no. 28ϕ bars**
IS 456:2000 Cl. 40.1	**7.**	**Development length** $$L_d = \phi * \left(\frac{0.87f_y}{T_{bd}}\right) = 1400\ mm$$ Load transferred from column to footing: Nominal bearing stress in the column $$\sigma_{br} = \frac{P_u}{A_c} = 5\ \text{N/mm}^2$$ Allowable bearing stress = 0.45σck =11.25 N/ =11.25 N/mm^2>7.5 N/mm^2 Hence, load can be transferred by bearing alone.	**Dowels bar are not required**

Design of Mat Foundation

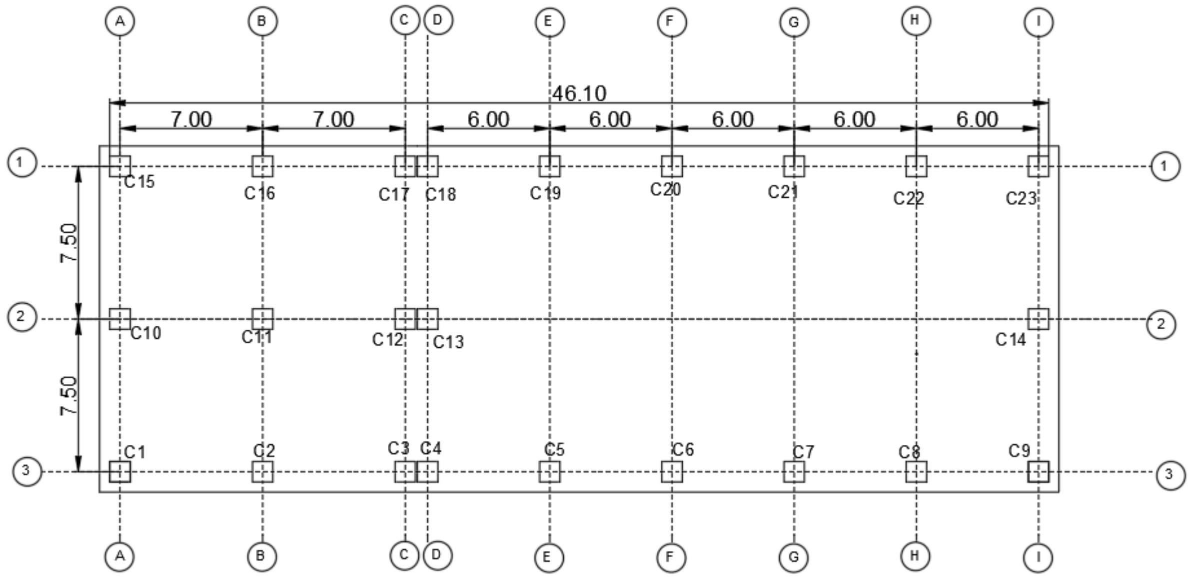


Fig: Column Layout of Powerhouse

Column	Axial forces	x	y	q
C1	1015.08	23.00	8.00	58.36
C2	1224.17	15.50	8.00	52.24
C3	1366.95	8.50	8.00	46.54
C4	1913.29	7.50	8.00	45.72
C5	1188.22	1.50	8.00	40.83
C6	1063.91	-4.50	8.00	35.94
C7	1060.00	-10.50	8.00	31.04
C8	1123.09	-16.50	8.00	26.15
C9	1604.99	-22.50	8.00	21.26
C10	1205.09	23.00	8.00	58.36
C11	1758.68	15.50	8.00	52.24
C12	1087.03	8.50	0.00	46.54
C13	997.12	7.50	0.00	45.72
C14	982.06	-23.00	0.00	20.85
C15	1015.08	23.00	0.00	58.36
C16	1224.17	15.50	-8.00	52.24
C17	1366.95	8.50	-8.00	46.54
C18	1913.28	7.50	-8.00	45.72
C19	1188.22	1.50	-8.00	40.83
C20	1063.91	-4.50	-8.00	35.94
C21	1060.00	-10.50	-8.00	31.04
C22	1123.09	-16.50	-8.00	26.15
C23	1604.97	-22.50	-8.00	21.26

Parallel to Y-axis

Strip	Width	Soil Pressure	Bending Moment
1-1	4.5	58.36	118.18
2-2	7	52.24	255.98
3-3	4	46.54	74.46
4-4	4	45.72	73.15
5-5	6	40.83	146.99
6-6	6	35.94	129.38
7-7	6	31.04	111.74
8-8	6	26.15	94.14
9-9	4	21.26	34.02

Parallel to X-axis

Strip	Width	Soil Pressure	Bending Moment
A-A	4.75	58.36	131.675
B-B	7.5	58.36	328.275
C-C	4.75	58.36	131.675

References	Steps	Calculations	Remarks
	1.	**Known data for design:** Maximum factored average upward soil pressure, q= 1.5*58.36= 87.54 kN/m2 Maximum span length, L = 7.5 m	**Q = 87.54 kN/m^2**
	2.	**Moment Calculation:** Maximum support moment, Ms= qL^2/10 Ms = 492.41 kN-m per m width Maximum span moment, Mm = qL^2/12 Mm = 410.34 kN-m per m width	
	3.	**Depth from moment consideration** Depth of footing, d = $\sqrt{\frac{M}{3.33*b}}$ $= \sqrt{\frac{492.41*10^6}{7.5*1000}}$ = 270 mm	**d = 300 mm**

	4.	**Depth from moment two-way shear consideration** Permissible punching shear, $\tau'_v = k_s * \tau_c$ Taking k_s= 1, τ'_v= 1*0.25*$\sqrt{25}$ = 1.25 N/mm^2 **For corner column C1** Perimeter b_o = 2(d/2+1500) = d+3000 mm Pu = 1.5*1015 = 1522.5 kN Nominal shear stress, $\tau_v = \frac{P_u}{b_o * d}$ = 1.25 d = 1350 mm D = 1350+50 = 1400 mm **For an edge column C2** Perimeter b_o=2(0.5d+1500) +(d+1000) =3d+4000 Pu = 1.5*1225 = 1837.5 kN Nominal shear stress, $\tau_v = \frac{P_u}{b_o * d}$ = 1.25 d = 1020 mm D = 1020+50 = 1070 mm	**τ'_v= 1.25 N/mm^2**
IS 456:2000 Cl. 40.1	**5.**	**Calculation of area of main reinforcement, Along X-axis** Reinforcement in the short directions is given by, BM = 0.87 * σ_y* A_t*(d -$\frac{\sigma_y * At}{\sigma_{ck}\, b}$) 1.5*492.41*$10^6$ = 0.87*500* A_t *(1400 - $\frac{500 * At}{25*1000}$) A_t = 1852 mm^2 **Along Y-axis** Reinforcement in the direction of y-axis is given by, BM=0.87* σ y* Ast*(d- $\frac{\sigma\ y * Ast}{b.\sigma\ ck}$) 1.5*97.47*$10^6$ = 0.87*500* A_t *(1400 - $\frac{500 * At}{25*1000}$) At = 878.88 mm^2/m Minimum reinforcement in slabs, = 0.12% of bD = 0.0012*1000*480 = 480<1852 mm^2 and 878.88 mm^2 So, reinforcement along X-axis, Provide 16mmϕ bars @ 100mm c/c (at top and bottom) Along Y-direction, Provide 16mmϕ bars @ 200mm c/c (at top and bottom)	**d = 1350mm** **D = 14000mm**

Design Summary,

- Overall depth = 1400mm
- Cover = 50mm
- Reinforcement along X axis (top) = 16mmϕ bars @ 100mm c/c
- Reinforcement along X axis (bottom) = 16mmϕ bars @ 200mm c/c
- Reinforcement along Y axis (top) = 16mmϕ bars @ 100mm c/c
- Reinforcement along Y axis (bottom) = 16mmϕ bars @ 200mm c/c

Design of Barrel Foundation

Rotor

1. Vertical load = 10 tonnes per pedestal
2. Radial load in direction towards stator = 2 tonnes per pedestal
3. Tangential load = 1.5 tonnes per pedestal
4. No. of sole plates = 6

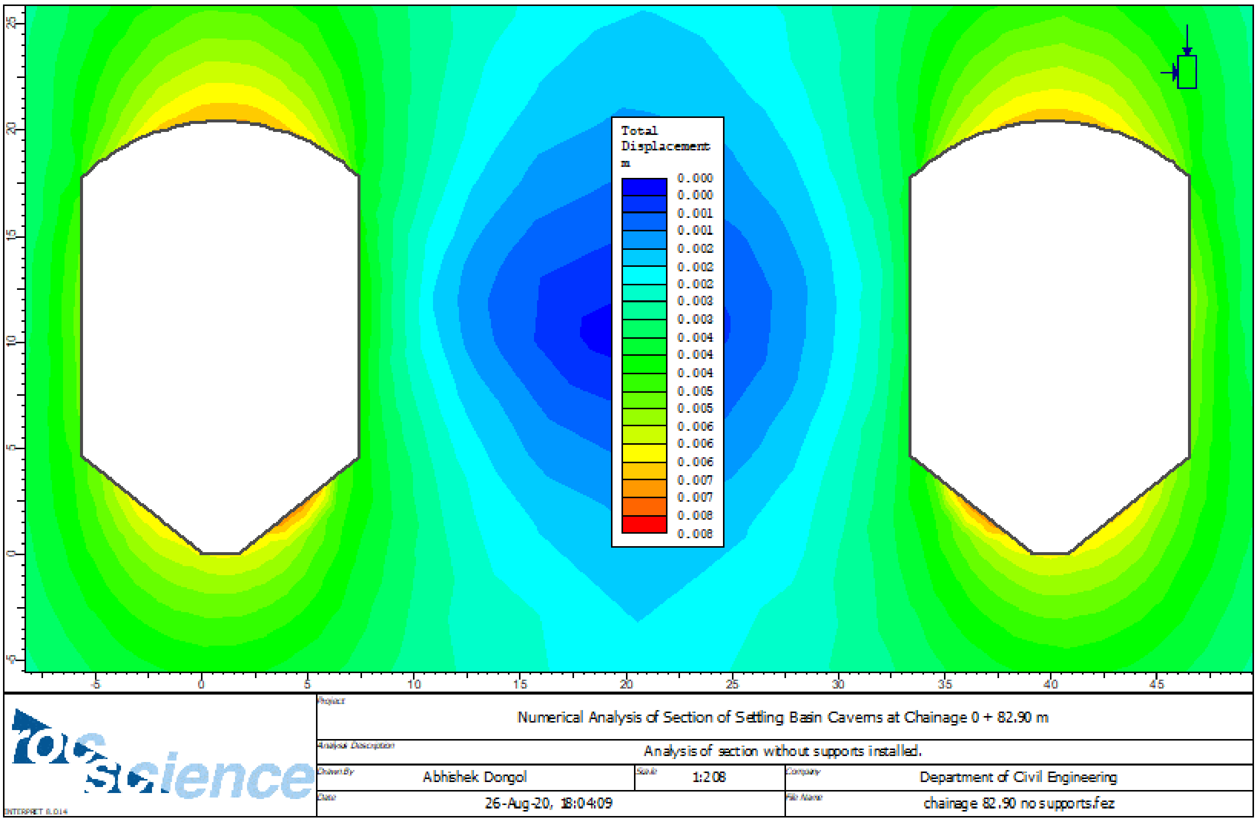
Stator

1. Vertical load = 10 tonnes per pedestal
2. Tangential load (under short-circuit condition) = 35 tones per pedestal
3. Tangential load (under normal condition) = 7 tones per pedestal
4. No. of pedestal = 6

References	Steps	Calculations	Remarks
	1	**Design of barrel foundation** Torque = 35*6*2.45+2*6*1.4 =546 ton /m D=2.2 m R= 3.4 and 1.4 m A= 2*pi*2.3*2.2 = 31.79 sq. m U=2*pi*2.3 = 14.45 m For Outer, Co = $\frac{2.2}{1-0.31} * (1 + 0.15\left(0.31 - \frac{1.78}{2-3.4}\right))$ = 2.1 m For Inner, Ci = $\frac{1.78}{1-0.31} * (1 + 0.15\left(0.31 - \frac{1.78}{2-1.6}\right))$ = 1.9 m Then,	

		F= t^3 * U =153.36 K=$\frac{0.3*F}{1+1.33*\frac{F}{4\,U^3}}$= 41 So = T*Co/K =28 ton/m^2 Si =25 ton/m^2 At1=$\frac{q}{6*Ro*\,\sigma st*rt}\ (Ro^3 - Ri^3)$ =22 cm^2 (for inner face) At2 = 10cm^2 (for outer face) For both inner and outer, At =15 cm^2 For hoop Ast = 15 cm^2 Take 25 diameter rebar @ 300 mm c/c both faces, Longitudinal Steel = $\frac{pi*q}{3*R*\sigma st} * (Ro^3 - Ri\text{^}3)$ = 321 cm^2 Minimum steel on both face = 0.15% = 953 cm^2 For longitudinal take dia. 25mm bar at 150 mm c/c So, Total steel provided = 986.96 sq. cm	**25mm rebar @ 300 mm c/c both faces.** **For longitudinal 25mm bar at 300 mm c/c**
	2.	**Check against Radial Load** Radial load per pedestal = 2 Total load = 12 ton Area = $\pi*(3.4^2-1.4^2)$ = 30.15 Stress = 12/30.15 = 0.39 ton/sq. m So, **neglect.**	
	3.	**Check against Vertical Load** Loads : Vertical load of rotor = 8.5 *6 = 51 ton Vertical load of stator =8.75 * 6 =52.5 ton Wt. of stator pedestal = 1 * 6 = 6 ton Wt. of barrel = 316 ton Total = 425 ton Area = 30.16 sq. cm Stress = 425/30.16 = 14 ton/sq. m (**neglect)**	
	4.	**For pedestal** At the pedestals, Provide 6 nos. of 16 mm	**6 nos. of 16 mm bars**

Annex B: Results of Numerical modeling of Settling Basin Cavern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Numerical Analysis of Section of Settling Basin Caverns at Chainage 0 + 82.90 m
Analysis of section without supports installed.
Abhishek Dongol
1:208
Department of Civil Engineering
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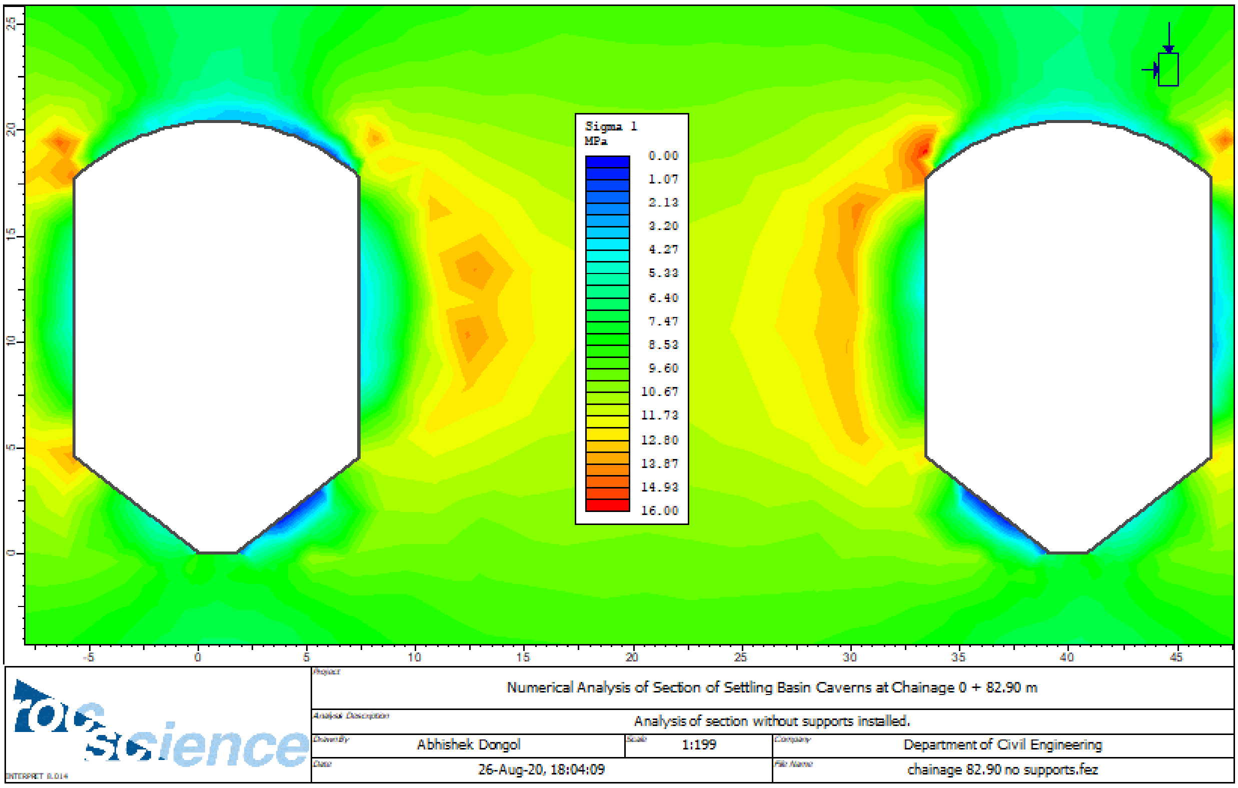
chainage 82.90 no supports.fez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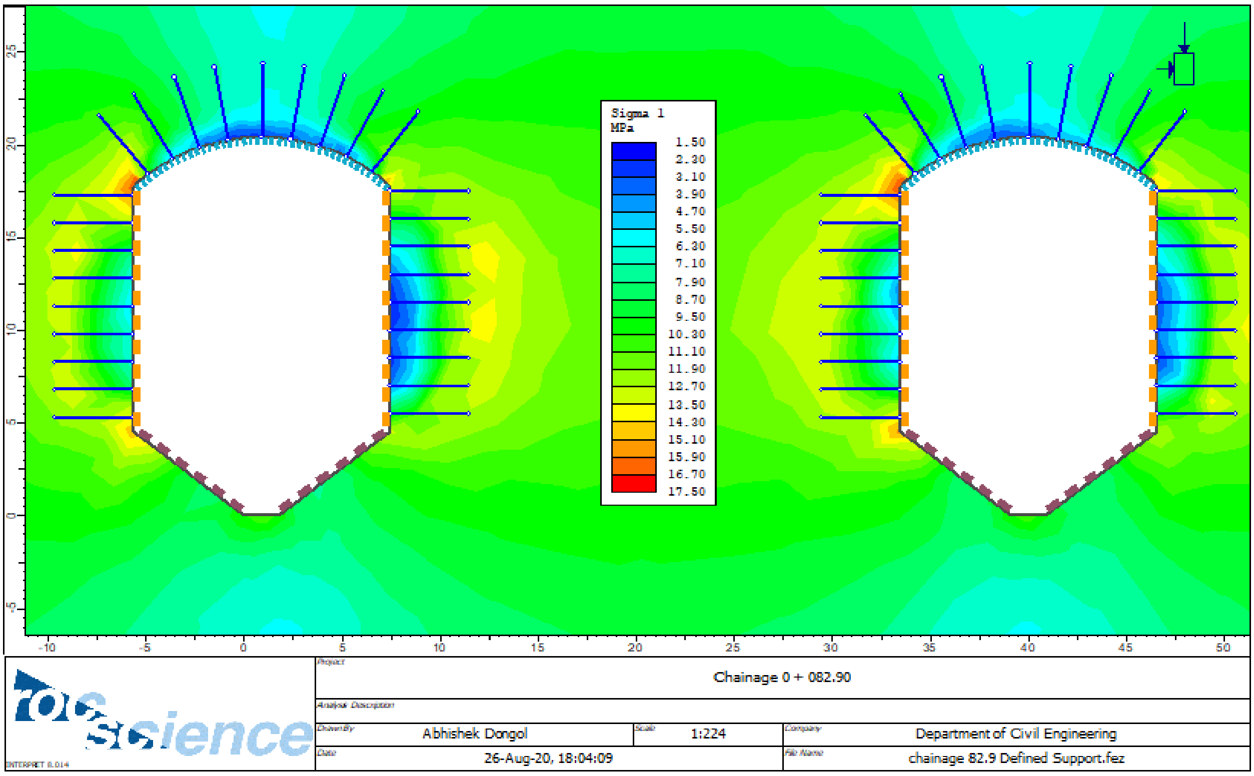
11.73
12.80
13.87
14.93
16.00
Numerical Analysis of Section of Settling Basin Caverns at Chainage 0 + 82.90 m
Analysis of section without supports installed.
Abhishek Dongol
1:199
Department of Civil Engineering
26-Aug-20, 18:04:09
chainage 82.90 no supports.fez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11.10
11.90
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Chainage 0 + 082.90
Analysis Description
Drawn By
Abhishek Dongol
Scale
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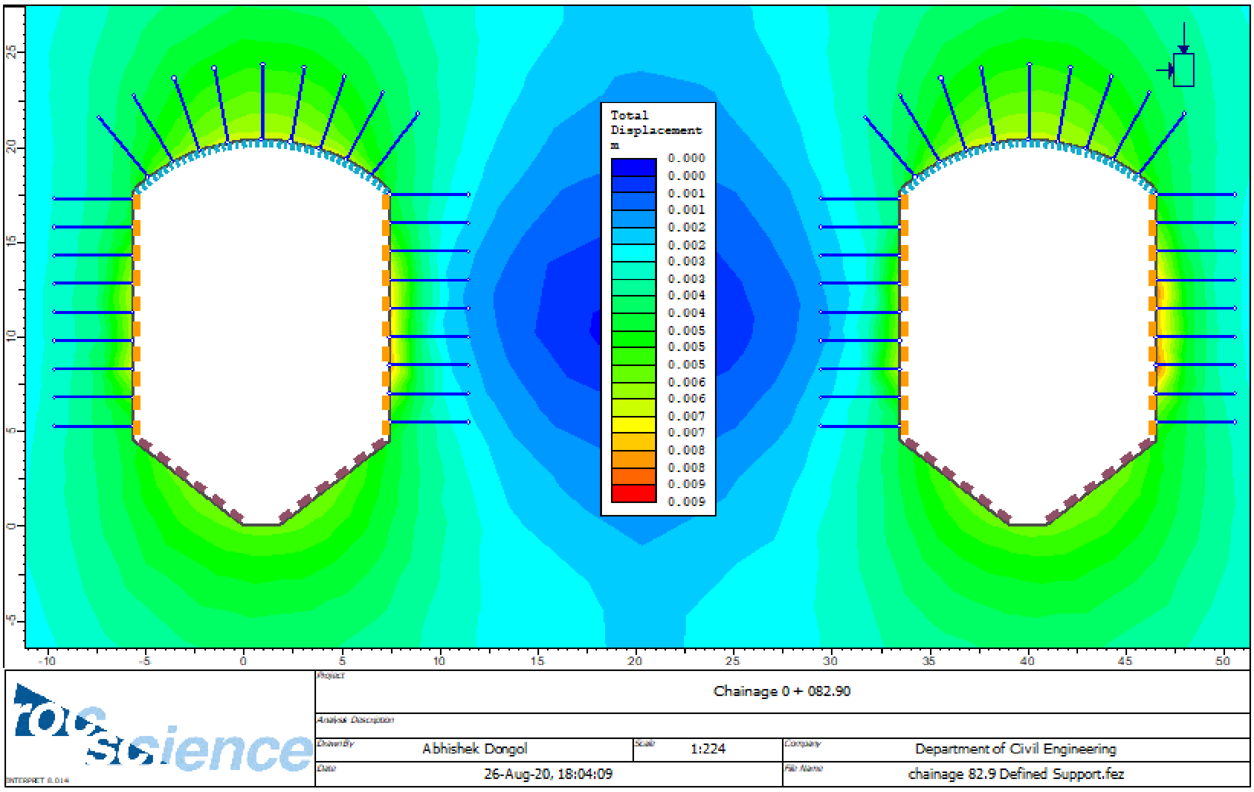
Company
Department of Civil Engineering
Date
26-Aug-20, 18:04:09
File Name
chainage 82.9 Defined Support.fez
INTERPRET 8.014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0.005
0.005
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0.009
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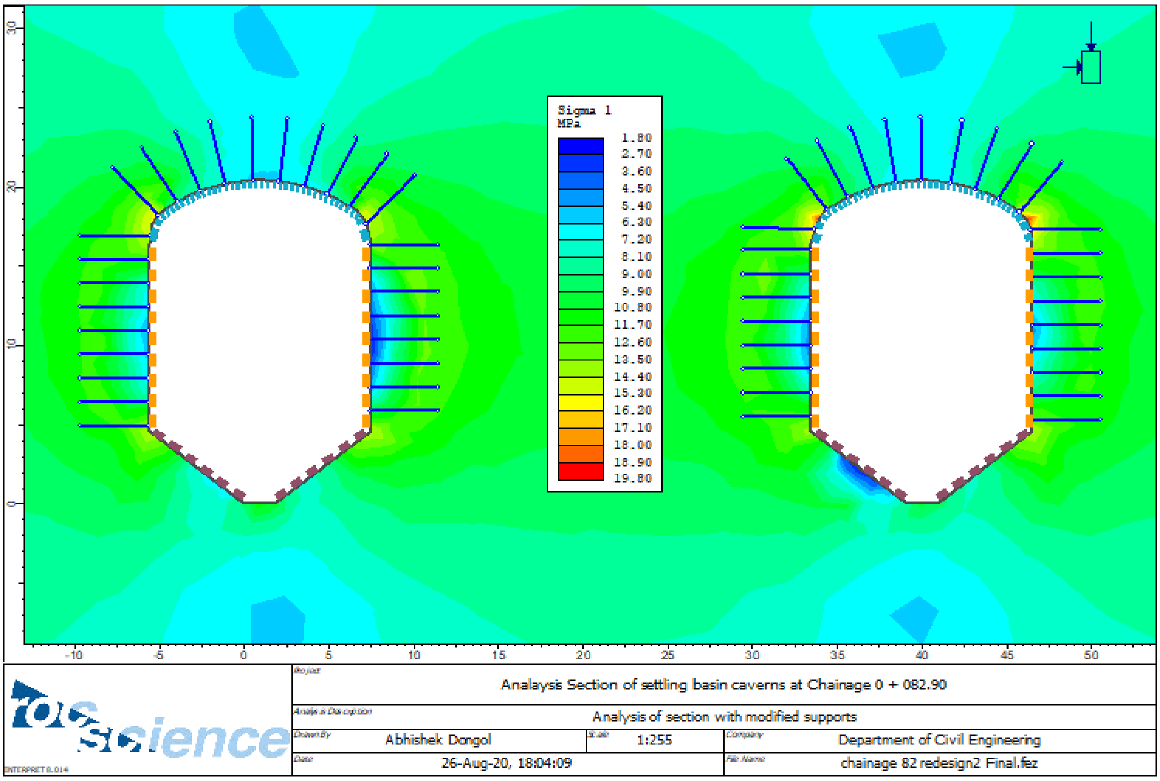
Project
Chainage 0 + 082.90
Analysis Description
Drawn By
Abhishek Dongol
Scale
1:224
Company
Department of Civil Engineering
Date
26-Aug-20, 18:04:09
File Name
chainage 82.9 Defined Support.fez
INTERPRET 8.014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Analaysis Section of settling basin caverns at Chainage 0 + 082.90
Analysis of section with modified supports
Abhishek Dongol
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Department of Civil Engineering
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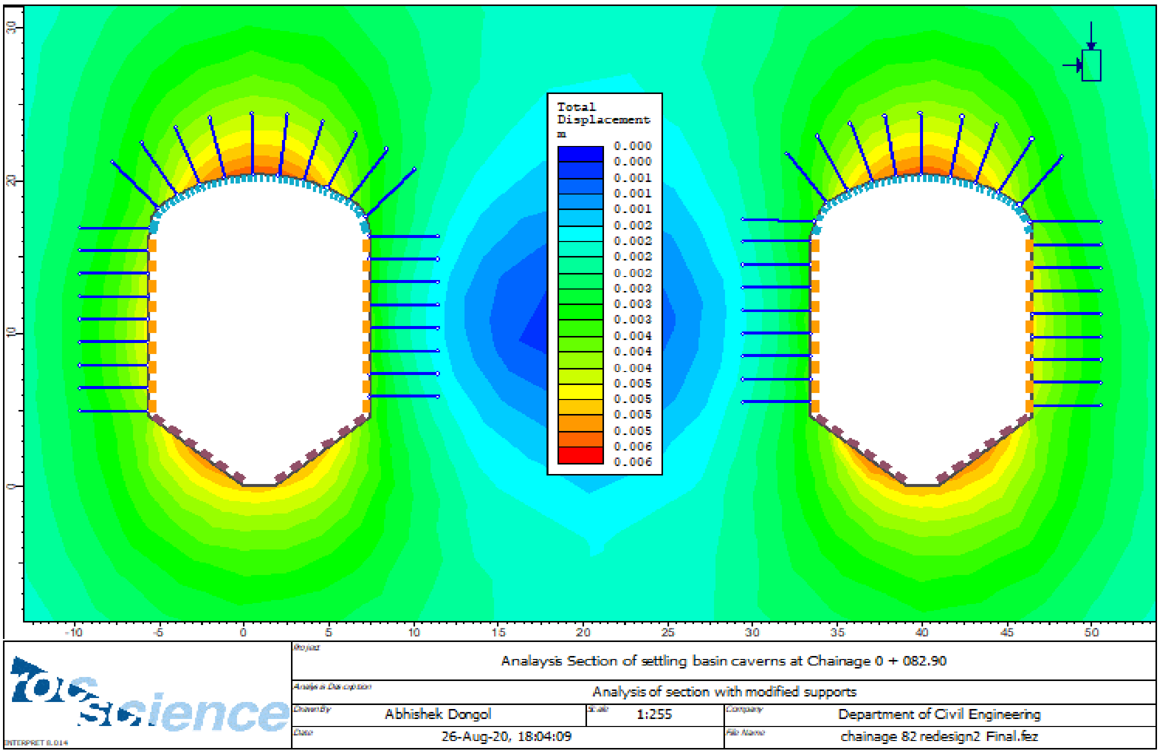
chainage 82 redesign2 Final.fez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Analaysis Section of settling basin caverns at Chainage 0 + 082.90
Analysis of section with modified supports
Abhishek Dongol
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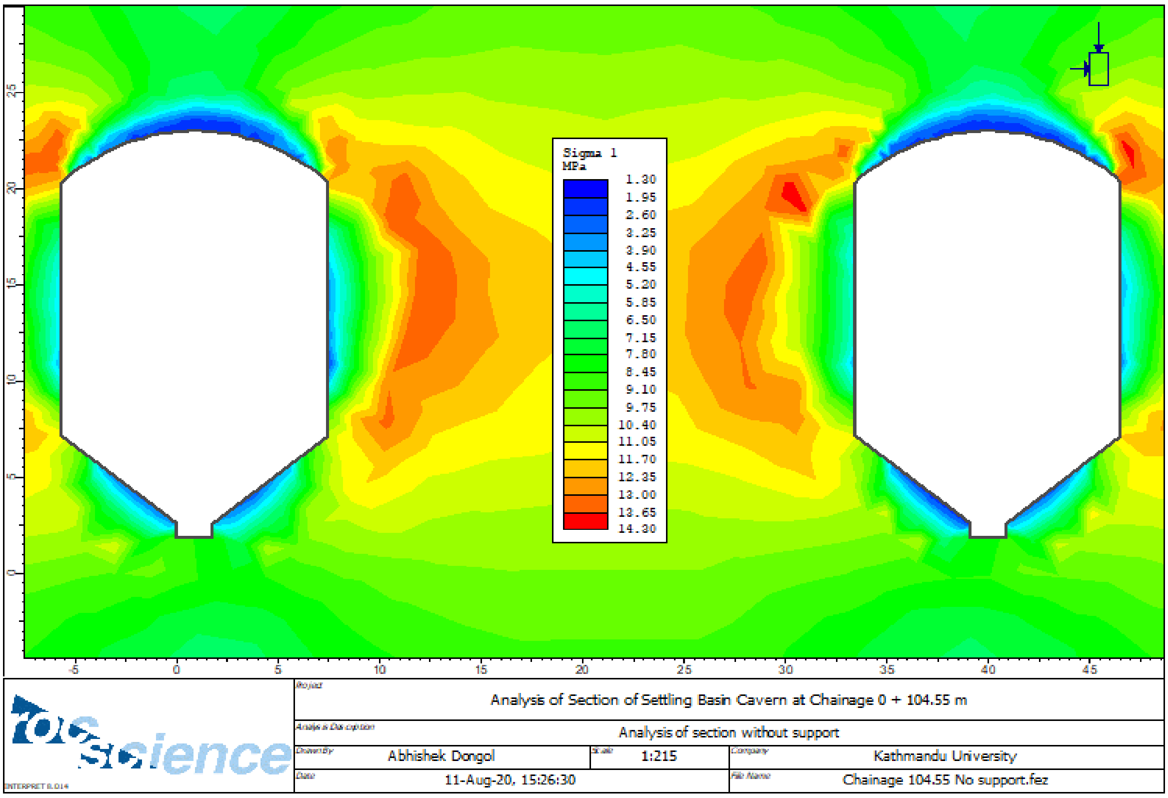
Department of Civil Engineering
26-Aug-20, 18:04:09
chainage 82 redesign2 Final.fez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Analysis of Section of Settling Basin Cavern at Chainage 0 + 104.55 m
Analysis Description
Analysis of section without support
Drawn By
Abhishek Dongol
Scale
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Company
Kathmandu University
Date
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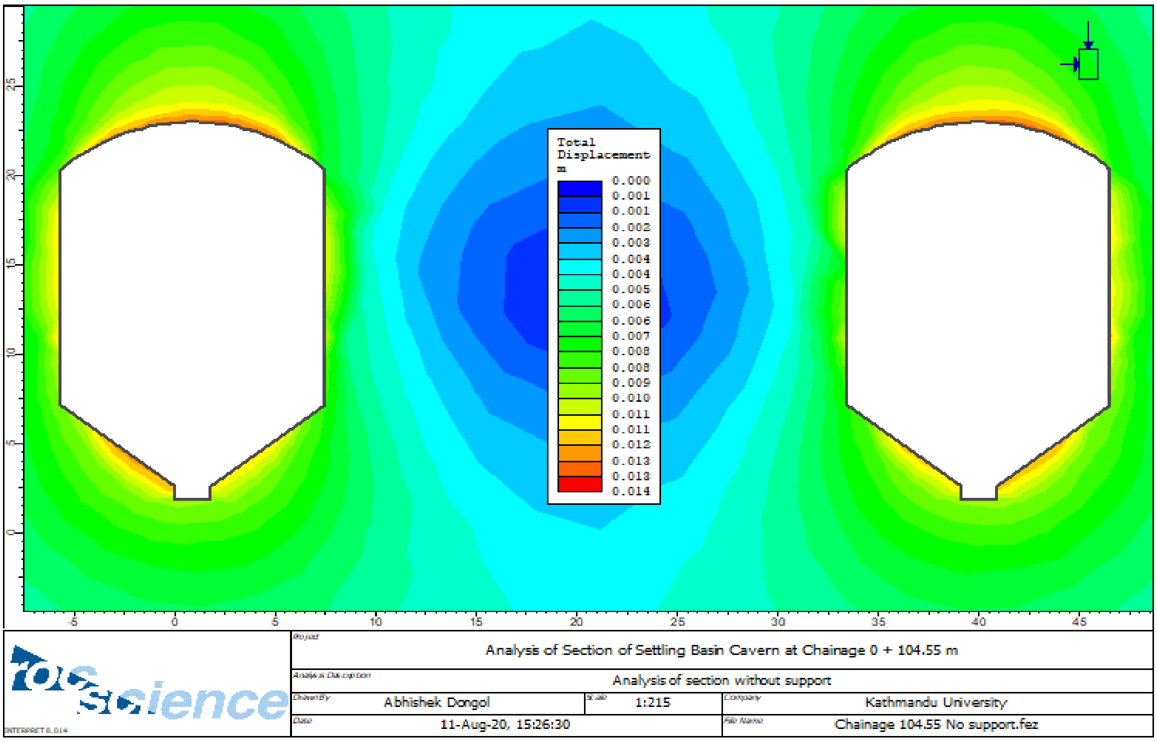
INTERPRET 8.014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Analysis of Section of Settling Basin Cavern at Chainage 0 + 104.55 m
Analysis Description
Analysis of section without support
Drawn By
Abhishek Dongol
Scale
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Company
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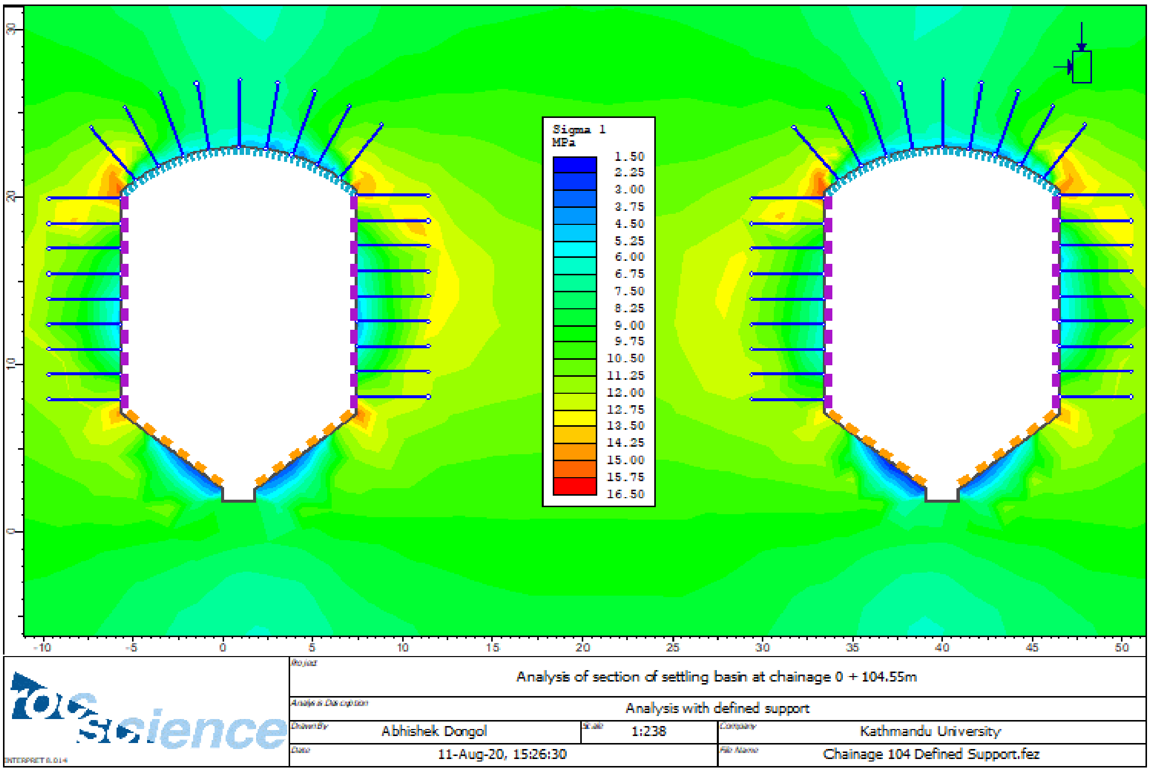
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Project
Analysis of section of settling basin at chainage 0 + 104.55m
Analysis Description
Analysis with defined support
Drawn By
Abhishek Dongol
Scale
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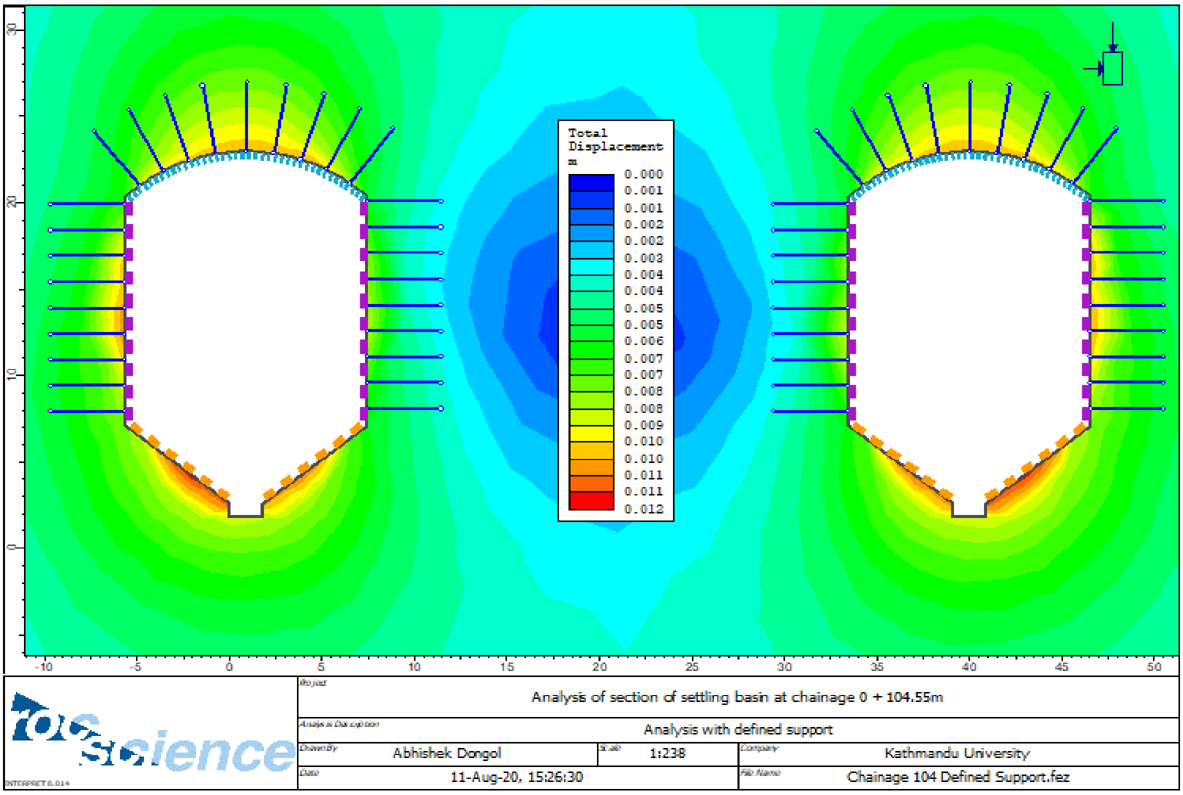
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Project
Analysis of section of settling basin at chainage 0 + 104.55m
Analysis Description
Analysis with defined support
Drawn By
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Scale
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Company
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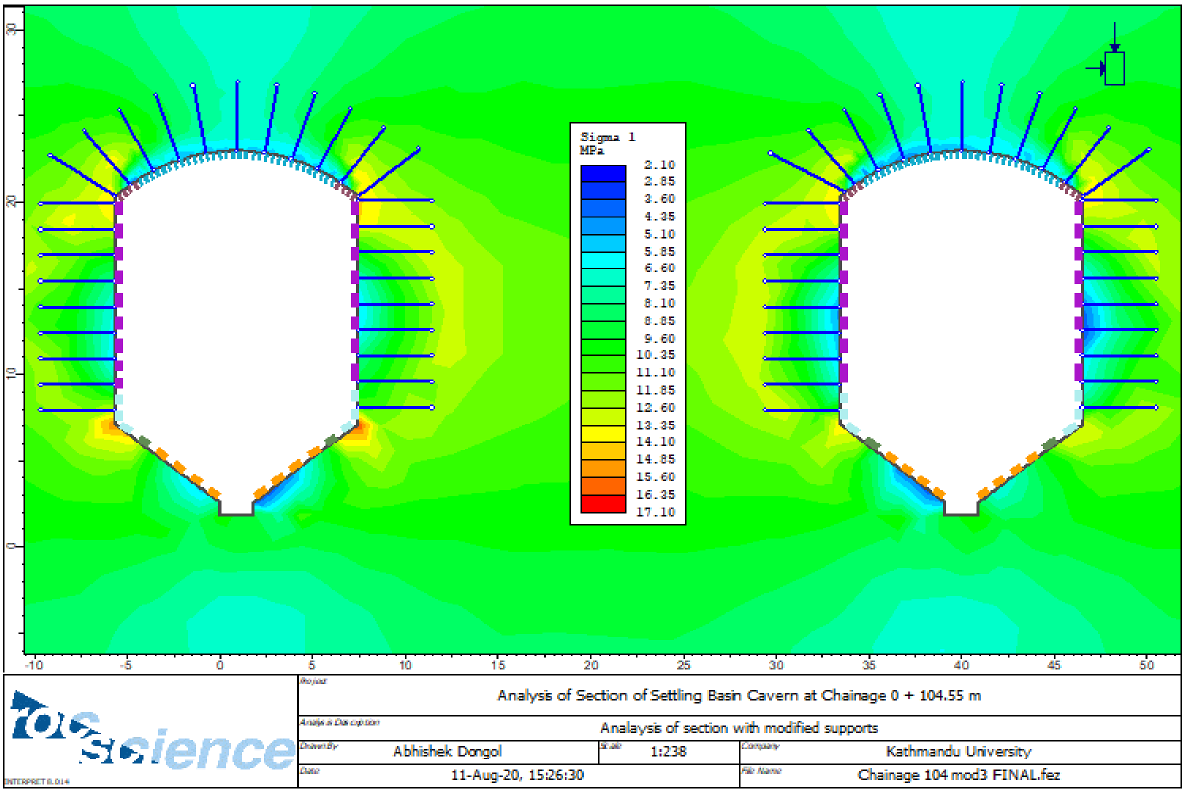
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Project
Analysis of Section of Settling Basin Cavern at Chainage 0 + 104.55 m
Analysis Description
Analaysis of section with modified supports
Drawn By
Abhishek Dongol
Scale
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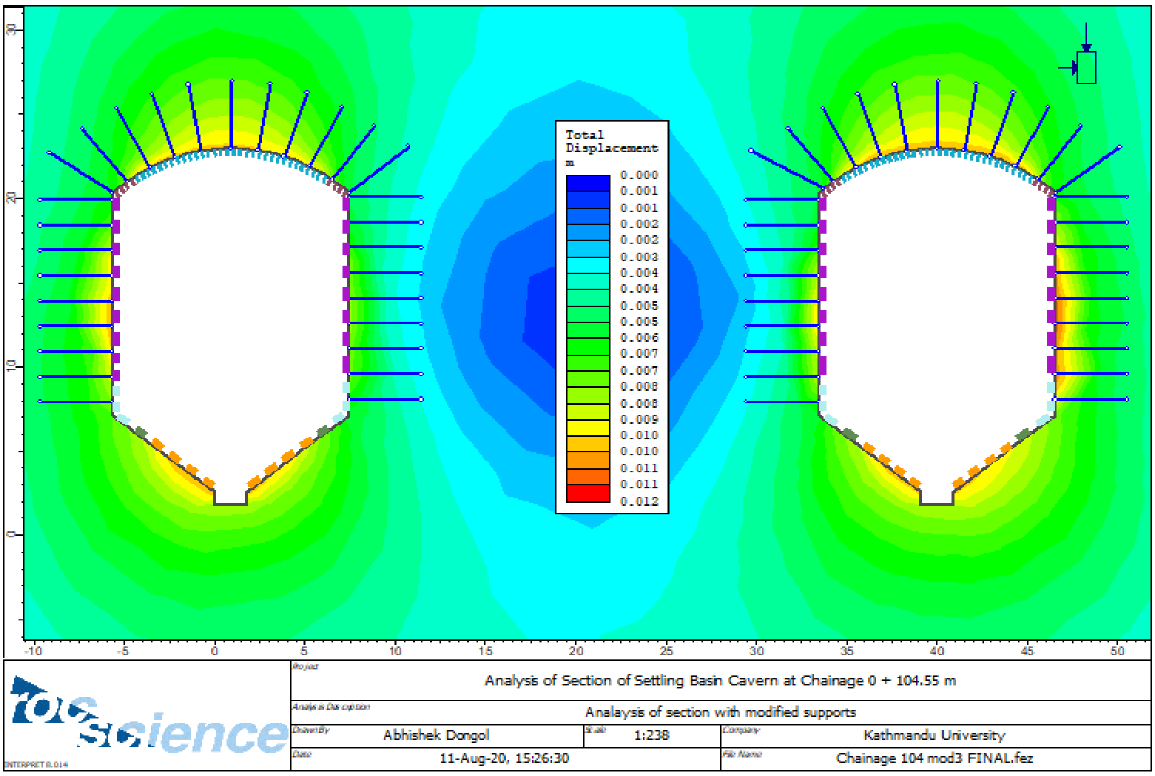
INTERPRET 8.014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Project
Analysis of Section of Settling Basin Cavern at Chainage 0 + 104.55 m
Analysis Description
Analaysis of section with modified supports
Drawn By
Abhishek Dongol
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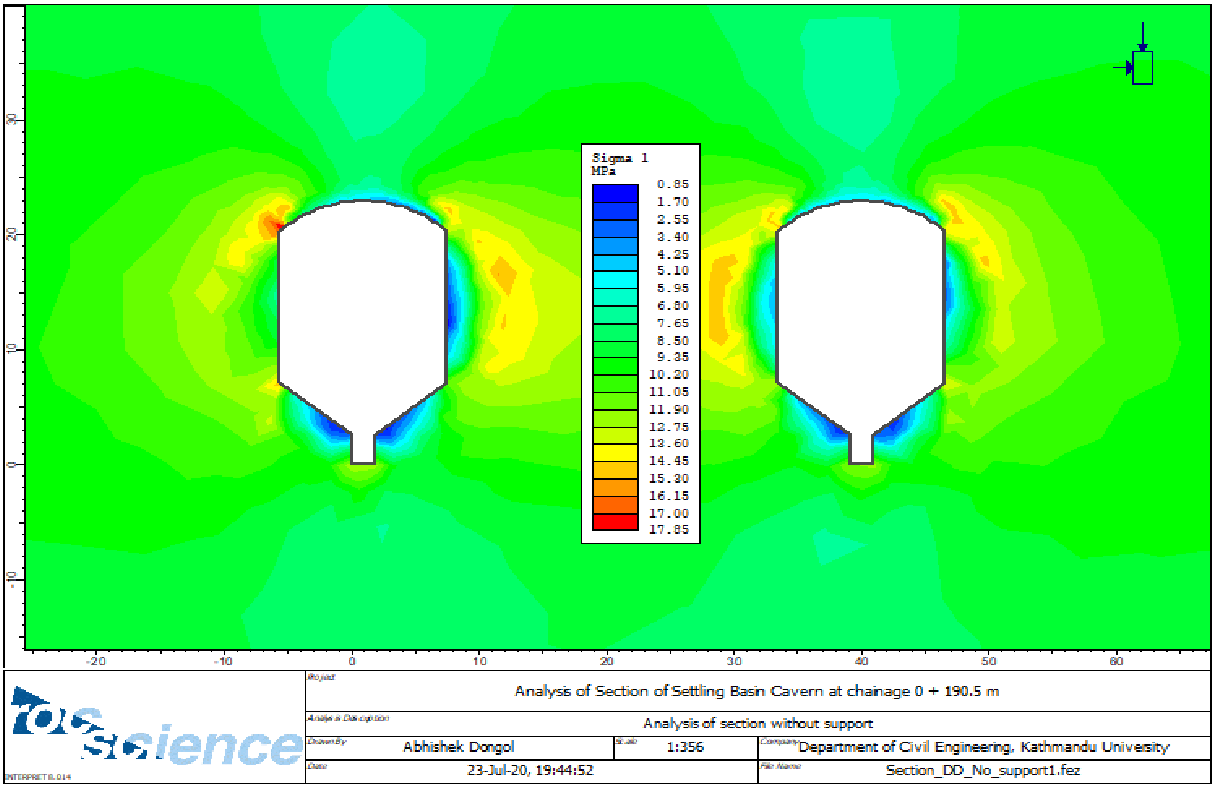
INTERPRET 8.014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Analysis of Section of Settling Basin Cavern at chainage 0 + 190.5 m
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Analysis of section without support
Drawn By
Abhishek Dongol
Scale
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Department of Civil Engineering, Kathmandu University
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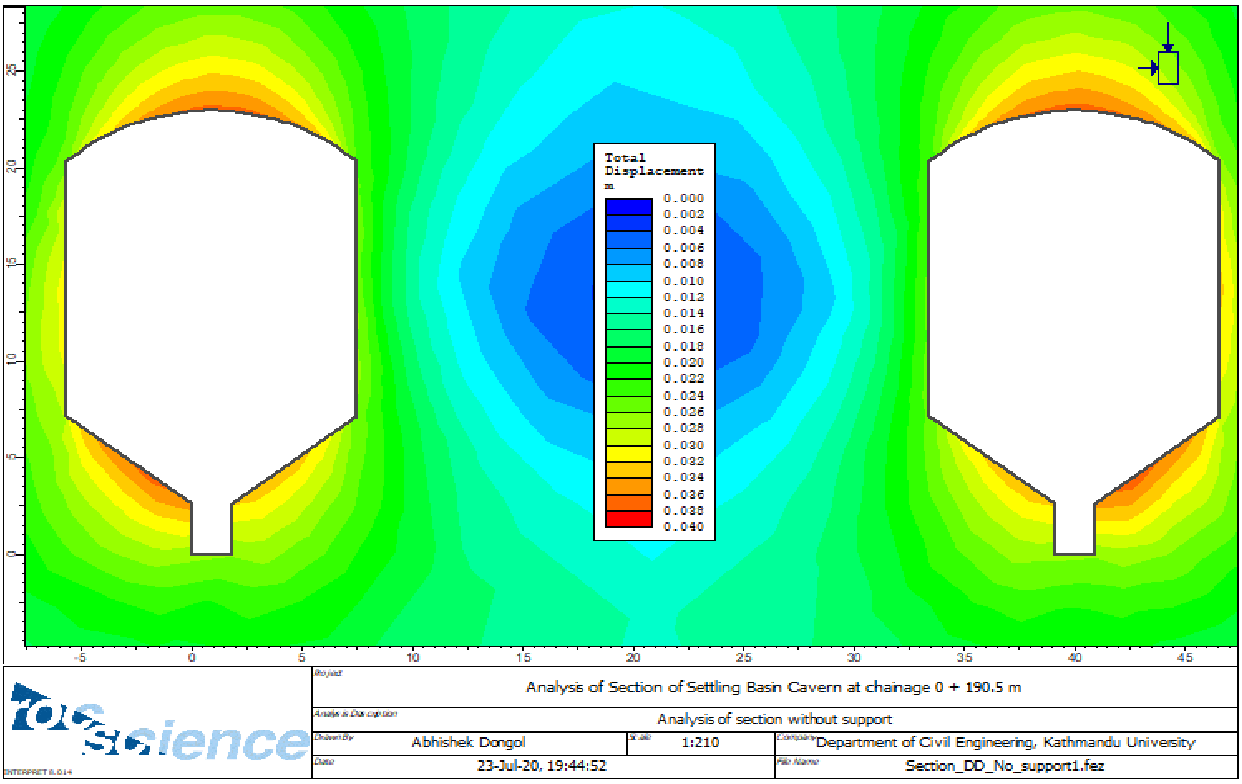
INTERPRET 6.014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0.028
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25
30
35
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Project
Analysis of Section of Settling Basin Cavern at chainage 0 + 190.5 m
Analysis Description
Analysis of section without support
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Scale
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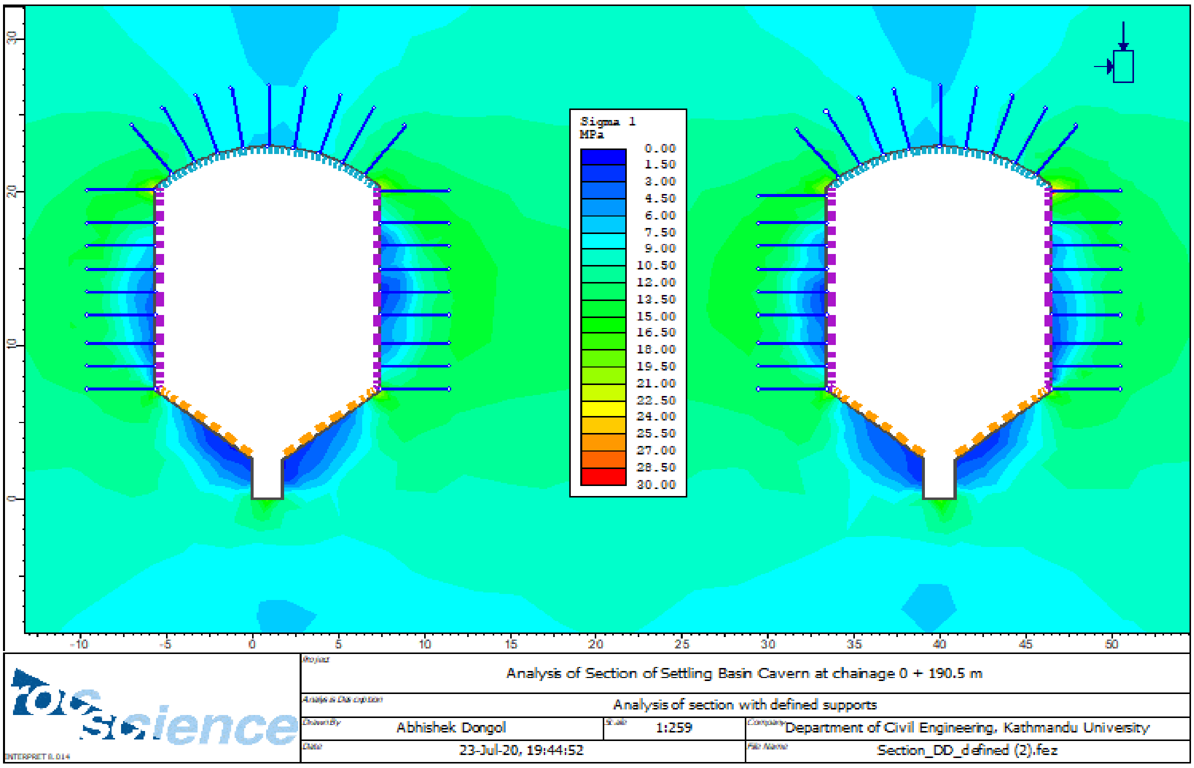
INTERPRET 6.014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16.50
18.00
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21.00
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27.00
28.50
30.00
Analysis of Section of Settling Basin Cavern at chainage 0 + 190.5 m
Analysis of section with defined supports
Abhishek Dongol
1:259
Department of Civil Engineering, Kathmandu University
23-Jul-20, 19:44:52
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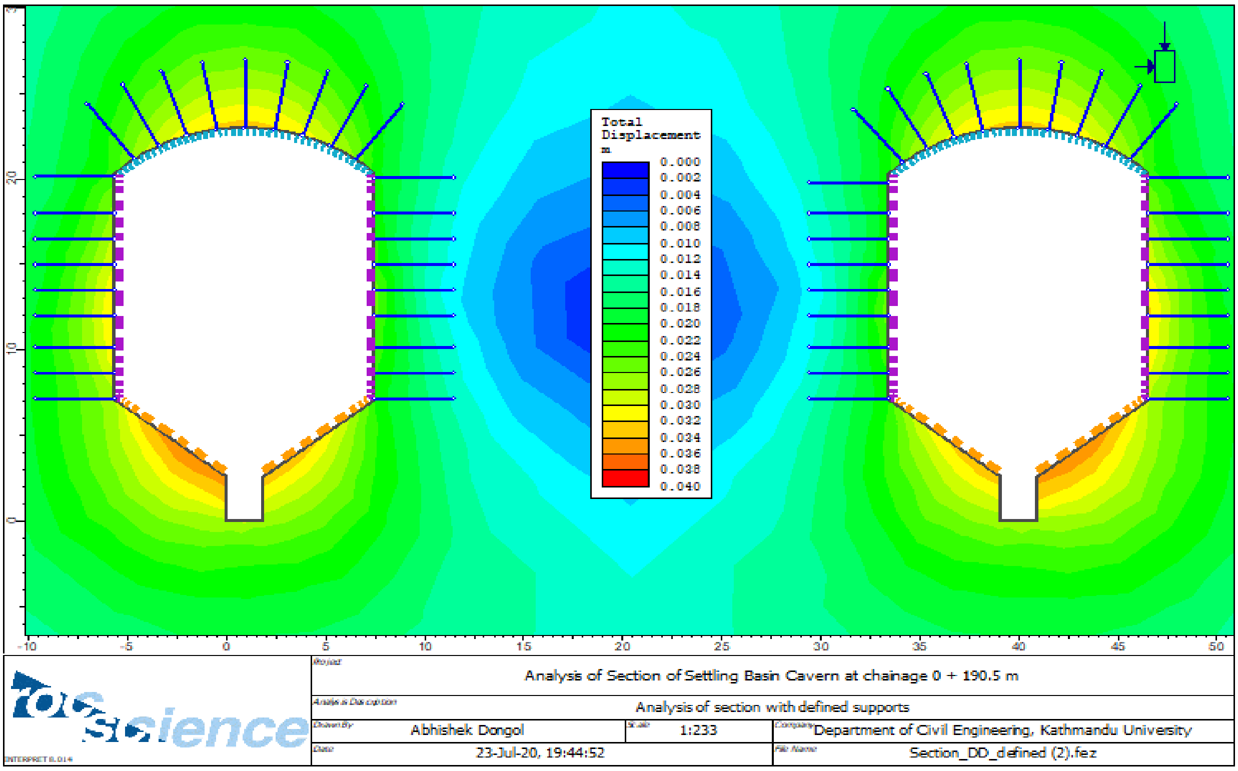
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0.024
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0.028
0.030
0.032
0.034
0.036
0.038
0.040
Analysis of Section of Settling Basin Cavern at chainage 0 + 190.5 m
Analysis of section with defined supports
Abhishek Dongol
1:233
Department of Civil Engineering, Kathmandu University
23-Jul-20, 19:44:52
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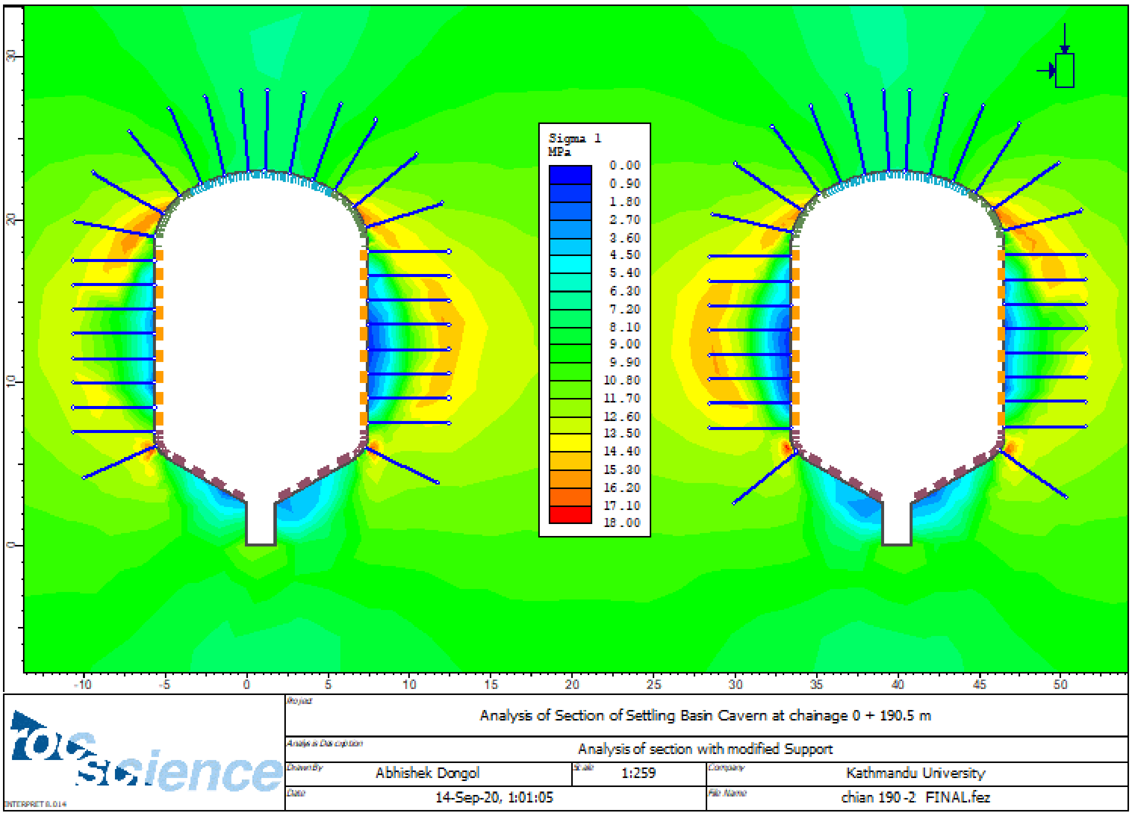
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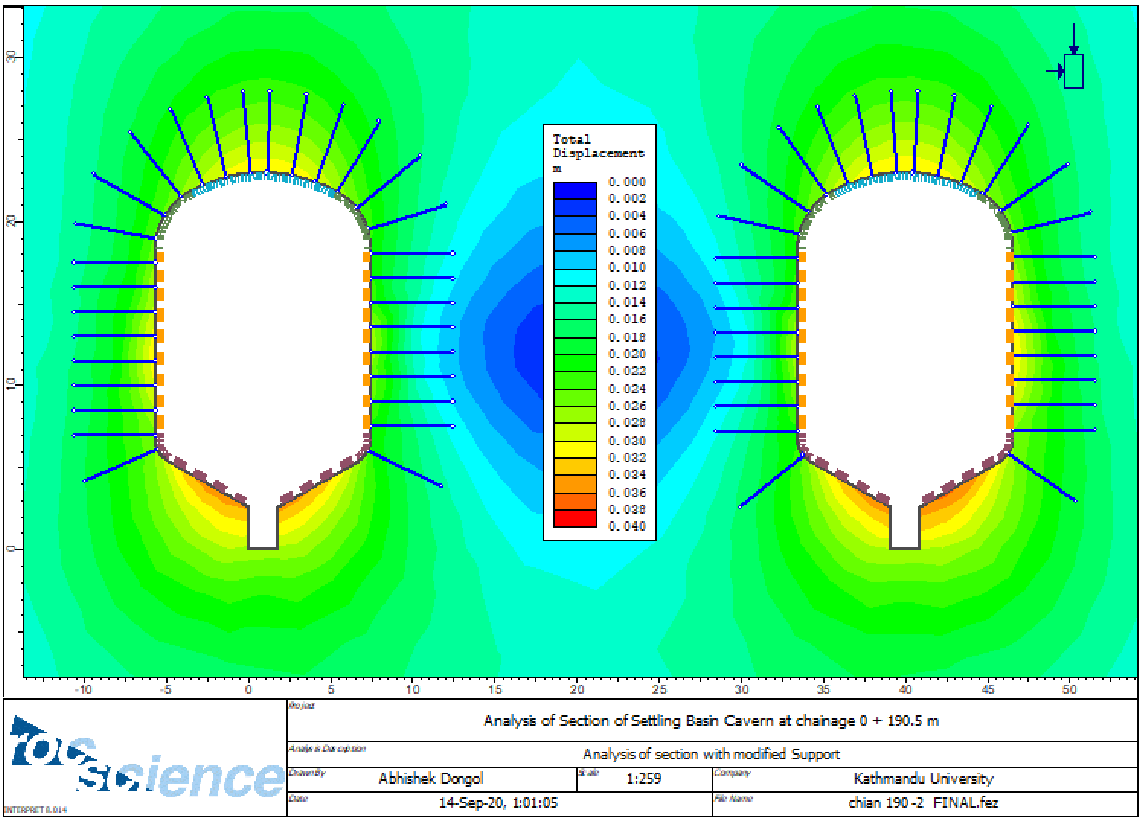
12.60
13.50
14.40
15.30
16.20
17.10
18.00
Analysis of Section of Settling Basin Cavern at chainage 0 + 190.5 m
Analysis of section with modified Support
Abhishek Dongol
1:259
Kathmandu University
14-Sep-20, 1:01:05
chian 190 -2 FINAL.fez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0.038
0.040
Analysis of Section of Settling Basin Cavern at chainage 0 + 190.5 m
Analysis of section with modified Support
Abhishek Dongol
1:259
Kathmandu University
14-Sep-20, 1:01:05
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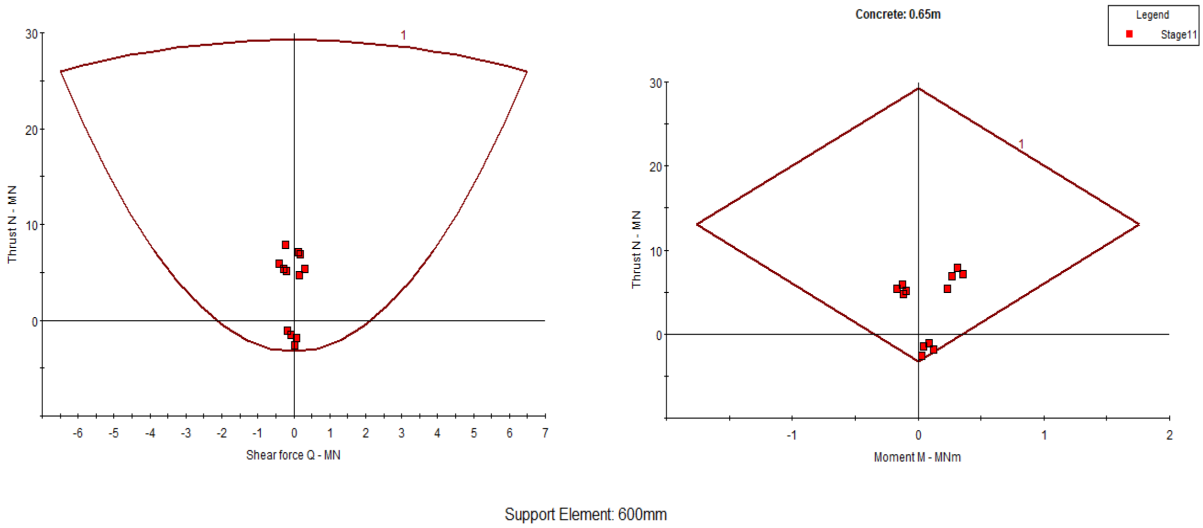
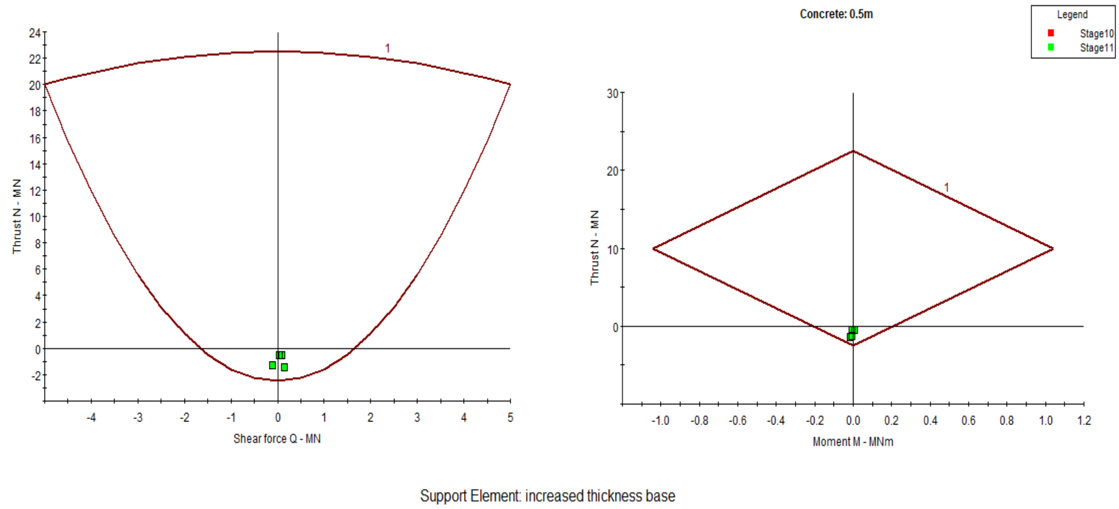
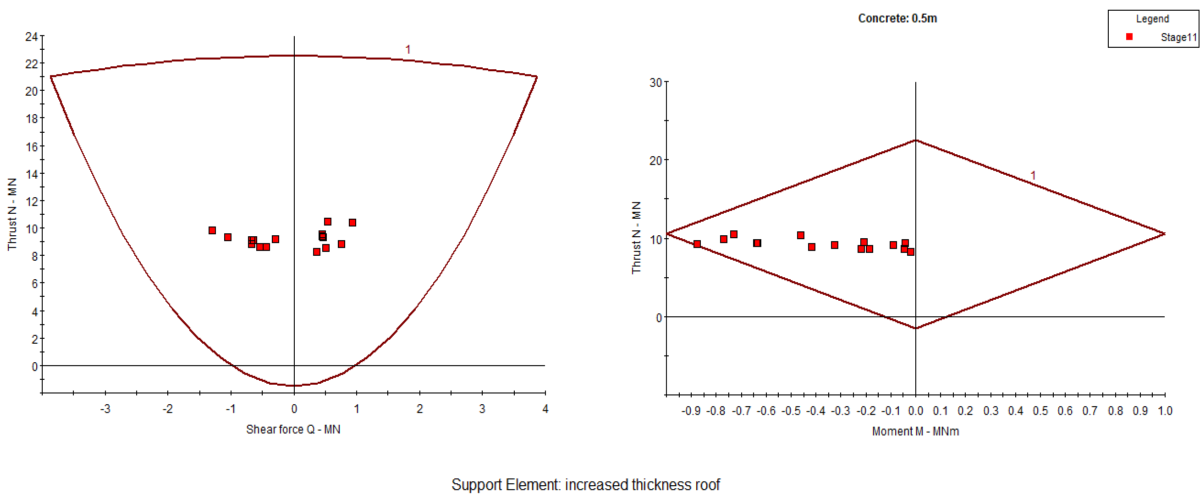


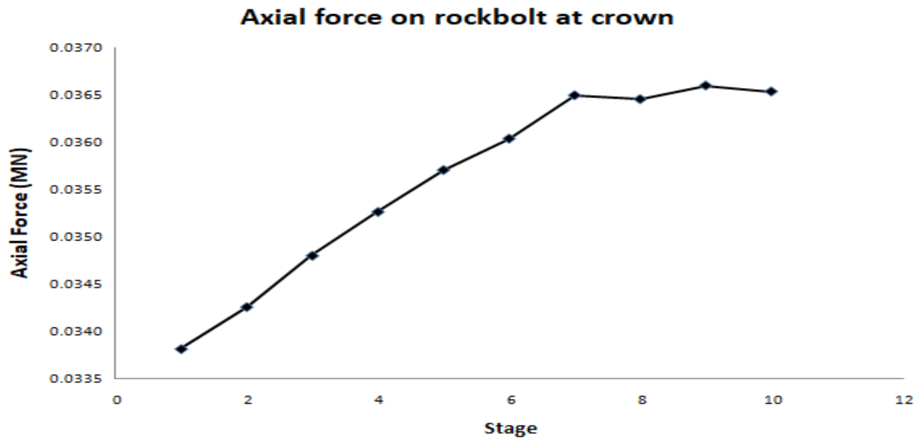
Support Capacity Plot for 650mm thick concrete liner at the bottom corner (chainage 0 + 104.55 m)



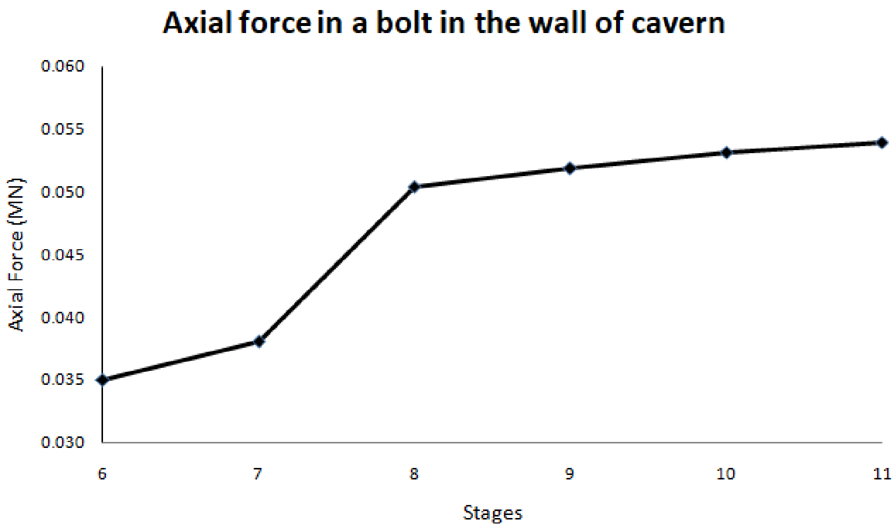
Support Capacity plot for 500mm thick concrete lining at base (chainage 0 + 104.55 m)



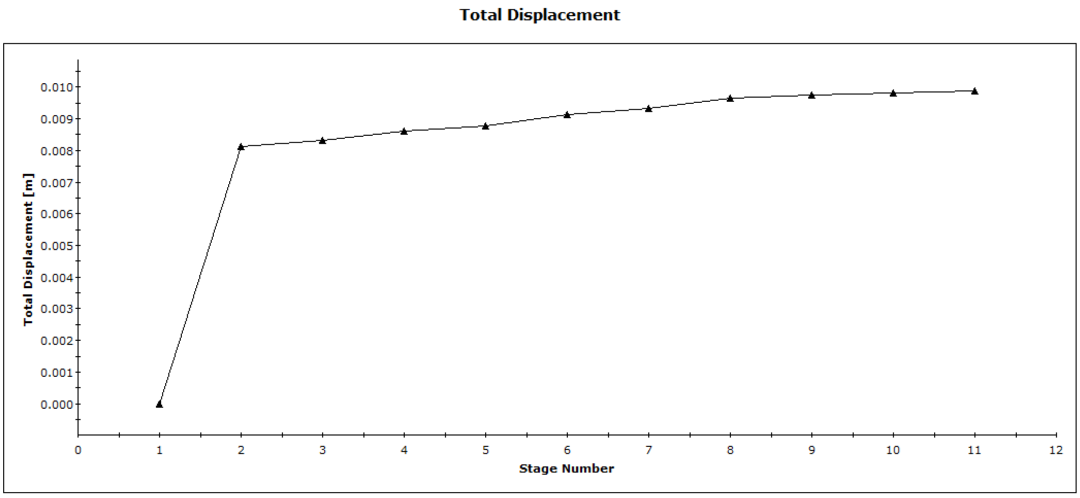
Support capacity plot for 500 mm concrete at roof corner (chainage 0 + 104.55 m)



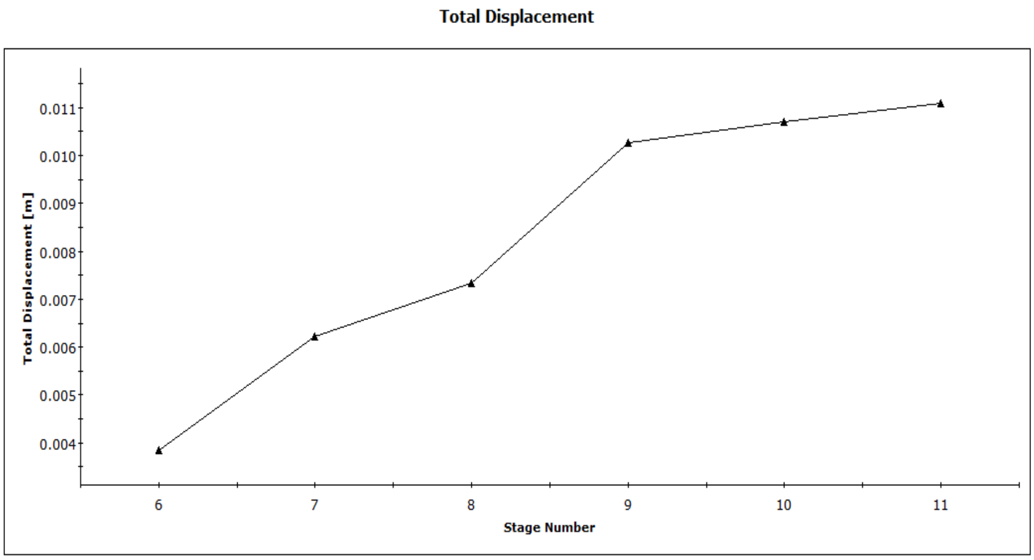
Axial force on a rockbolt at the crown of left bay (chainage 0 + 104.55 m)



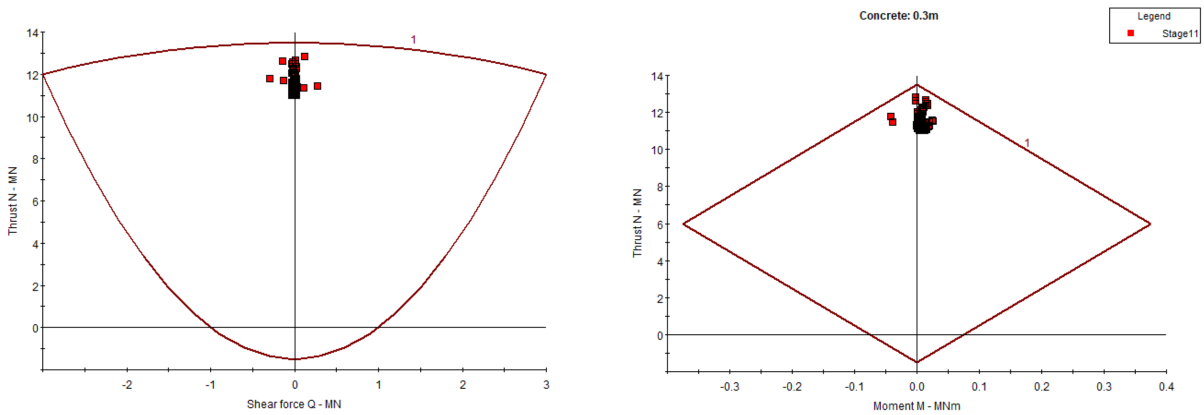
Axial force in a bolt installed at stage 6 at the wall of the left bay v/s stage (Chainage 0 + 104.55m)



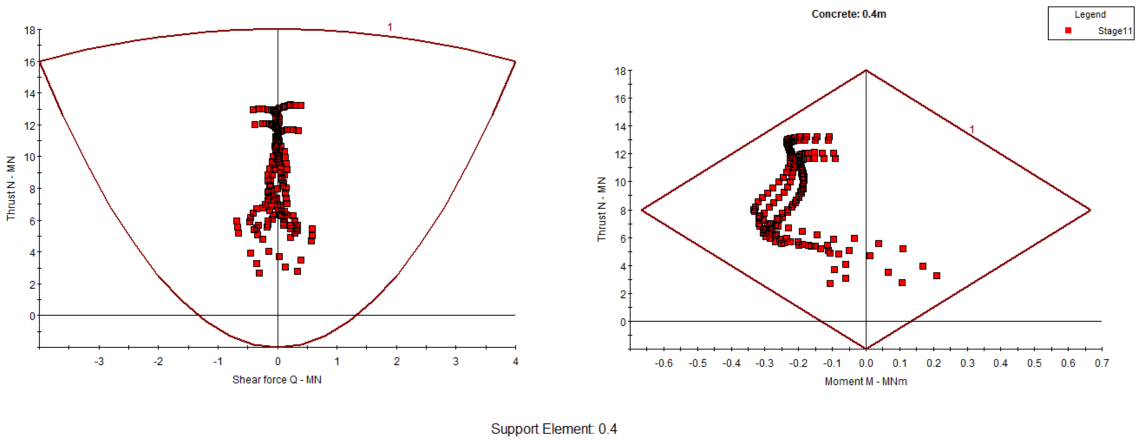
Displacement of crown of left bay v/s stage (Chainage 0 + 104.55m)



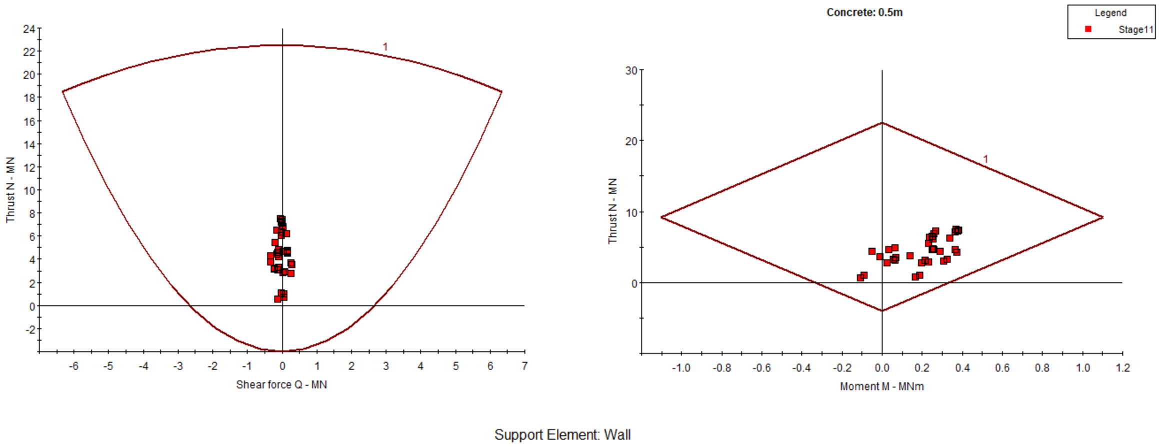
Displacement of wall of left bay v/s stage (Chainage 0 + 104.55m)



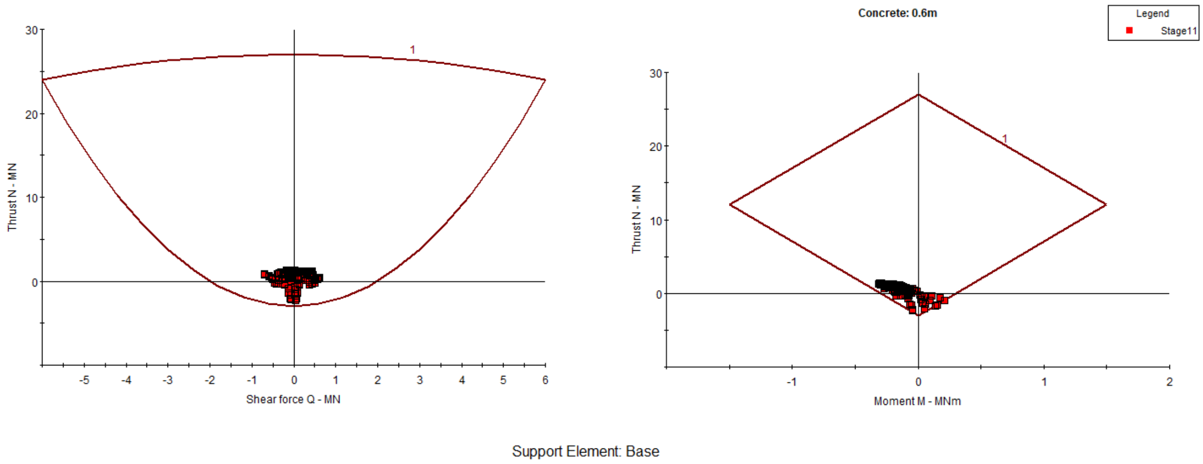
Support Capacity Plot for 300 mm thick liner at crown (chainage 0 + 190.5 m)



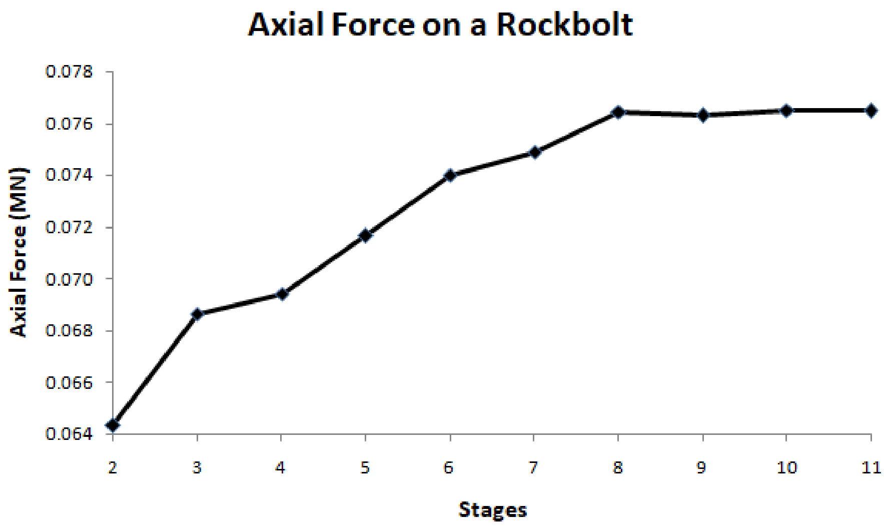
Support Capacity Plot for 400 mm concrete lining at corner (chainage 0 + 190.5 m)



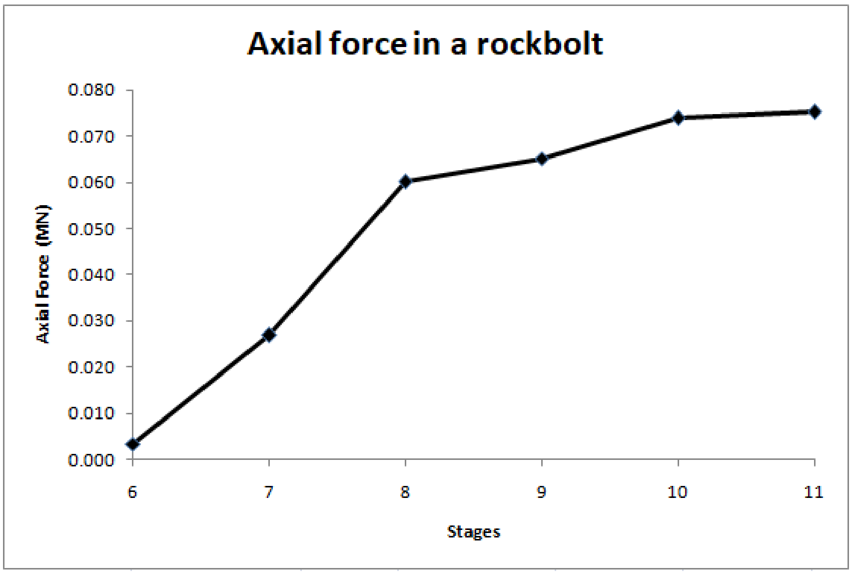
Support Capacity Plot for 500 mm wall concrete (chainage 0 + 190.5 m)



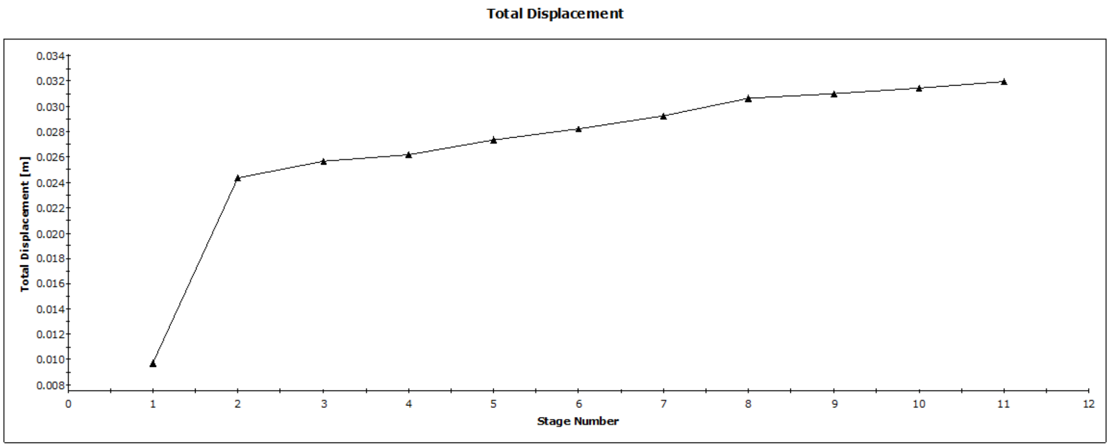
Support Capacity Plot for 600 mm thick concrete at base(chainage 0 + 190.5 m)



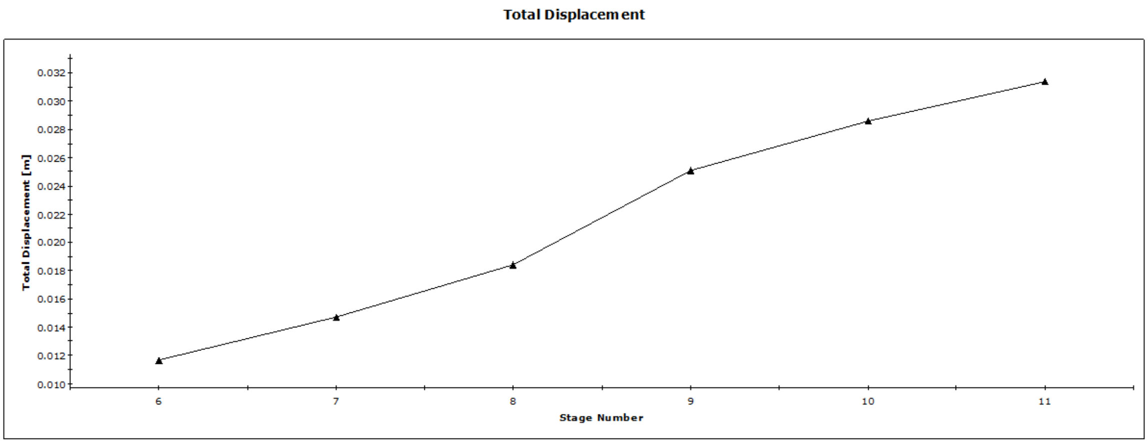
Axial Force on a Rockbolt at the crown (chainage 0 + 190.5 m)



Axial Force on a Rockbolt at the wall (chainage 0 + 190.5 m)



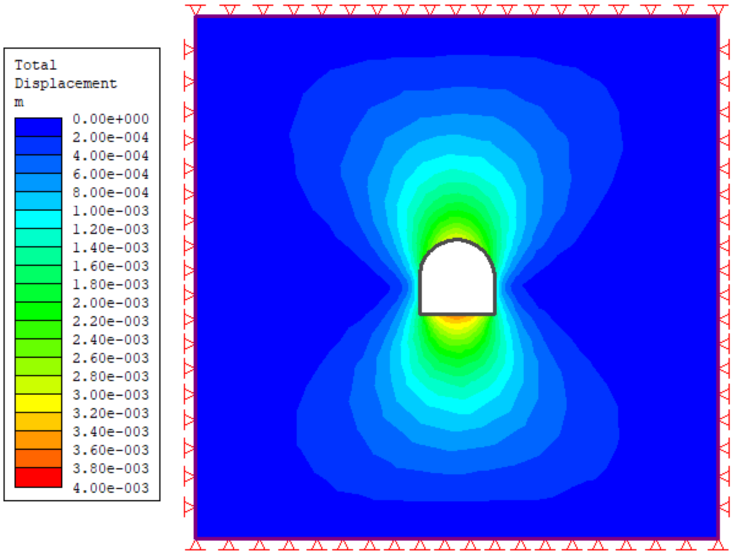
Displacement of crown vs stage (chainage 0 + 190.5 m)



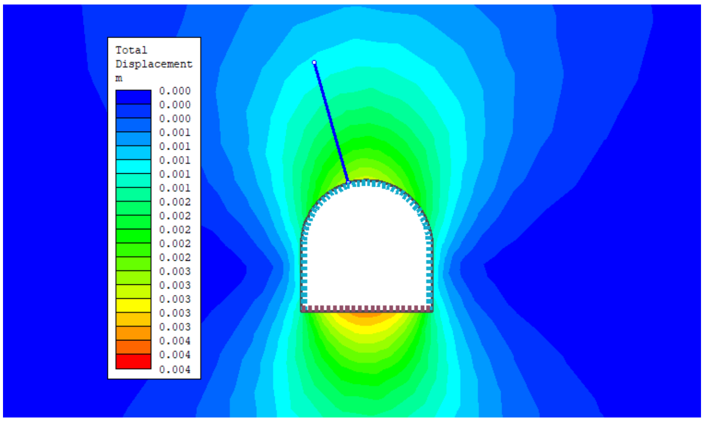
Displacement of Wall vs stage (chainage 0 + 190.5 m)

Annex C: Results of Numerical modeling of tunnel

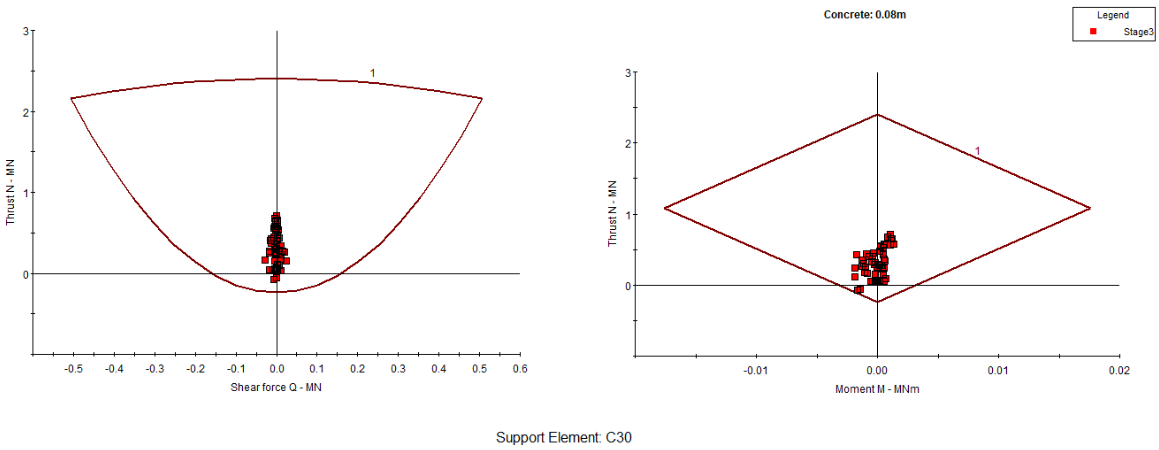
Numerical Analysis for chainage 0+058.15 using Generalized-Hoek-Brown



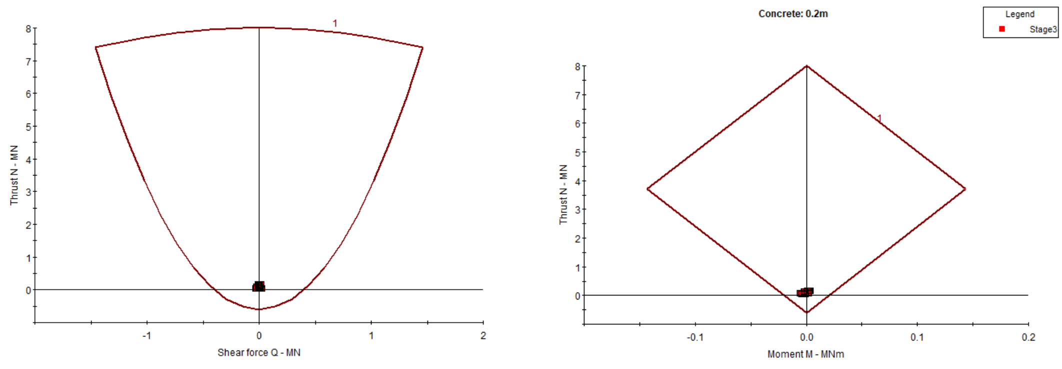
Total displacement before installation of support



Total displacement after installation of support



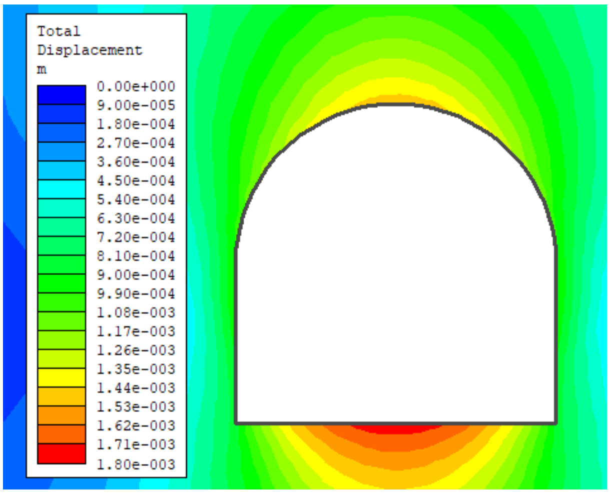
Support capacity curve of chainage 0+058.15



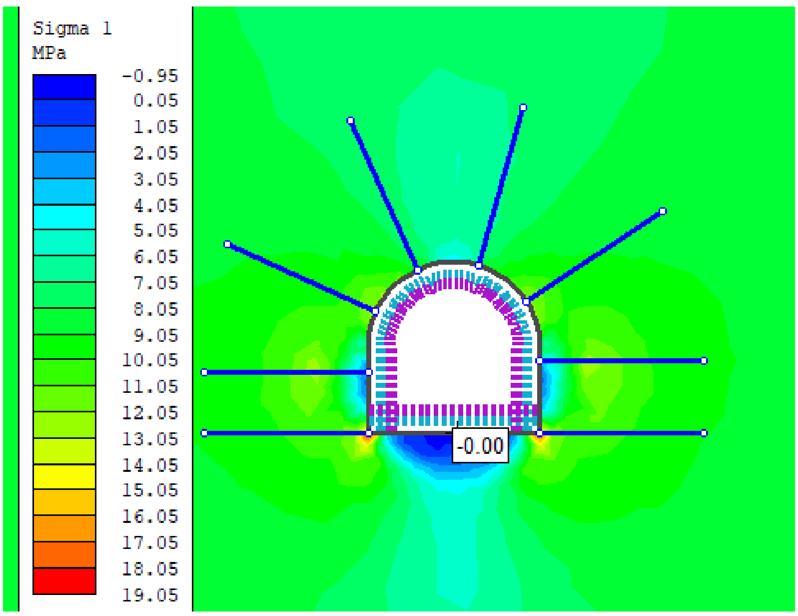
Support Element: invert 200 mm

Support capacity curve of chainage 0+058.15

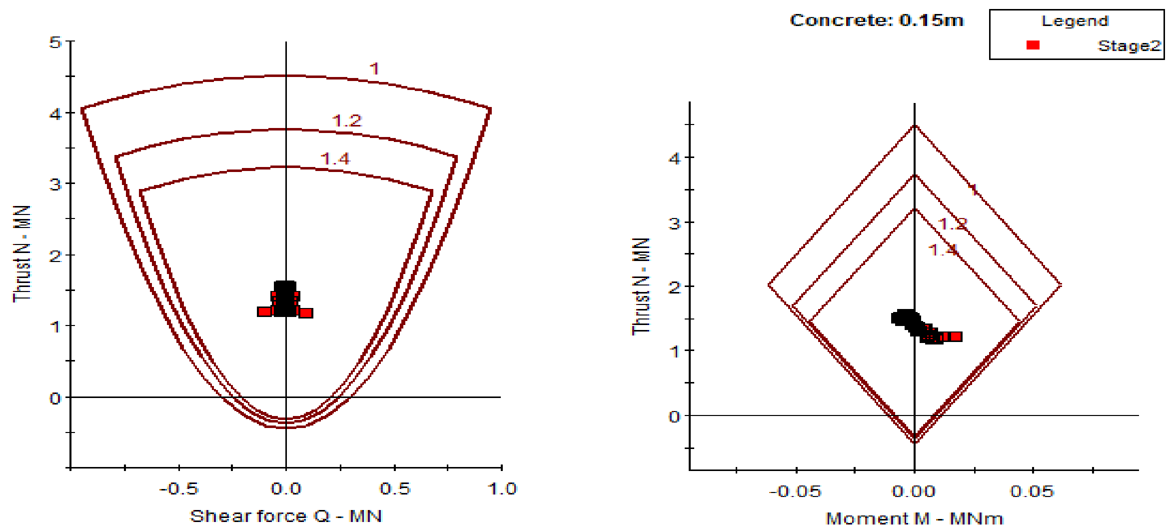
Numerical Analysis for chainage 0+101.00 using Generalized-Hoek-Brown



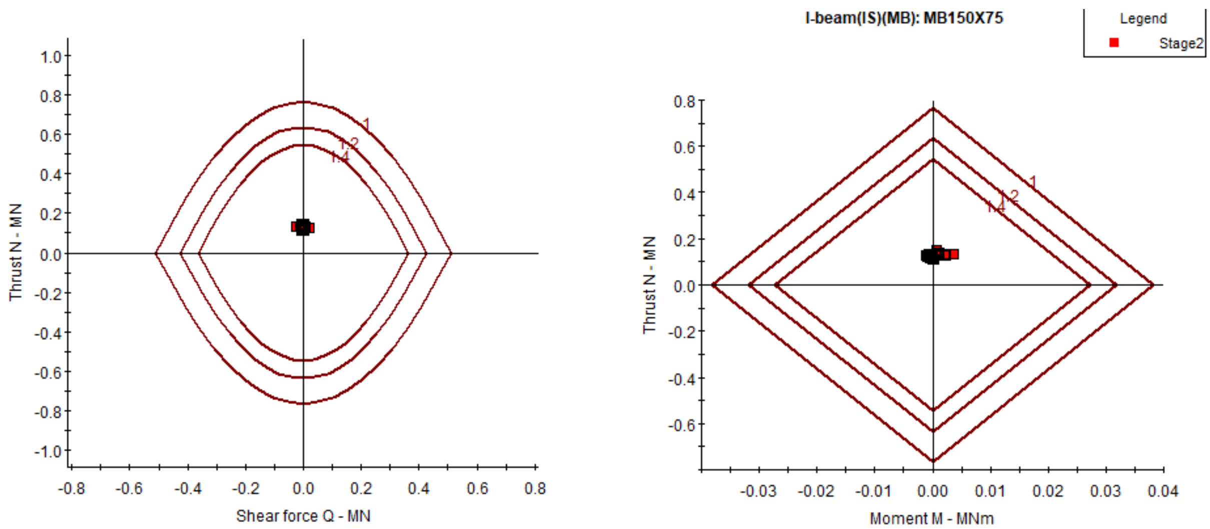
Total displacement before installation of support



Sigma 1 after installation of support



Support capacity curve of chainage 0+101.00



Support capacity curve of chainage 0+101.0

Annex D: Estimation of Quantities

Estimation of Quantities

ESTIMATION OF QUANTITIES							
1	**EARTHWORK**						
I/No.	**Description of works**	**Nos.**	**Length (L) in m**	**Breadth (B) in m**	**Height (H) in m**	**Quantity in cu.m.**	**Remark**
1.1	**EARTHWORK IN EXCAVATION BELOW GROUND LEVEL** The excavation includes mat foundation, isolated foundation depth and depth up to the draft tube level. Excavation at the edges is carried out in slope of 1:3 for stability.						
	Earthwork in excavation	1	15	16	5.8	**1392**	
		1	31	16	11	**5456**	
		16	2	2	2.35	**150.4**	
TOTAL						**6998.4 cu.m**	

3	**PCC WORKS** Providing, laying , compacting & curing plain cement Concrete M15 (1:2:4) for RCC footings foundation mass concreting with thickness of 100 mm						
I/No.	**Description of works**	**Nos.**	**Length (L) in m**	**Breadth (B) in m**	**Height (H) in m**	**Quantity in cu.m.**	**Remark**
3.1	Mat foundation	1	46	16	0.1	73.6	
	SUB TOTAL					**73.6**	
3.2	Erection bay	1	15	16	0.1	24	
3.3	Machine bay	1	31	16	0.1	49.6	
	SUB TOTAL					**73.6**	
	TOTAL					**147.2**	

4	**FORMWORKS** Centering and shuttering with steel or water proof plywood material for all kinds of R.C.C. work including all necessary propping, scaffolding, staging, supporting etc. all complete as per drawing, specification, and instruction of site engineer							
I/No.	**Description of works**		**Nos.**	**Length (L) in m**	**Breadth (B) in m**	**Height (H) in m**	**Quantity in m^2**	**Remark**
4.1	**FOOTING**							
	Mat Foundation	RCC	1	15		0.85	12.75	
			1	31		0.85	26.35	
	SUB TOTAL						**39.1**	
4.2	**BELOW GROUND LEVEL**							
COLUMN								
i	Column(1m*1m)		9	4		5.8	208.8	

	Column(1m*1m)	14	4		11	616	
	SUB TOTAL					**824.8**	
SHEAR WALL							
i	Shear wall(X axis)	2	15		5.8	174	Erection Bay
Ii	Shear wall(X axis)	2	31		11	682	Machine Hall
v.	Shear wall(Y-axis)	2	16		5.8	185.6	Erection Bay
	SUB TOTAL					**1041.6**	
4.3	**ABOVE GROUND LEVEL**						
COLUMN							
i	Column(1m*1m)	23	4		4	368	From Ground level to first floor
	Column(1m*1m)	20	4		4.5	360	From first floor to second floor
	Column(0.5m*0.5m)	20	2		5	200	From second floor to top
	Column(0.4m*0.4m)	16	1.6		4.5	115.2	Control Unit
	SUB TOTAL					**1043.2**	
BEAM							
i.	Beam(0.4m*0.6m) X-axis	36	8	2		576	Machine Hall/Erection bay
	Beam(0.25m*0.4m) X-axis	12	8	1.3		124.8	Machine Hall/Erection bay
ii.	Beam(0.4m*0.6m) Y-axis	12	7.6	2		182.4	Machine Hall/Erection bay
	Beam(0.25m*0.4m) Y-axis	4	7.6	1.3		39.52	Machine Hall/Erection bay
iii.	Beam(0.25m*0.4m) X-axis	12	6.5	1.3		101.4	Control Unit
	Beam(0.25m*0.4m) Y-axis	12	5.5	1.3		85.8	
	SUB TOTAL					**1109.92**	
	TOTAL					**4058 sq.m**	

		Nos.	Length(L) in m	Breadth(B) in m	Height(H) in m	Quantity in cu.m.	Remark
5	**RCC WORKS**						
5.1	**FOOTING**						
i.	Mat foundation	1	46	16	1	736	
	SUB TOTAL					**736**	
5.2	**BELOW GROUND LEVEL**						
	COLUMN						
i.	Column(1m*1m)	9	1	1	5.8	52.2	
ii	Column(1m*1m)	14	1	1	11	154	
	SUB TOTAL					**206.2**	
	SHEAR WALL						
i.	Shear wall(X axis)	2	15	0.45	5.8	78.3	Erection Bay
ii.	Shear wall(X axis)	2	31	0.8	11	545.6	Machine Hall
v	Shear wall(Y-axis)	2	16	0.45	5.8	83.52	Erection Bay
	SUB TOTAL					**707.42**	
5.3	**ABOVE GROUND LEVEL**						
	COLUMN						
i.	Column(1m*1m)	23	1	1	5.8	133.4	From Ground level to first floor
	Column(1m*1m)	20	1	1	4	80	From first floor to second floor
	Column(1m*1m)	20	1	1	4.5	90	From second floor to corvel
	Column(0.5m*0.5m)	24	0.5	0.5	5	30	From crane beam to truss level
Ii	Column(0.4m*0.4m)	16	0.4	0.4	4.5	11.52	Control Unit
	SUB TOTAL					**344.92**	

BEAM							
i.	Beam(0.4m*0.6m) X-axis	36	8	0.4	0.6	69.12	Machine Hall/Erection bay
	Beam(0.25m*0.4m) X-axis	12	8	0.25	0.4	9.6	Machine Hall/Erection bay
ii.	Beam(0.4m*0.6m) Y-axis	12	7.6	0.4	0.6	21.888	Machine Hall/Erection bay
	Beam(0.25m*0.4m) Y-axis	4	7.6	0.25	0.4	3.04	Machine Hall/Erection bay
	Beam(0.25m*0.4m) X-axis	12	6.5	0.25	0.4	7.8	Control unit
	Beam(0.25m*0.4m) Y-axis	12	5.5	0.25	0.4	6.61	
	SUB TOTAL					**118.048**	
	TOTAL					**1965.772**	

6	**BRICKWORK** Providing & laying local chimney made brick masonry work in cement, sand mortar (1:5) in superstructure finished in perfect line & level including wetting the bricks, packing the joints & curing the work complete in all thickness of walls as per drawings						
6.1	**Machine Hall and Erection Bay**						
i	Exterior walls	2	31	0.23	13.5	192.51	Along X-axis in Machine Hall
		2	15	0.23	13.5	93.15	Along X-axis in Erection Bay
		1	16	0.23	13.5	49.68	Along Y-axis in Machine Floor
		1	16	0.23	13.5	49.68	Along Y-axis in Erection Bay
	Deduction					38.50	10% for doors and windows
	Sub Total					**385.02**	
	Total					**346.518**	

7	**PLASTERING** Providing, laying & curing 12.5 mm thick cement sand (1:3) plastering in ceiling, beams and column surfaces including chipping & wetting the concrete surfaces finished in perfect plumb, lines and level as per drawings, specifications and instructions of the site engineer

I/No	Description of works	Nos.	Length (L) in m	Breadth (B) in m	Height (H) in m	Quantity in sq.m.	Remark
	INSIDE						
i.	Shear wall(X axis)	2	15		5.8	174	Erection Bay
ii.	Shear wall(X axis)	2	31		11	682	Machine Hall
iv.	Shear wall(Y-axis)	2	16		5.8	185.6	Erection Bay
v.	Machine Hall		Perimeter =	94	24.5	2303	With Projections
vi.	Erection Bay		Perimeter =	62	19.3	1196	
vii.	Top and Bottom of Beams						With Projections
a.	Beam(0.4m*0.6m) X-axis	36	8	0.4		115.2	Machine Hall/Erection bay
	Beam(0.4m*0.6m) Y-axis	12	8	0.25		24	Machine Hall/Erection bay
b.	Beam(0.25m*0.4m) X-axis	12	7.6	0.4		36.48	Machine Hall/Erection bay
	Beam(0.25m*0.4m) Y-axis	4	7.6	0.25		12.16	Machine Hall/Erection bay
	Beam(0.25m*0.4m) X-axis	12	6.5	0.25		19.5	Control Unit
	Beam(0.25m*0.4m) Y-axis	12	5.5	0.25		16.50	
	SUB TOTAL					**4667.42**	

			OUTSIDE				
i.	Machine Hall	1	31		24.5	759.5	
ii.	Erection Bay	1	15		19.3	289.5	
Iii	Control Unit	1	16.6		4.5	74.7	
	SUB TOTAL					**1123.7**	
	TOTAL					**5791.12**	

8	**REINFORCEMENT** Work including straightening, cleaning, cutting, bending, binding with 20 SWG annealed wire & fixing in position as per drawing, bar bending schedule for raft foundation column, beam, wall, stair, slab in all R.C.C. works as per specification, drawing & instruction of site engineer.							
		Diameter of bar	**Nos. of Bars**	**L**	**c/s area of bar**	**Volume (m^3)**	**Density (kg/m^3)**	**Weight(kg)**
8.1	**Mat Foundation**							
i.	Top along X	16	418	46	0.000201	0.0093	7850	73.005
ii	Bottom along X	16	209	46	0.000201	0.0093	7850	73.005
iii	Top along Y	16	145	16	0.000201	0.0032	7850	25.12
iv	Bottom along Y	16	72	16	0.000201	0.0032	7850	25.12
	Sub Total							**196.25**

8.2	**COLUMNS**							
	Column ID	**Reinforcement Type**	**Nos. of Bars**	**Nos. of Columns**	**Length(m)**	**Diameter (mm)**	**Density (kg/m^3)**	**Weight(kg)**
i	(1m*1m)	Longitudinal	16	63	19.5	12-32 mm 4-25mm	7850	94672.44

		Lateral Ties	210	63	3.5	8	7850	18271.21
ii	(0.5m*0.5m)	Longitudinal	12	20	5	25	7850	4624.03
		Lateral Ties	70	20	2	8	7850	1104.83
iii	(0.4m*0.4m)	Longitudinal	8	16	4.5	16	7850	909.12
		Lateral Ties	45	16	2	8	7850	568.20
	Sub Total							**120149.83**

8.3	**BEAMS**							
		Reinforcement Type	**Nos. of Bars**	**Nos. of Beams**	**Length of bars (m)**	**Diameter (mm)**	**Density (kg/m^3)**	**Weight(kg)**
i	Along X-axis	Bottom Bar	2	38	9	20	7850	1685.99
		Top Bar	4	38	9	20	7850	3371.983
		Stirrups	55	38	1.8	8	7850	1484.425
ii	Along Y-axis	Bottom Bar	2	14	7.6	20	7850	524.530
		Top Bar	5	14	7.6	20	7850	1311.3268
		Stirrups	28	46	1.8	8	7850	914.703
		Sub Total						**9292.657**

8.4	SHEAR WALL							
		Nos	**L(m)**	**No. of bar**	**Dia. of bar (mm)**	**Volume of Steel (m^3)**	**Density (Kg/m^3)**	**Weight (kg)**
a	Vertical Bar (outer face)	2	46	460	20	13.28	7850	104314.57
	Vertical Bar (inner face)	2	46	460	16	8.5	7850	66774.61
b	Horizontal Bar(outer face)	2	46	29	16	0.5	7850	4209.7
	Horizontal Bar(inner face)	2	46	23	12	0.23	7850	1876.99
c	Vertical Bar (outer face)	2	16	16	20	0.16	7850	1262.02
	Vertical Bar (inner face)	2	16	21	16	0.13	7850	1060.31
d	Horizontal Bar(outer face)	2	16	29	16	0.18	7850	1464.2
	Horizontal Bar(inner face)	2	16	23	16	0.14	7850	1161.3
	Sub Total							**182123.82**

8.5	CORBELS							
	Reinforcement Type	**Nos. of Corbels**	**Nos. of Bars**	**Dia. of Bars (mm)**	**Length (m)**	**Volume of Bar (m^3)**	**Density (kg/m^3)**	**Weight(kg)**
i.	Tension Bar	24	8	25	2.8	0.215	7850	1692.678
ii.	Shear Reinforcement	24	10	12	4.55	0.12	7850	969.5
	Sub Total							**2662.17**

9	ROOF TRUSS					
S.N	**Description**	**No.**	**Length(m)**	**Area of Cross Section(cm^2)**	**Unit Weight(kg/m)**	**Quantity(kg)**
	STEEL WORK					
i.	Purlins ISNB 110M (Machine Hall)	104	5.25	18.4	14.5	3074.4
iii.	Top Chords ISNB 100M	108	1.37	15.5	12.20	2012.67
iv.	Bottom Chords ISNB175M	108	1.33	26.260	25.1	5516.5
v.	Struts ISNB 100M(Sum)	45	2.4	15.5	12.20	9684.650
vi.	Struts ISNB 100M(Sum)	45	2.4	15.5	12.20	11954
	TOTAL					**33559.82**
	ROOF COVERING					
SN	**Description**	**No.**	**Length(m)**	**Breadth(m)**	**Unit Weight(kg/m^2)**	**Quantity(m^2)**
i	CGI Sheet		46	16		**736**

ABSTRACT OF ESTIMATED COST

Item No	Particulars of Item	Quantity	Unit	Rate	Per	Rs
1	Earthwork in Excavation	6998.4	m^3	221.09	m^3	15,47,276.256
2	Formwork Plain Ordinary	4058	m^2	539.06	m^2	21,87,505.48
3	Ordinary PCC works	147.2	m^3	8626.38	m^3	12,69,803.136
4	12 mm thick plastering (1:4)	5791.12	m^2	366.44	m^2	21,22,098.013
5	Concrete M25 for R.C.C	1965.772	m^3	22201.9	m^3	4,36,43,873.4
6	Brickworks in 1:4 cement mortar	346.518	m^3	17238.5	m^3	59,73,450.54
7	Reinforcement Steel	70	ton	115345	ton	80,74,150
8	Roof and Side Covering	736	m^2	898.59	m^2	6,61,362.24
TOTAL						**6,54,79,519.07**
Add 3% Contingencies						**19,64,385.572**
Add 2% Work Charged Establishment						**13,09,590.381**
GRAND TOTAL						**6,87,53,495.02**

Annex E: Detailed Drawings of Powerhouse

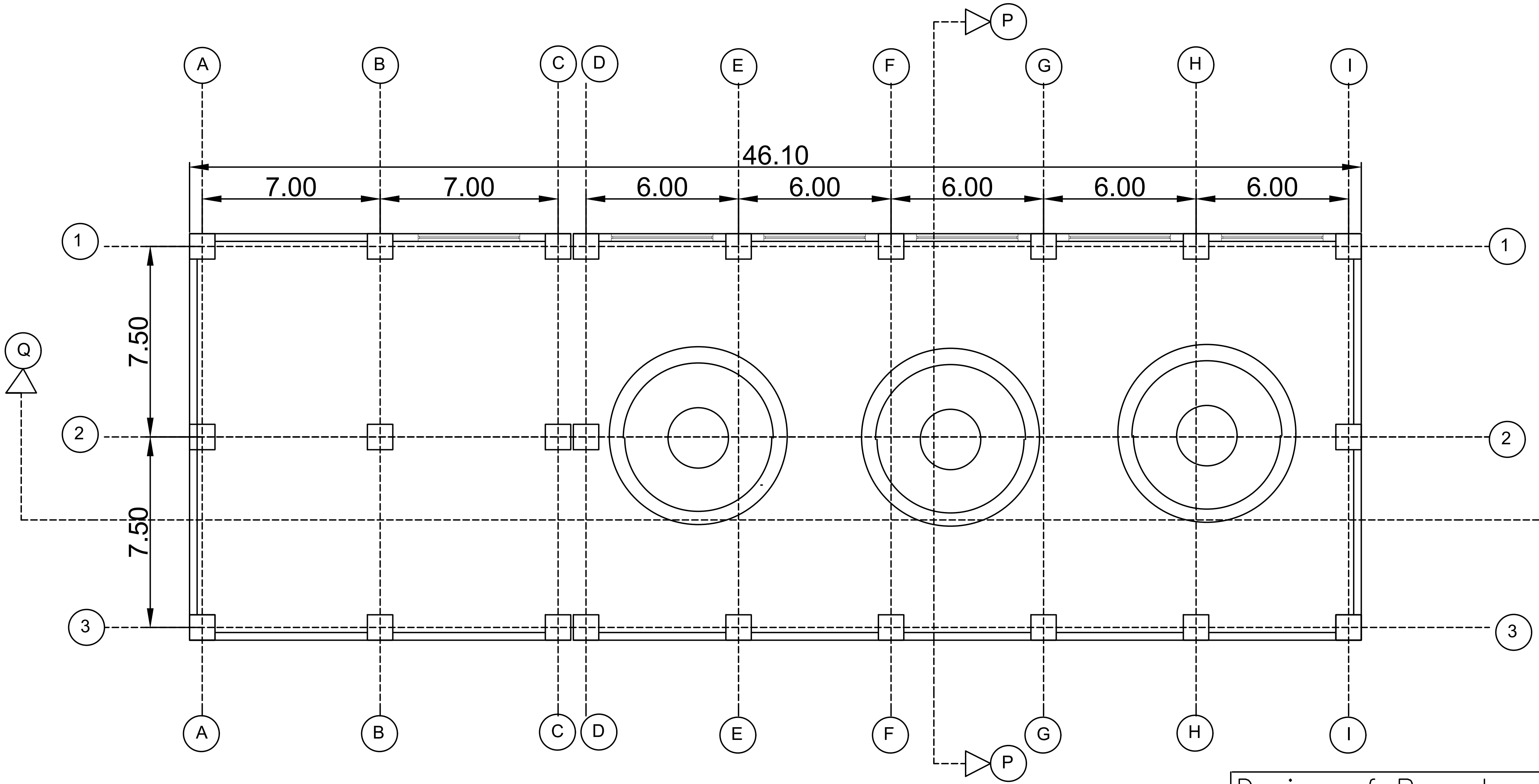
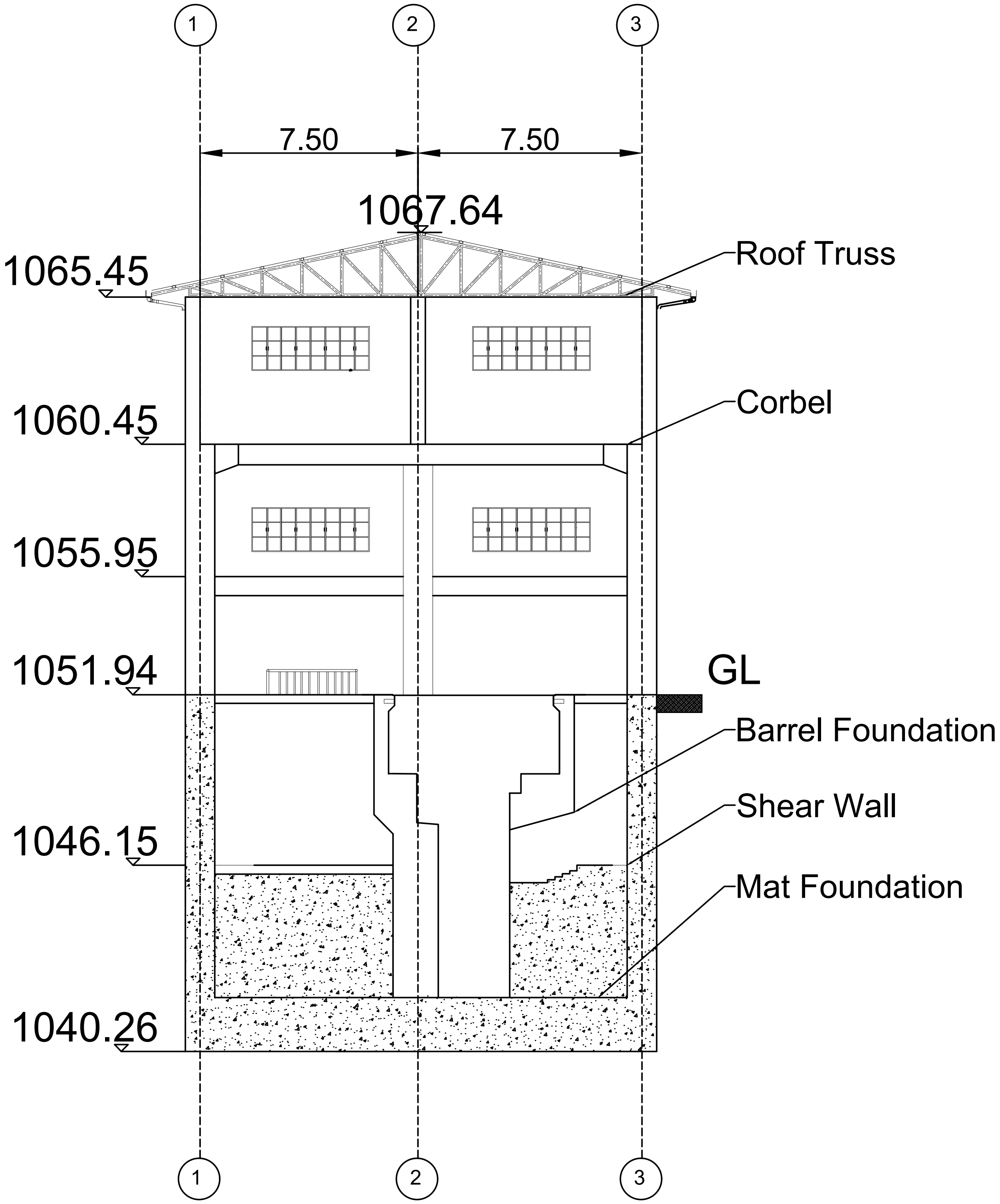


Fig: Plan of SuperMadi Hydroelectric Plant
Scale: 1:210

Notes:

1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Strucutre			
Powerhouse Drawing Plan		Designed	Group 2
		Drawn	Group 2
		Checked	
	KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre		Scale : As shown
			Sheet No: 1
			Revision

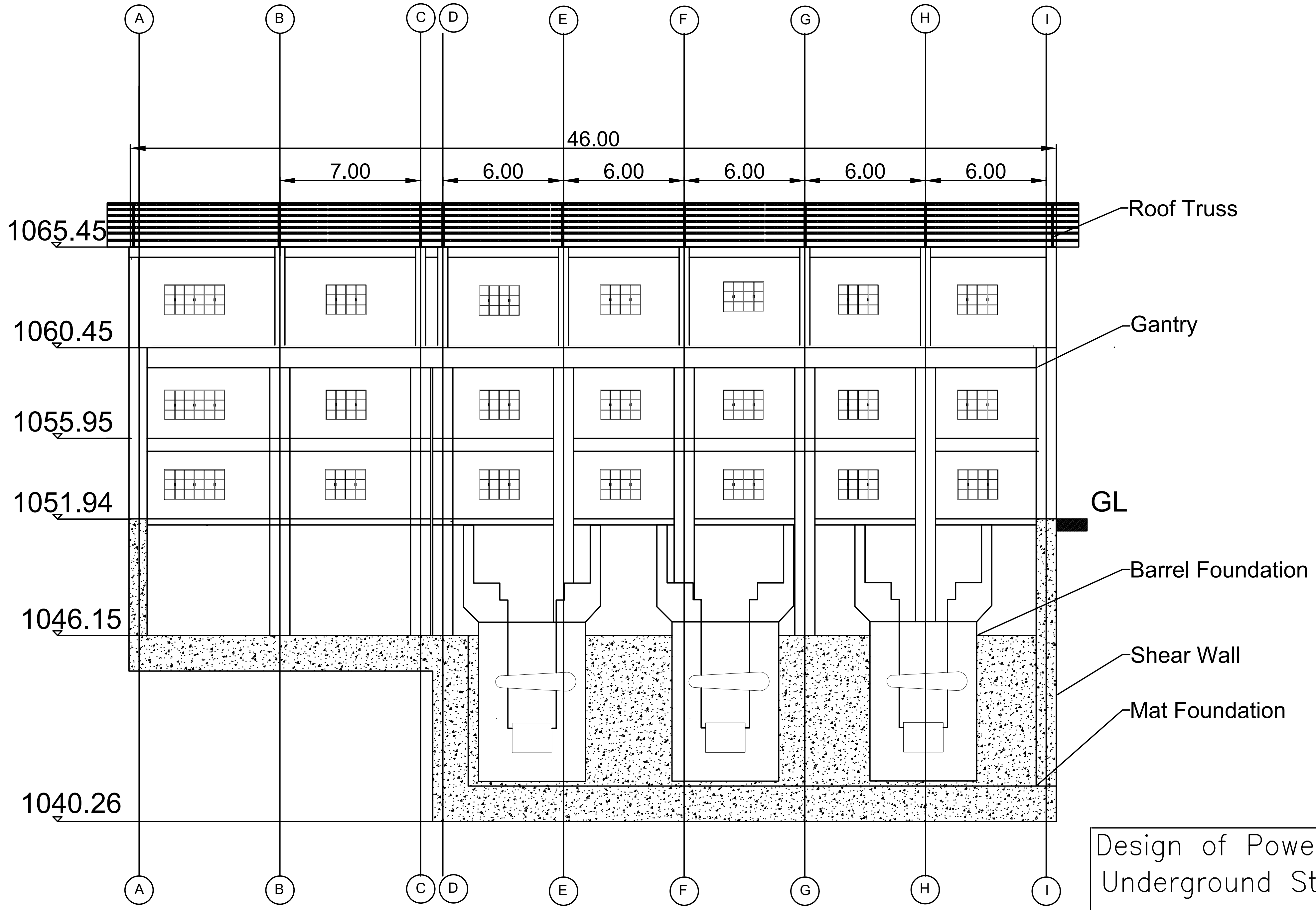


SECTION P-P
1:205

Notes:

1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Structure		
Powerhouse Drawing Section P–P	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre		Scale : As shown
		Sheet No: 2
		Revision



SECTION Q-Q
SCALE: 1:250

Notes:
1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Strucutre

Powerhouse Drawing Secion Q–Q	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre	Scale : As shown	
	Sheet No: 3	
	Revision	

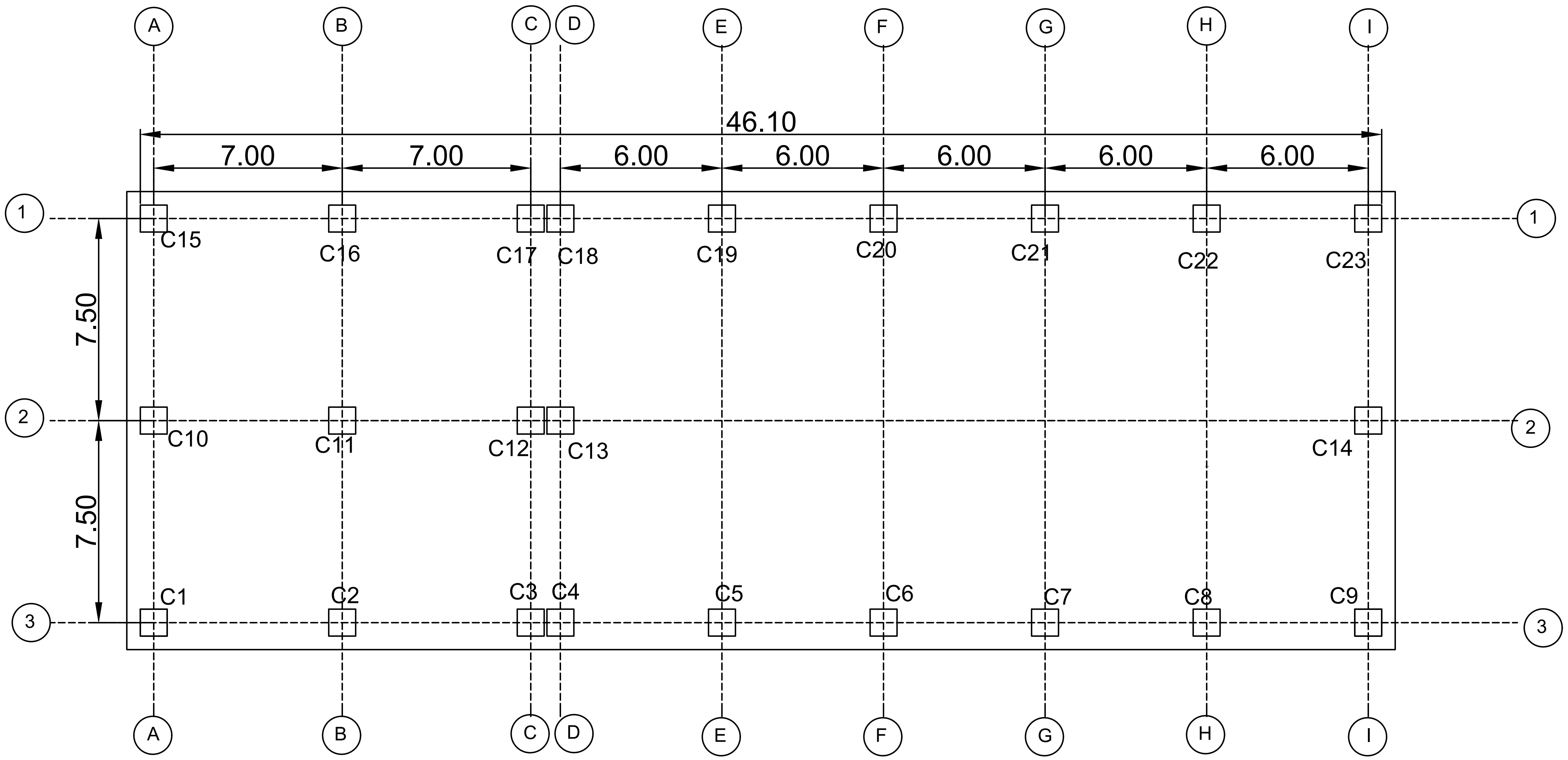


Fig: Column Layout of Powerhouse

Notes:

1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Structure

Column Layout of Powerhouse	Designed	Group 2
	Drawn	Group 2
	Checked	

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Department of Civil Engineering
Dhulikhel,Kavre

Scale : As shown

Sheet No: 4

Revision

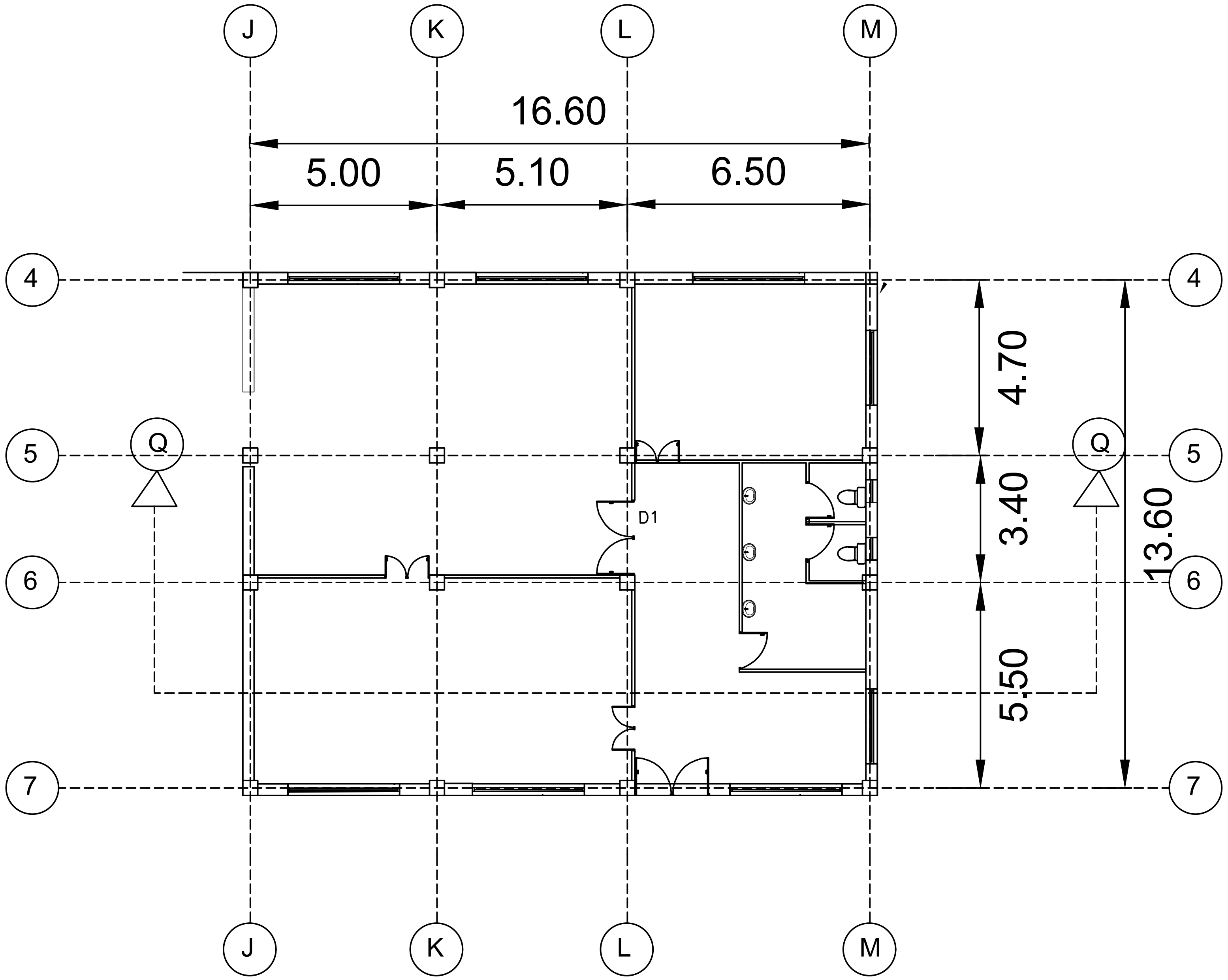


Fig: Plan of Control Room
Scale: 1:160

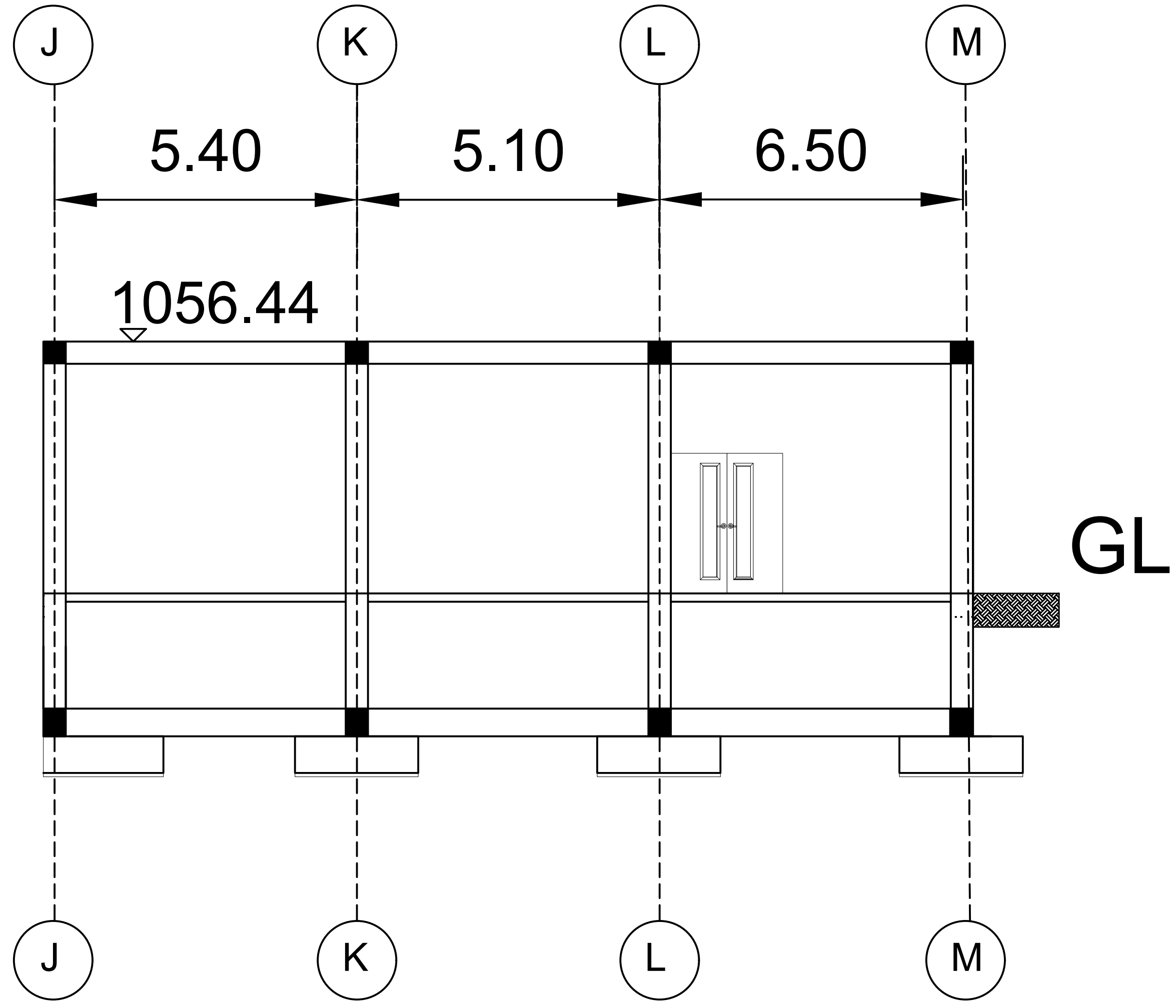
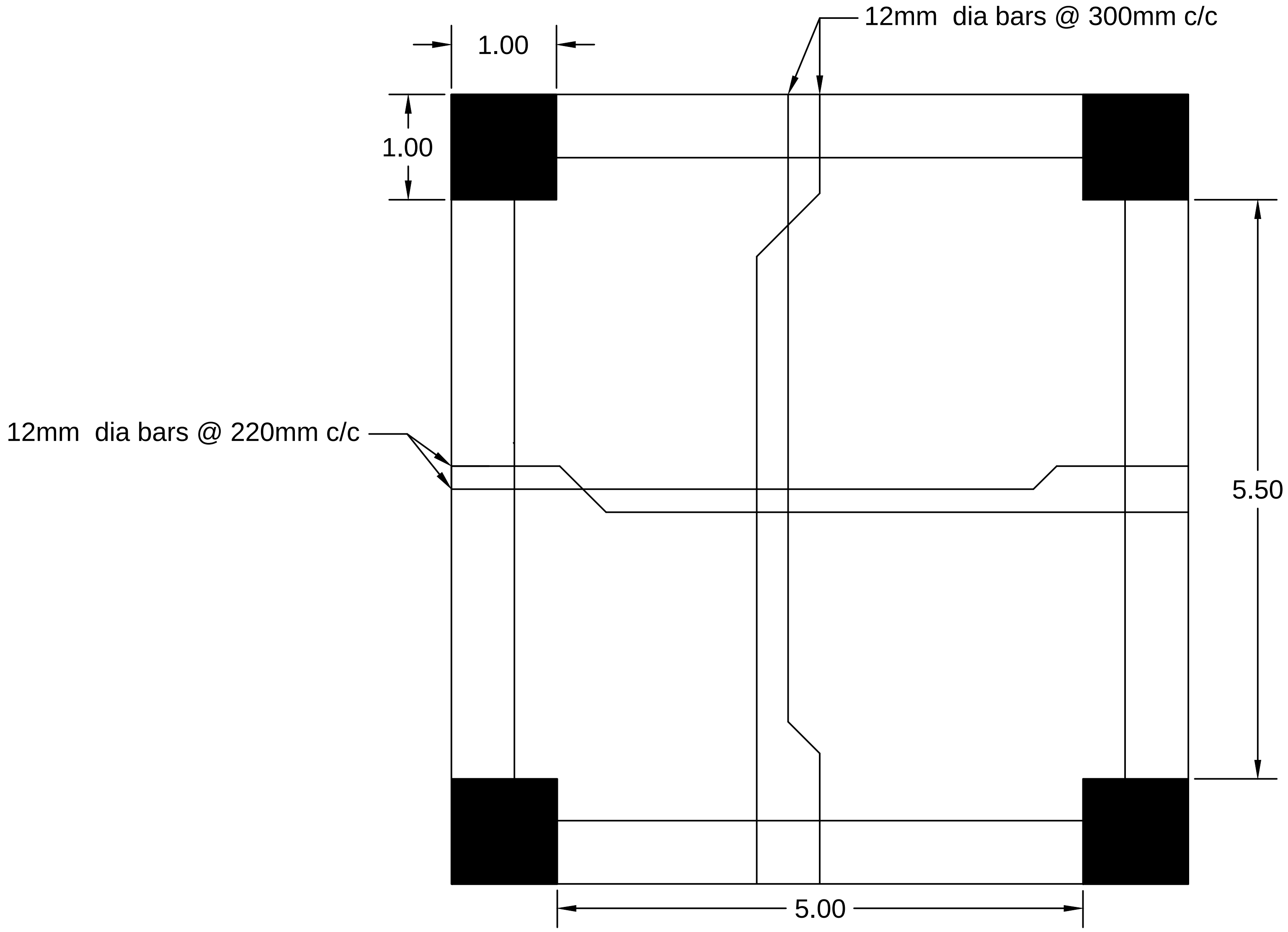


Fig: Section Q-Q of Control Room
Scale: 1:135

Notes:

1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Structure		
Plan and Section of Control Room Building	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre		Scale : As shown
		Sheet No: 5
		Revision

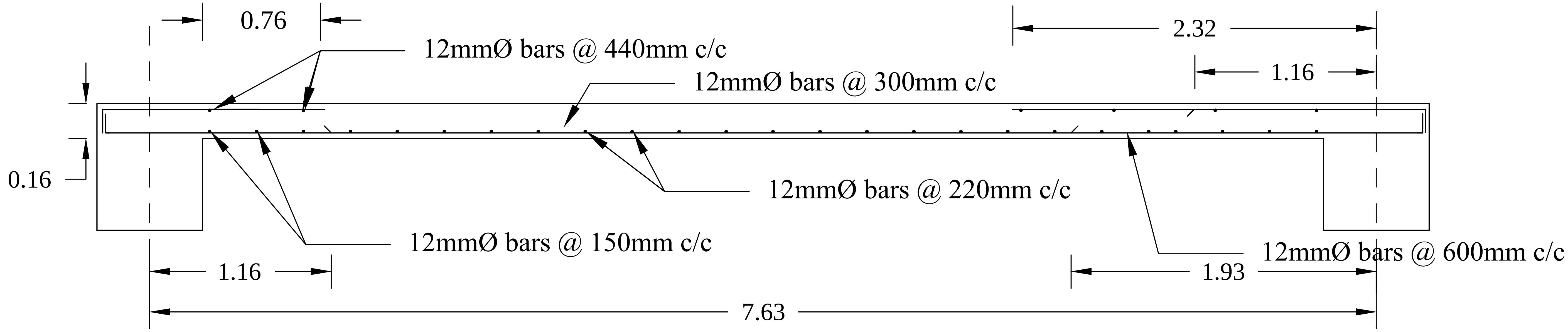


Plan of Floor Slab Reinforcement Detailing

Scale:1:60

Notes:
1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Structures		
Rebar Detailing of Erection Bay Floor Slab	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre		Scale : As shown
		Sheet No: 06
		Revision



Section along long span

Adjacent edge discontinuous

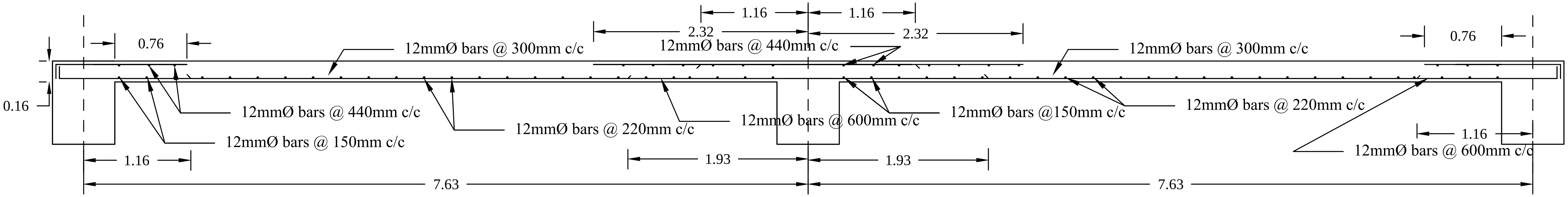
Scale: 1:35

Notes:

1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Structures		
Section Along long Span of Floor Slab	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre	Scale : As shown	
	Sheet No: 07	
	Revision	

Floor Slab



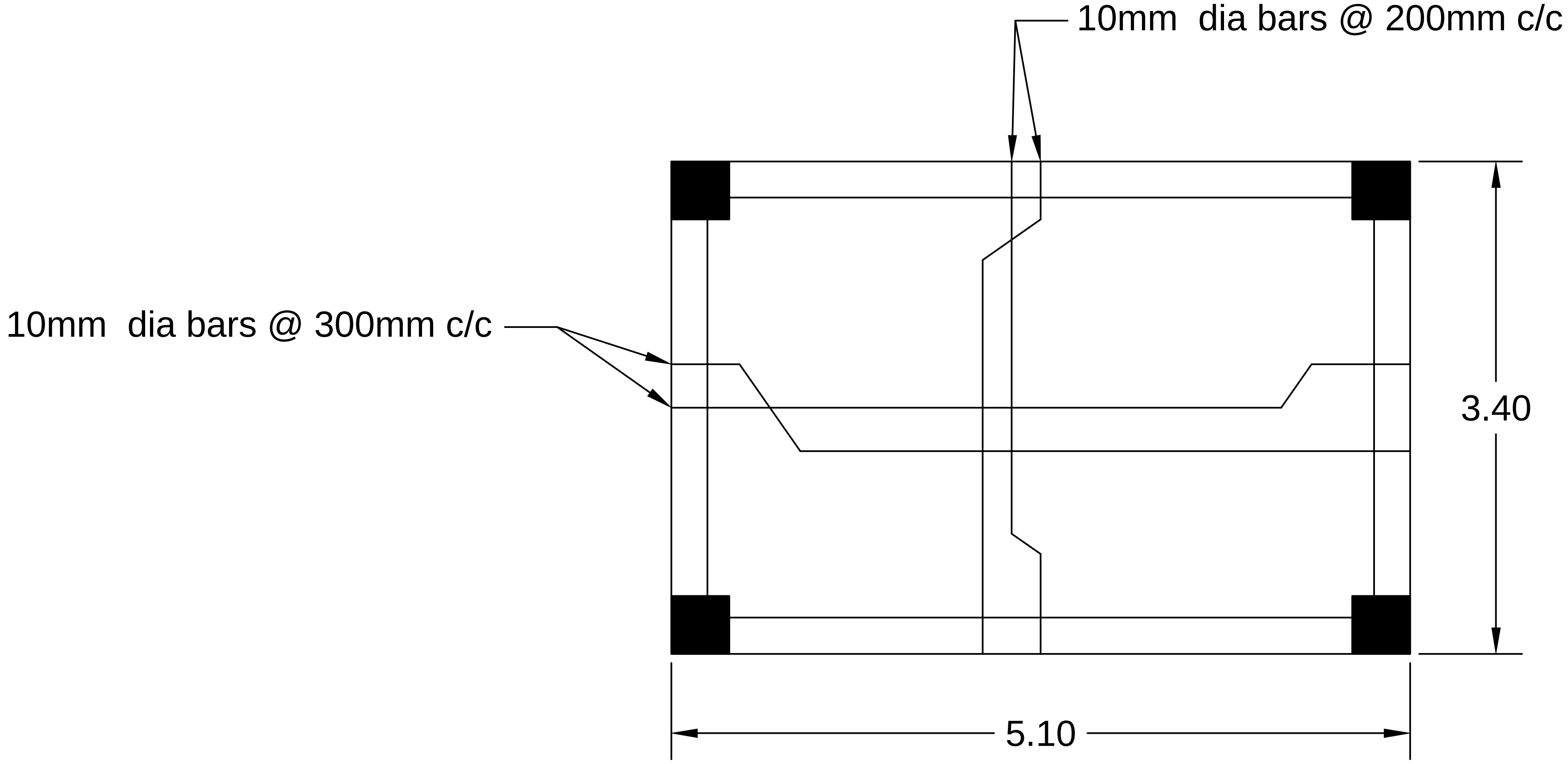
Section along long span

Scale: 1:40

Notes:

1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Structures		
Section Along Long Span of Floor Slab	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre	Scale : As shown	
	Sheet No: 08	
	Revision	

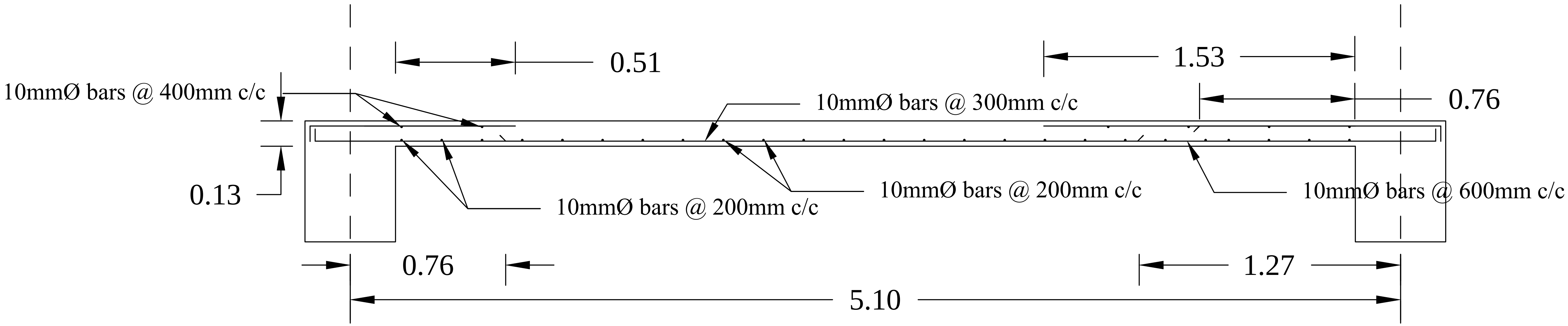


Plan of Roof Slab Reinforcement Detailing
Scale: 1:45

Notes:
1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Structures		
Rebar Detailing of Control Room Roof Slab	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre	Scale : As shown	
	Sheet No: 09	
	Revision	

Roof Slab



One short edge discontinuous

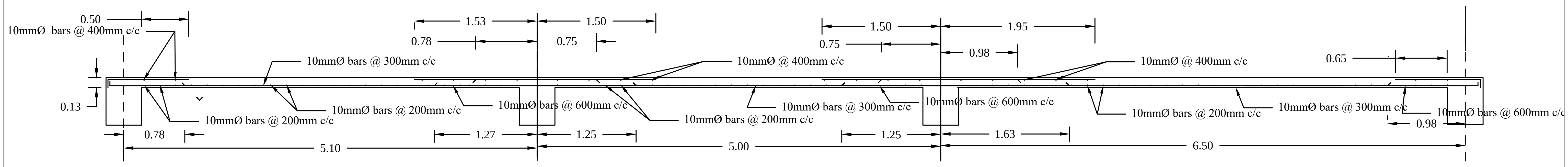
Section along long span

Scale: 1:20

Notes:
1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Structures		
Section Along Long Span of Control Room	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre		Scale : As shown
		Sheet No: 10
		Revision

Roof Slab



Section along long span

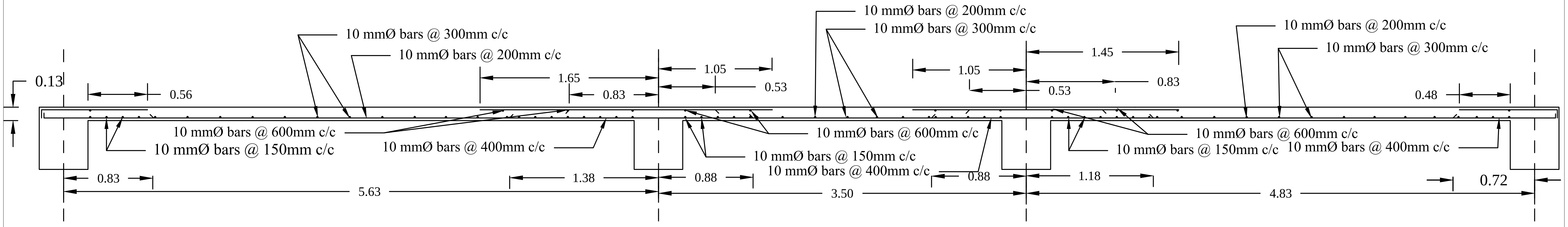
Scale: 1:50

Notes:

1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Structures		
Control Room Section Along Long Span Grid	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre	Scale : As shown	
	Sheet No: 11	
	Revision	

Roof Slab

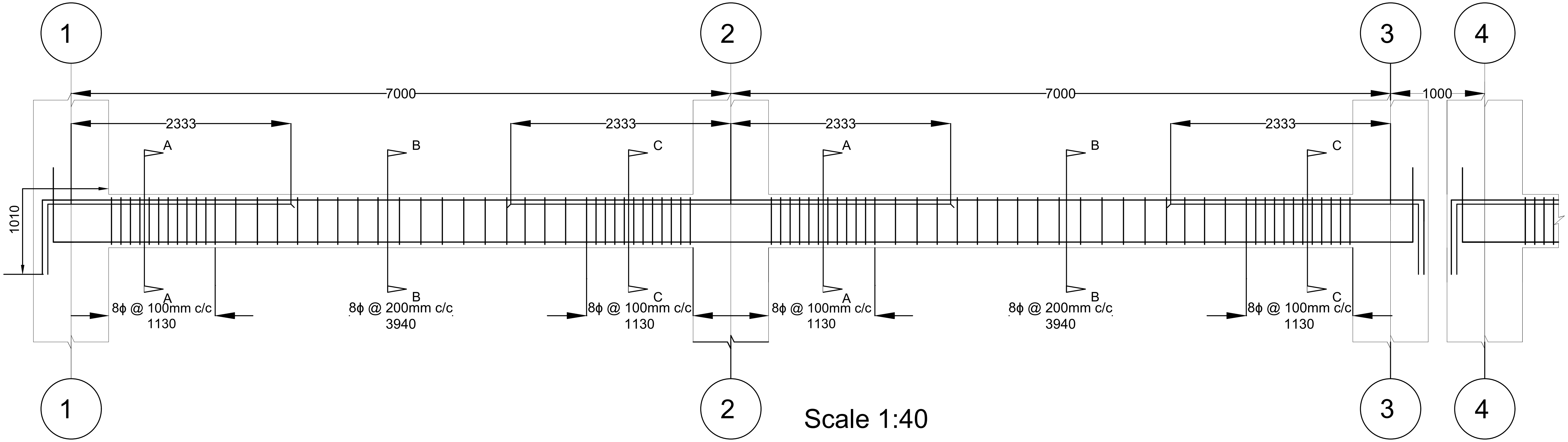


Section along short span

Scale: 1:35

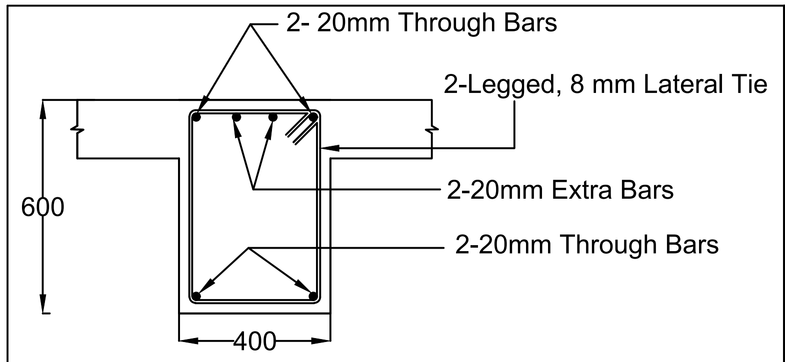
Notes:
1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Structures		
Section Along short span of Control Room Roof Slab	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre		Scale : As shown
		Sheet No: 12
		Revision

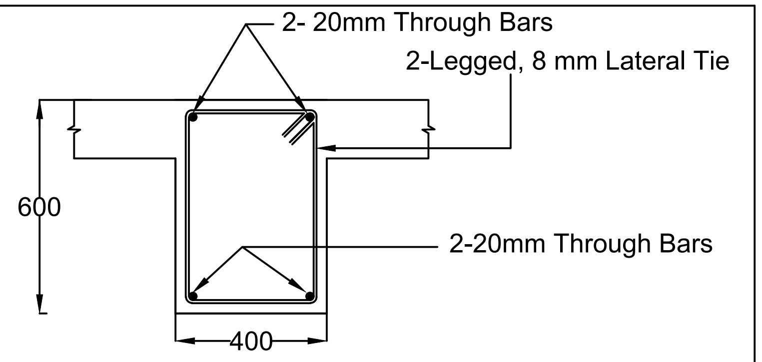


Scale 1:40

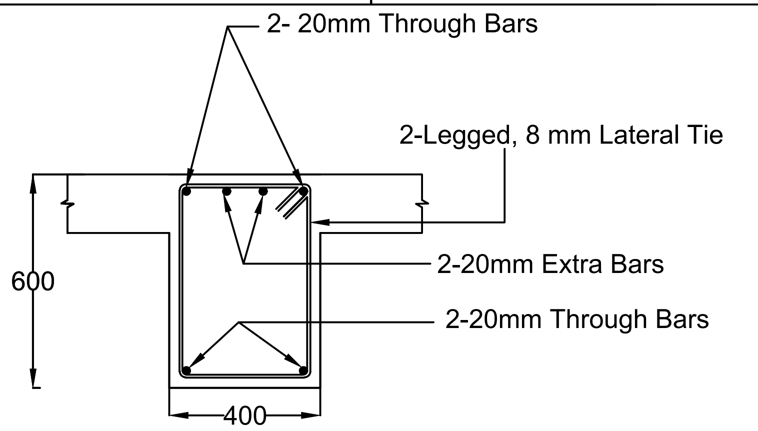
Longitudinal Section Along X-X Grid of Powerhouse (A3-B3 and B3-C3)



Section at A-A



Section at B-B



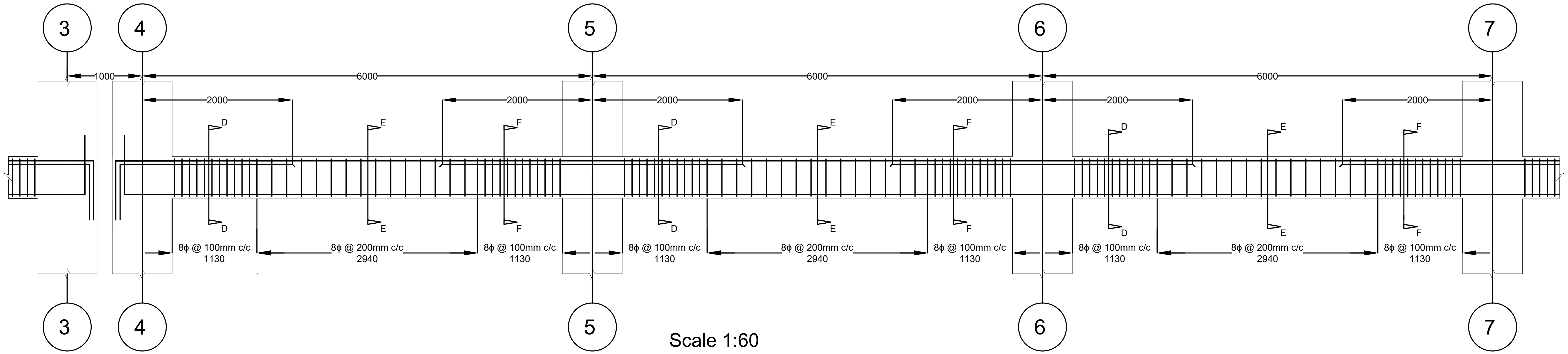
Section at C-C

Scale 1:80

Notes:

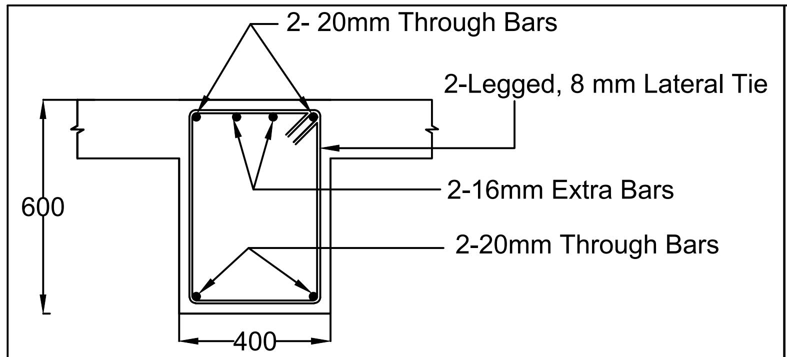
1. All the dimensions are measured in mm and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Strucutre		
Longitudinal Section Along X–X of power-house Beam	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre		Scale : As shown
		Sheet No: 13
		Revision

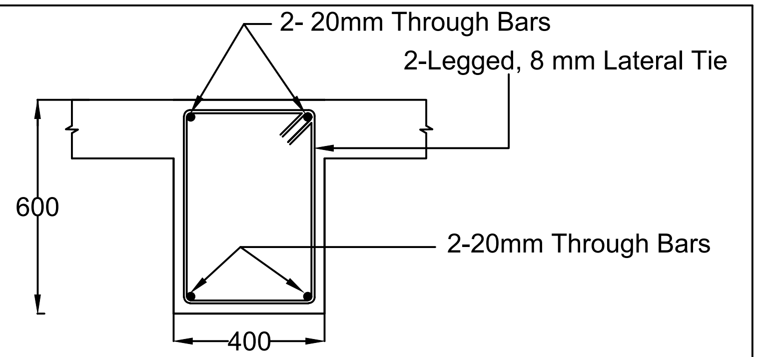


Scale 1:60

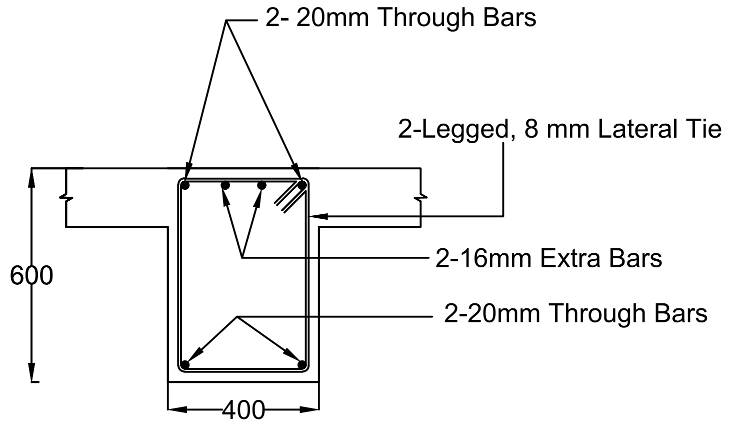
Longitudinal Section Along X-X Grid of Powerhouse (D3-E3, E3-F3 and F3-G3)



Section at D-D



Section at E-E



Section at F-F

Scale 1:80

Notes:

1. All the dimensions are measured in mm and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Strucutre

Longitudinal Section Along X–X of power-house Beam	Designed	Group 2
	Drawn	Group 2
	Checked	

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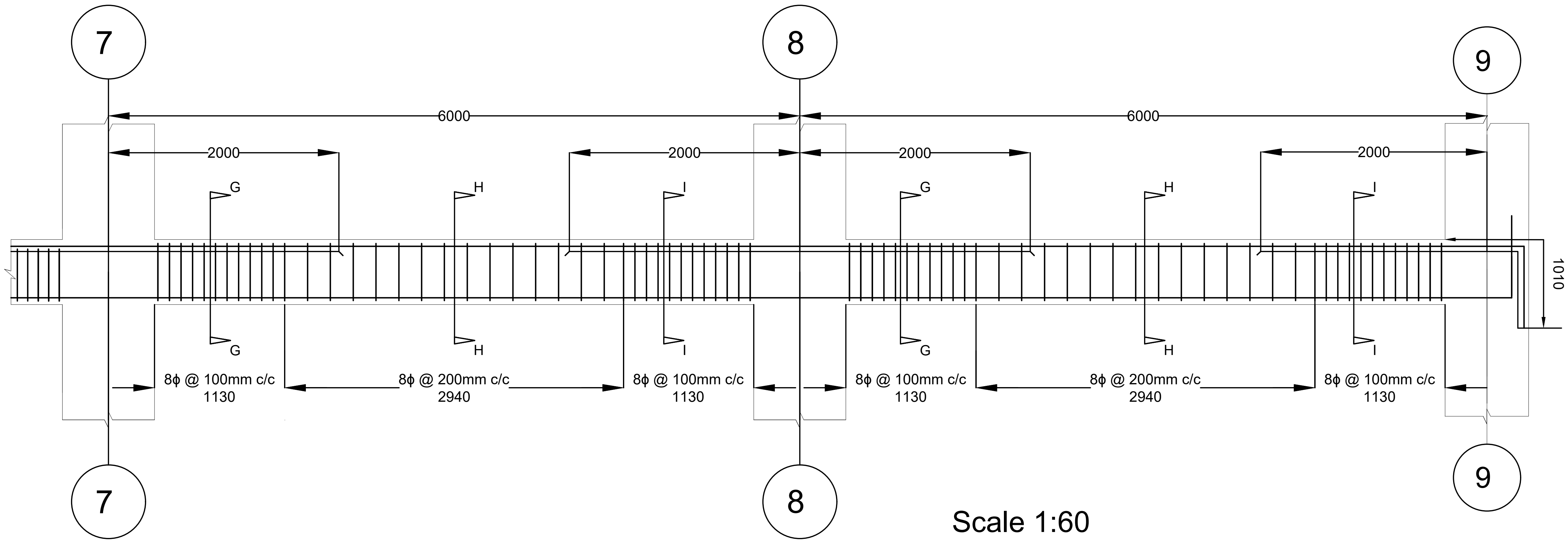
Dhulikhel,Kavre

Scale : As shown

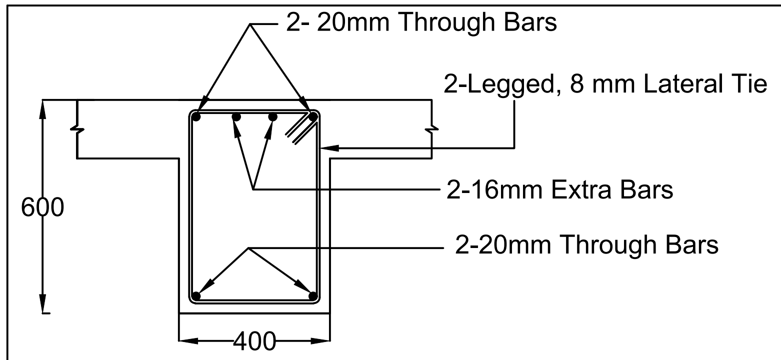
Sheet No: 14

Revision

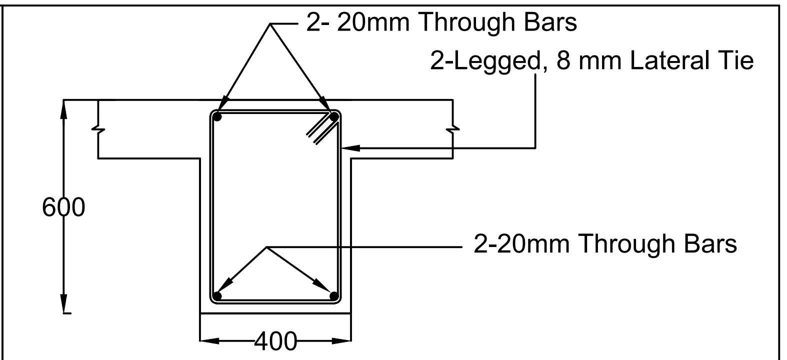


Scale 1:60

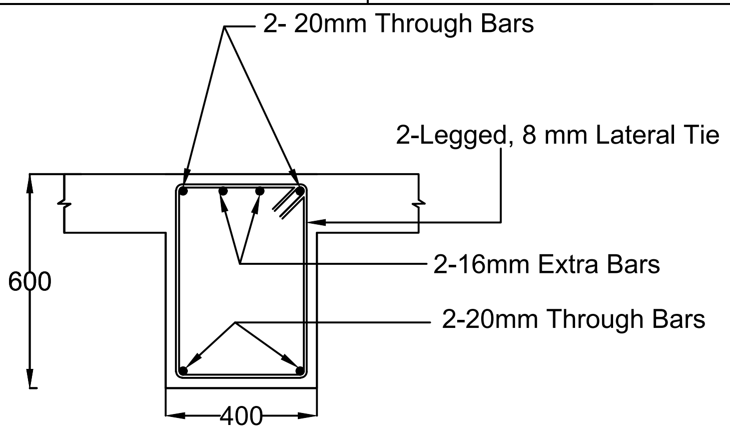
Longitudinal Section Along X-X Grid of Powerhouse (G3-H3 and H3-I3)



Section at G-G



Section at H-H



Section at I-I

Scale 1:80

Notes:

1. All the dimensions are measured in mm and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Strucutre

Longitudinal Section Along X–X of power–house Beam	Designed	Group 2
	Drawn	Group 2
	Checked	

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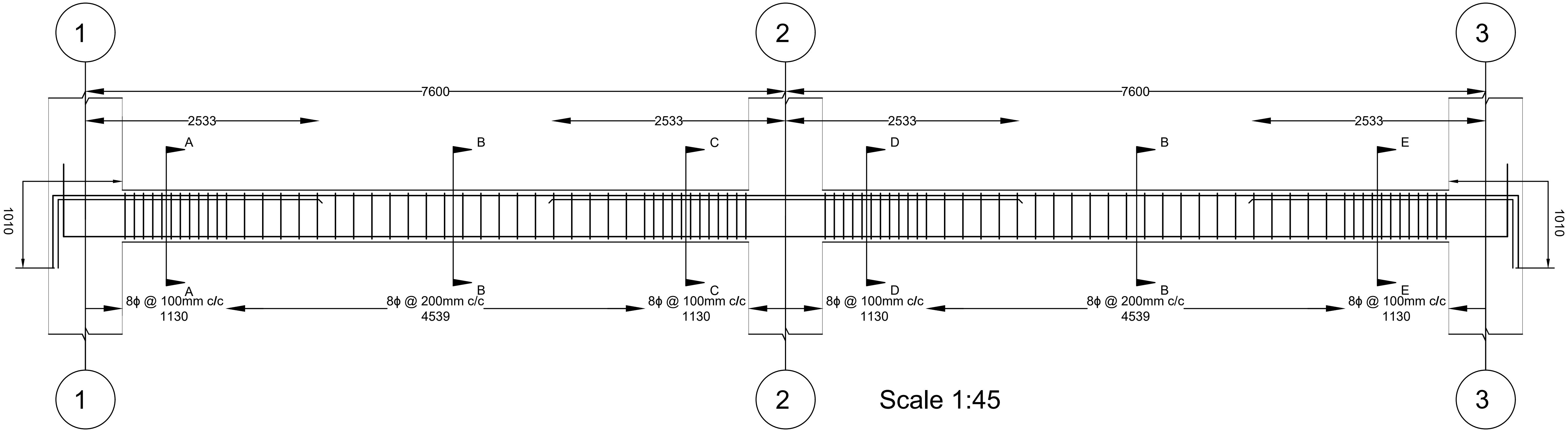
Scale : As shown

Sheet No. : 15

Revision

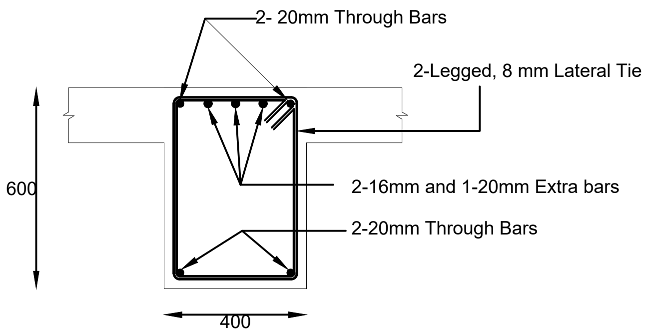


Scale 1:45

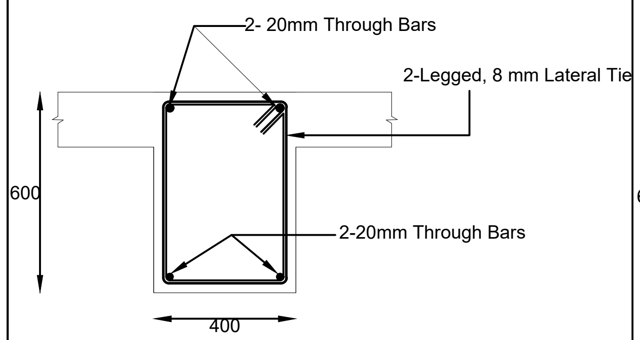
Longitudinal Section Along Y-Y Grid of Powerhouse (A2-A3 and A1-A2)

Notes:

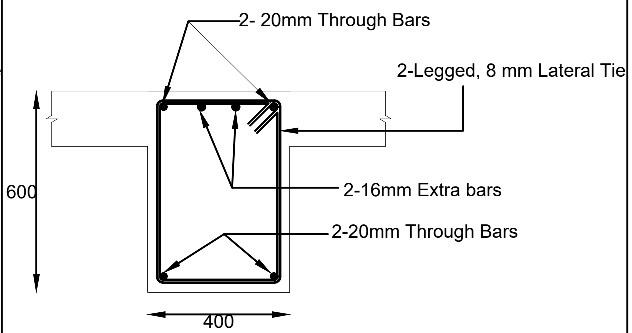
1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.



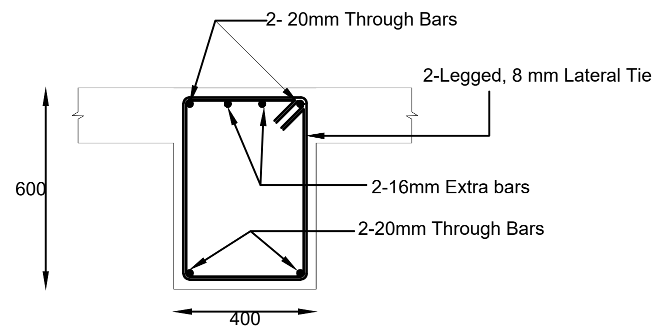
Section at A-A



Section at B-B

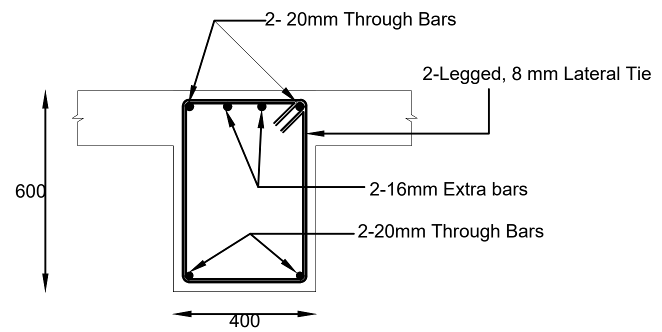


Section at C-C



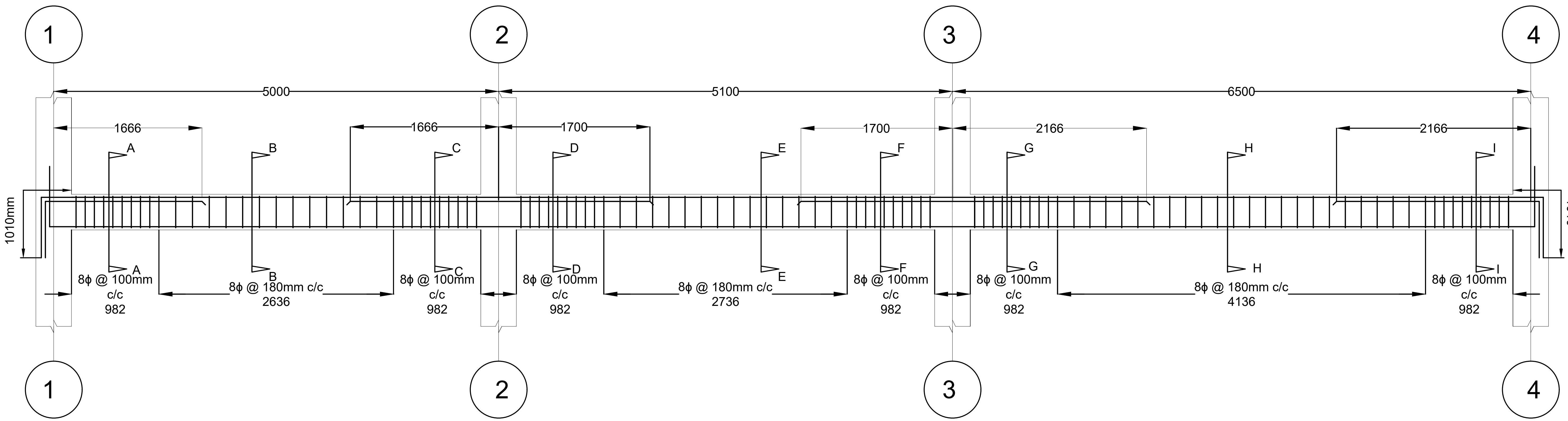
Section at D-D

Scale 1:85



Section at E-E

Design of Powerhouse and Analysis of Underground Structures		
Longitudinal Section along Y–Y of powerhouse beam	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre		Scale : As shown
		Sheet No: 16
		Revision

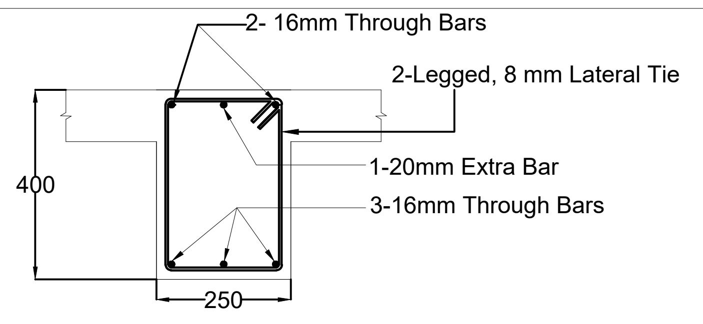


Longitudinal Section Along X-X Grid of Control Unit (C31-C32, C32-C33 and C33-C34)

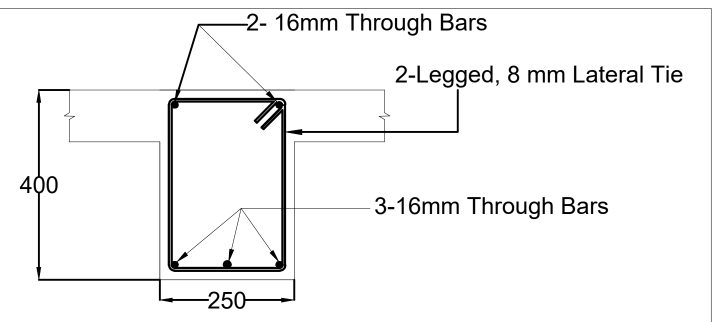
Scale 1:45

Notes:

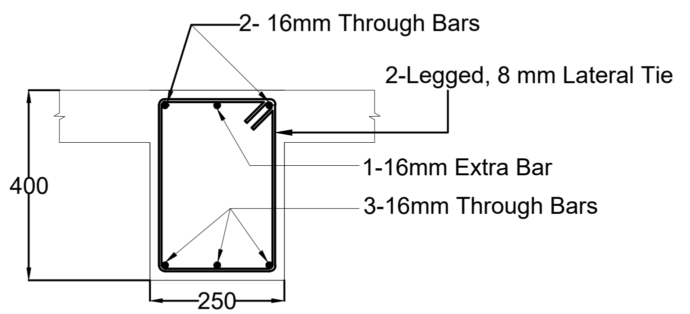
1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.



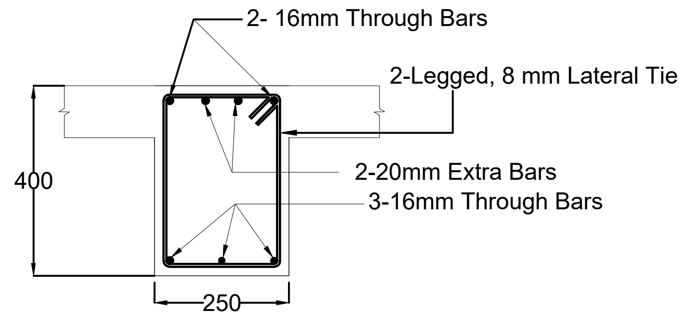
Section at A-A
Section at F-F
Section at G-G



Section at B-B
Section at E-E
Section at H-H



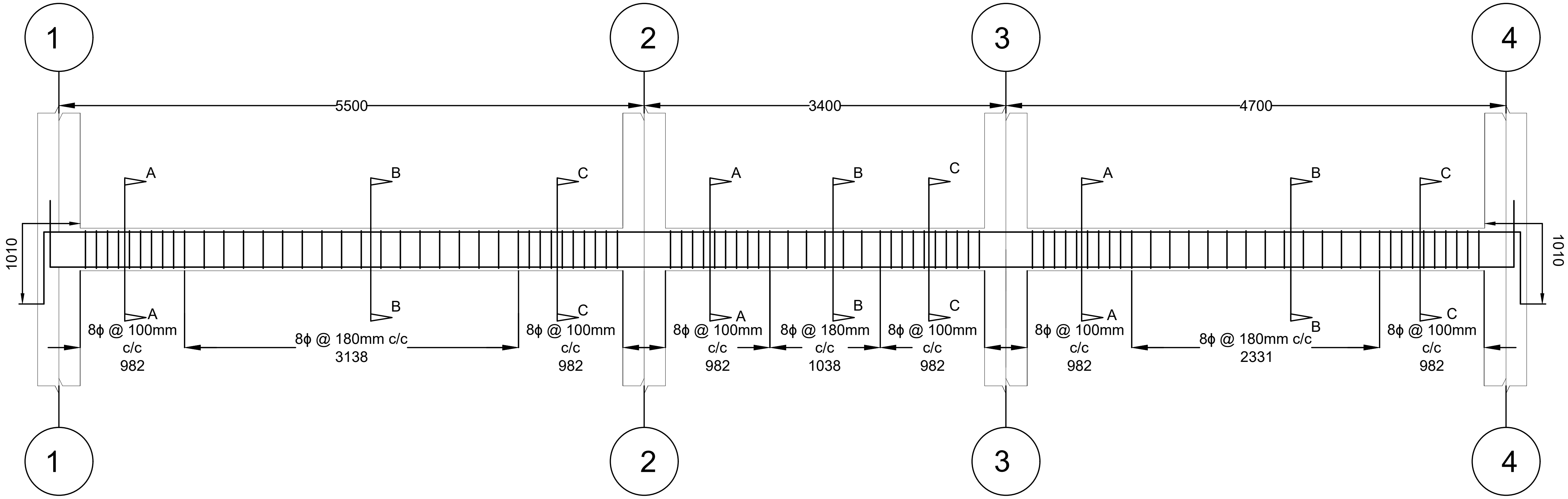
Section at C-C
Section at D-D



Section at I-I

Scale 1:90

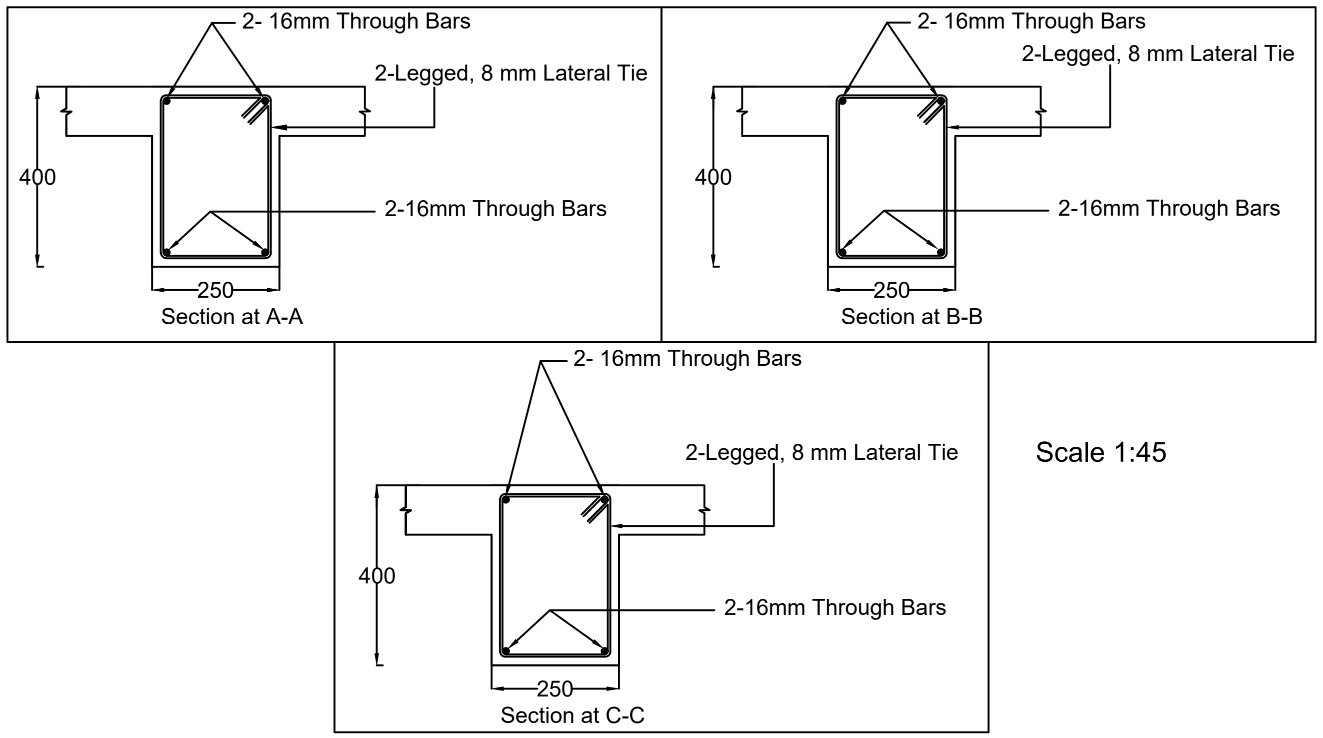
Design of Powerhouse and Analysis of Underground Structures		
Longitudinal section along X–X of control unit beam	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre		Scale : As shown
		Sheet No: 17
		Revision



Longitudinal Section Along Y-Y Grid of Control Unit (C35-C31, C31-C33, C33-C26)

Scale 1:45

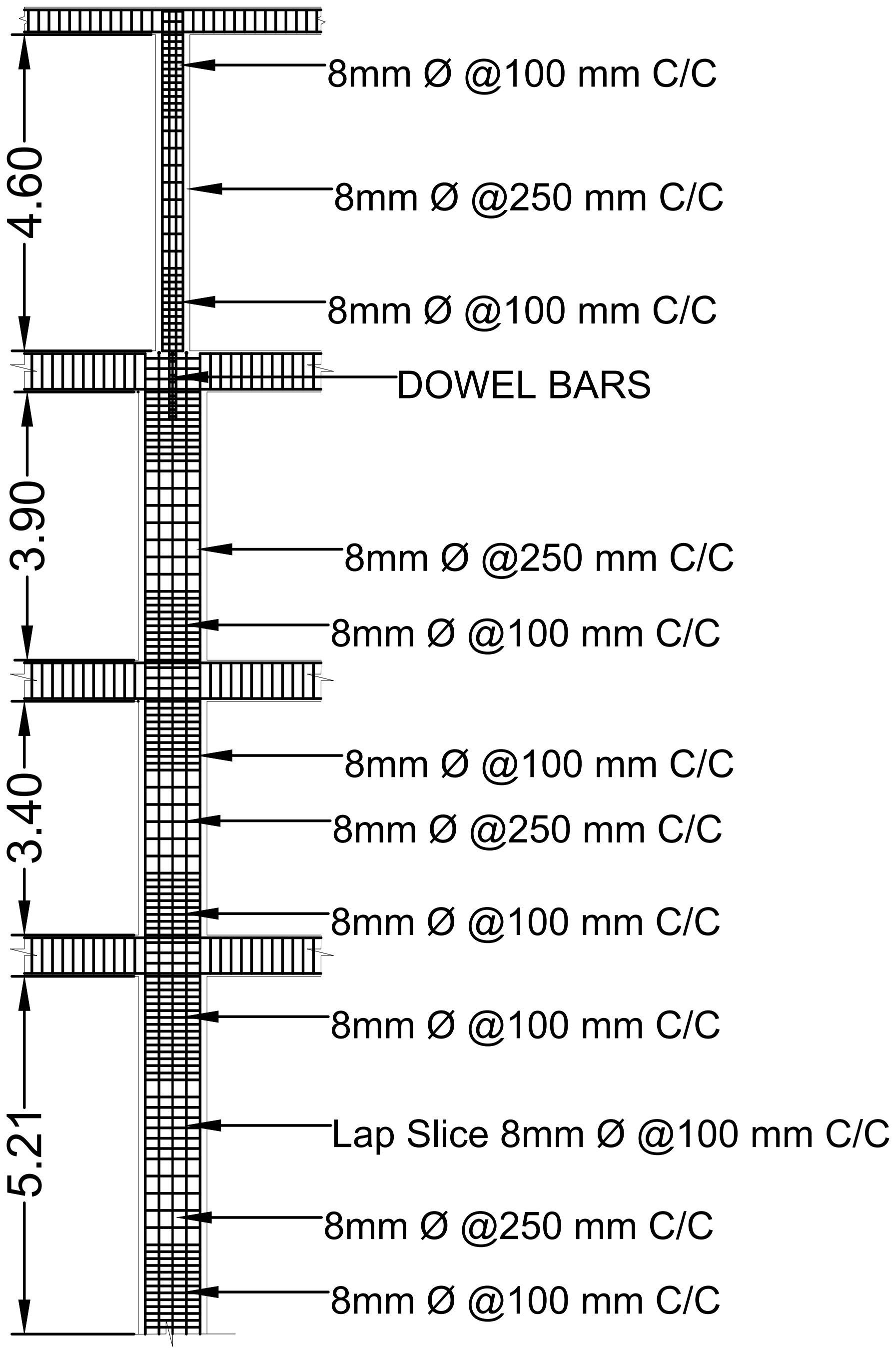
Notes:
1. All the dimensions are measured in mm and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.



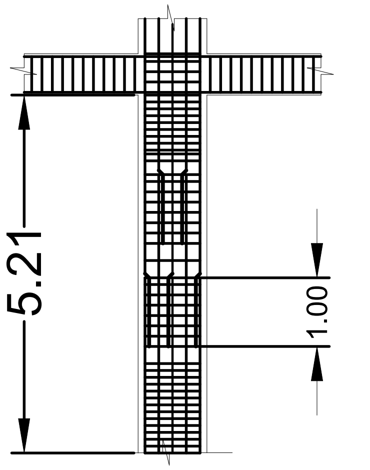
Scale 1:45

Design of Powerhouse and Analysis of Underground Structures

Longitudinal section along Y–Y of control unit beam	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre		Scale : As shown
		Sheet No: 18
		Revision

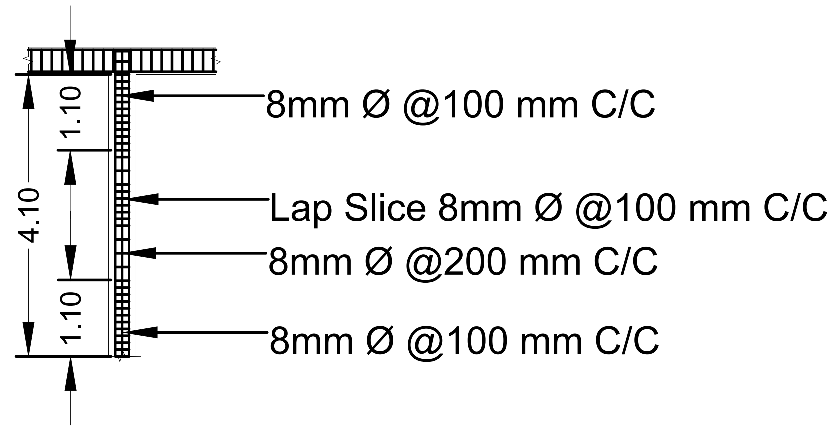


L-Section of Powerhouse Column



LAPPING IN COLUMN

SCALE: 1:110

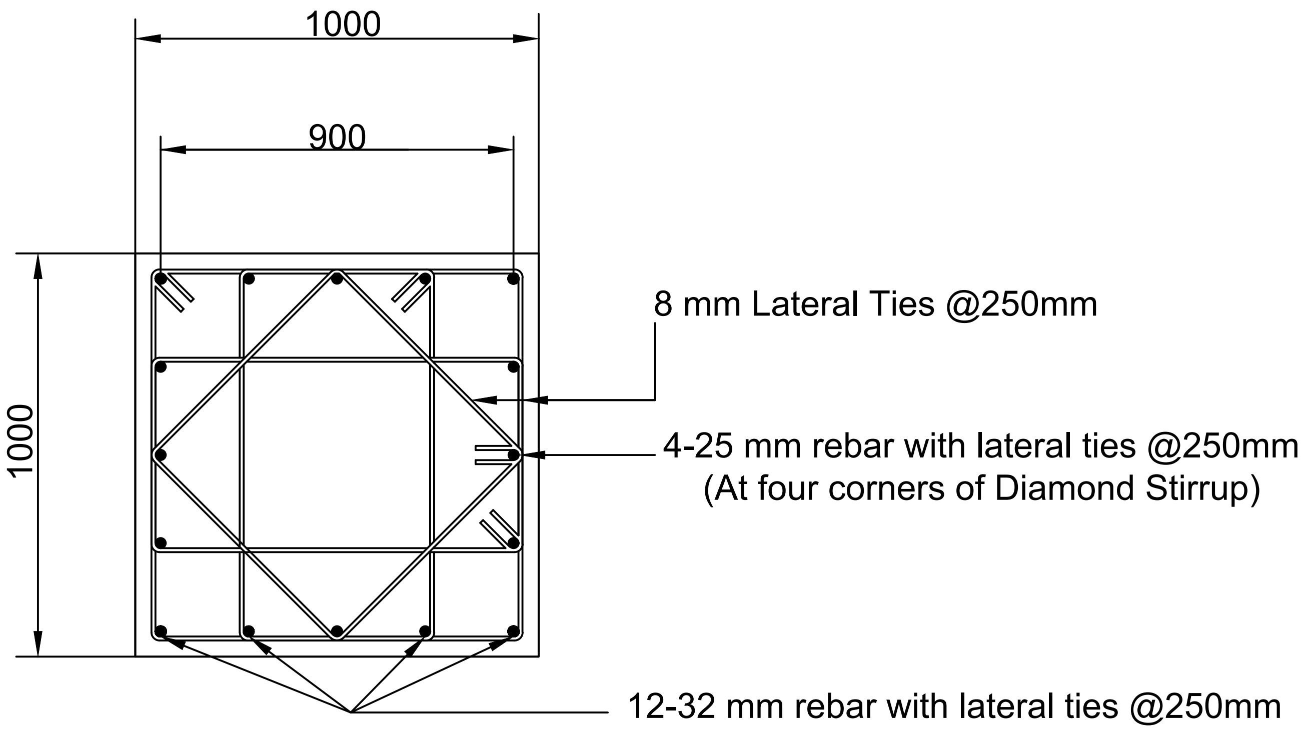


L-Section of Control Room Column

Notes:

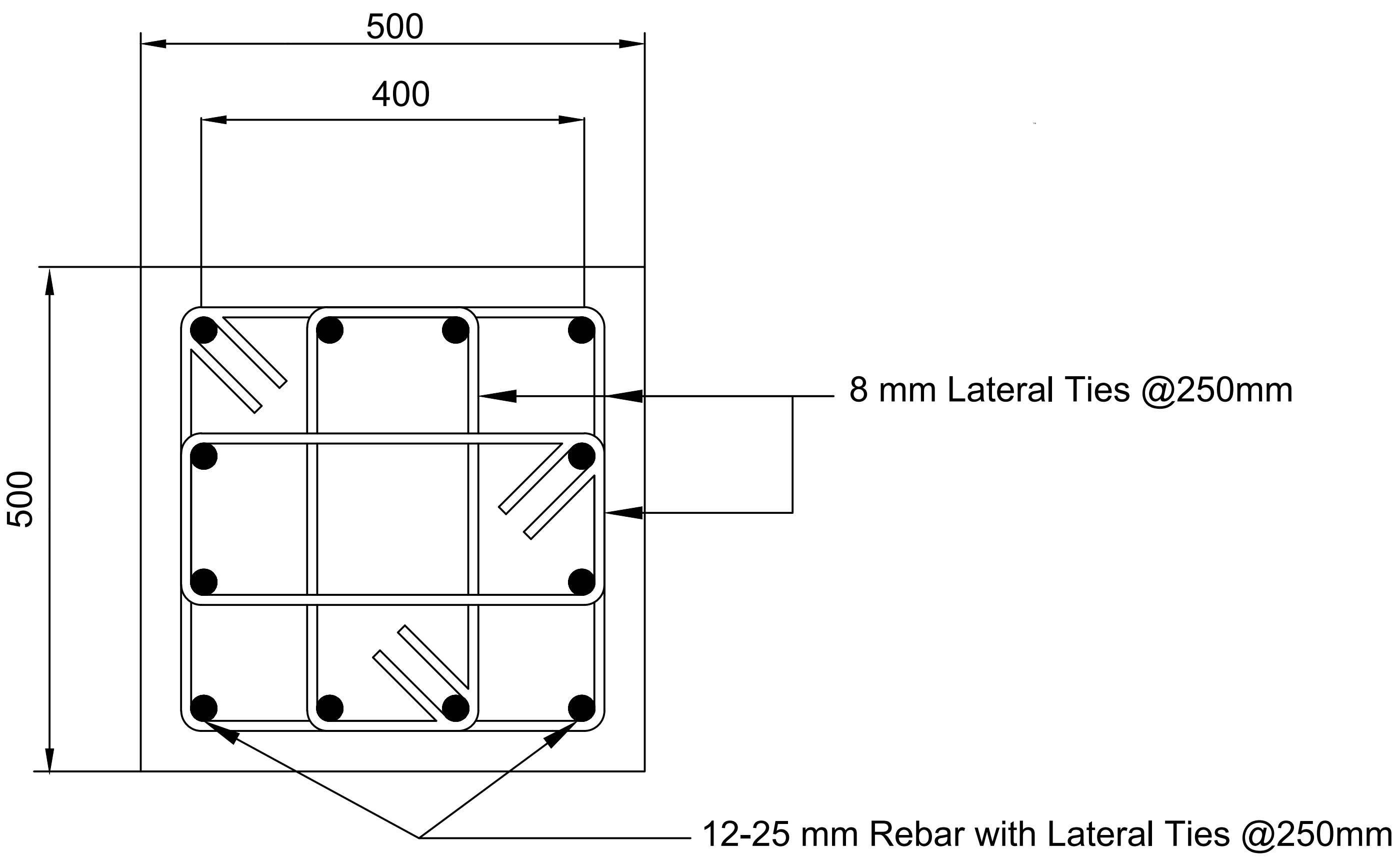
1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Strucutre		
Detailing of PowerHouse and Control Room Column	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre		Scale : As shown
		Sheet No:19
		Revision



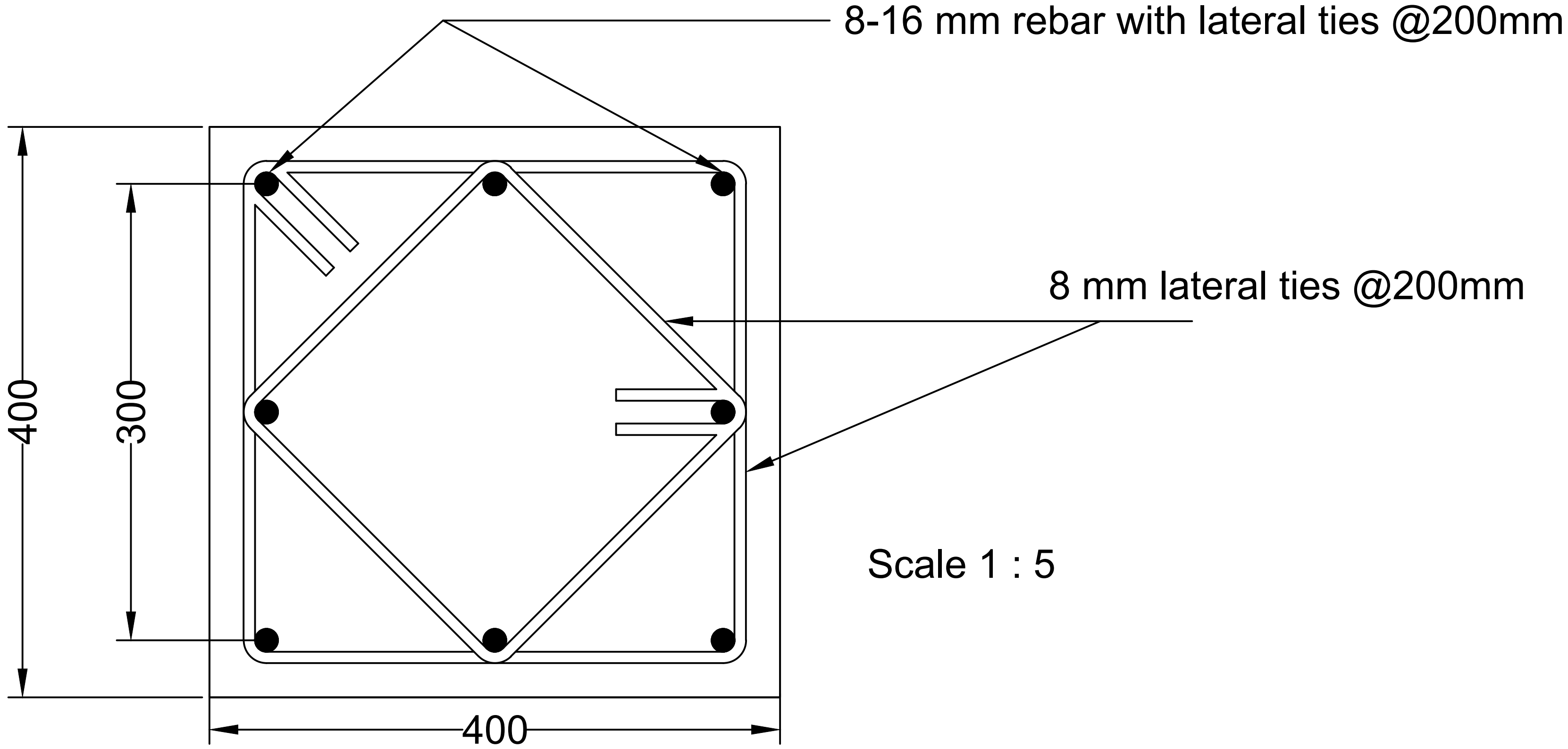
Section of Column C3
Below Gantry Girder

Scale 1 : 15



Section of Column C3 above
Gantry Girder

Scale 1 : 6



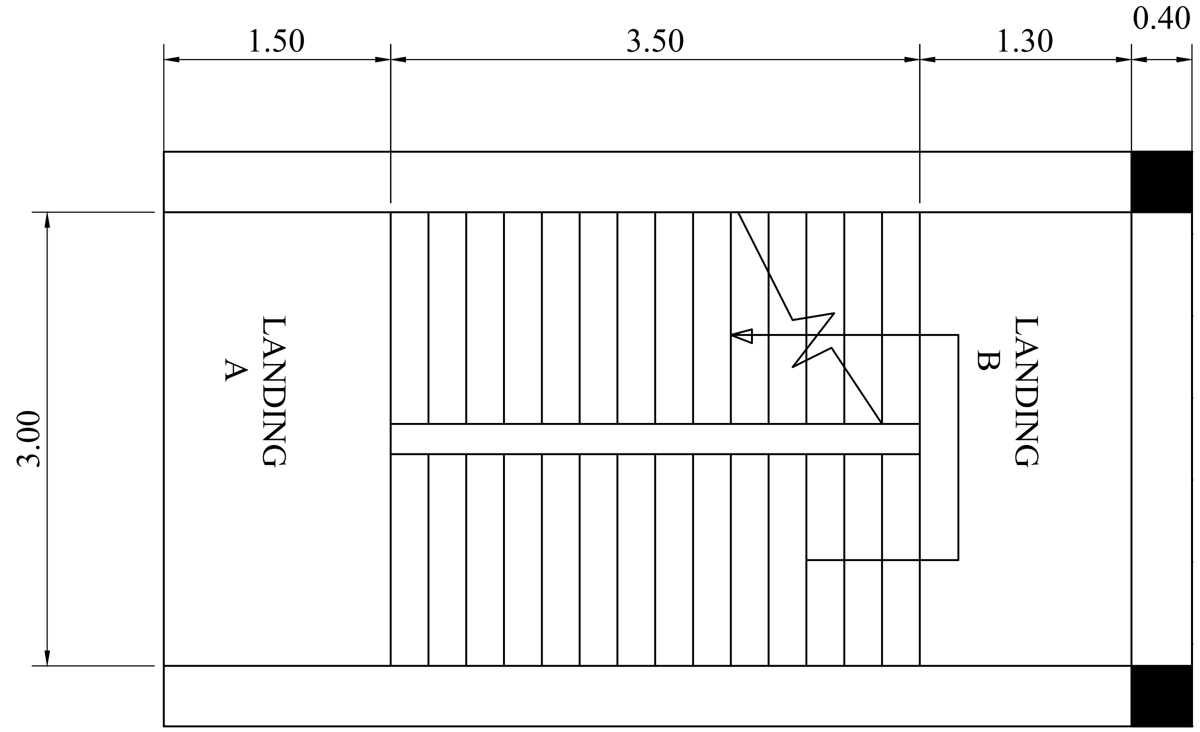
Scale 1 : 5

Section of Column C31
(Control Room)

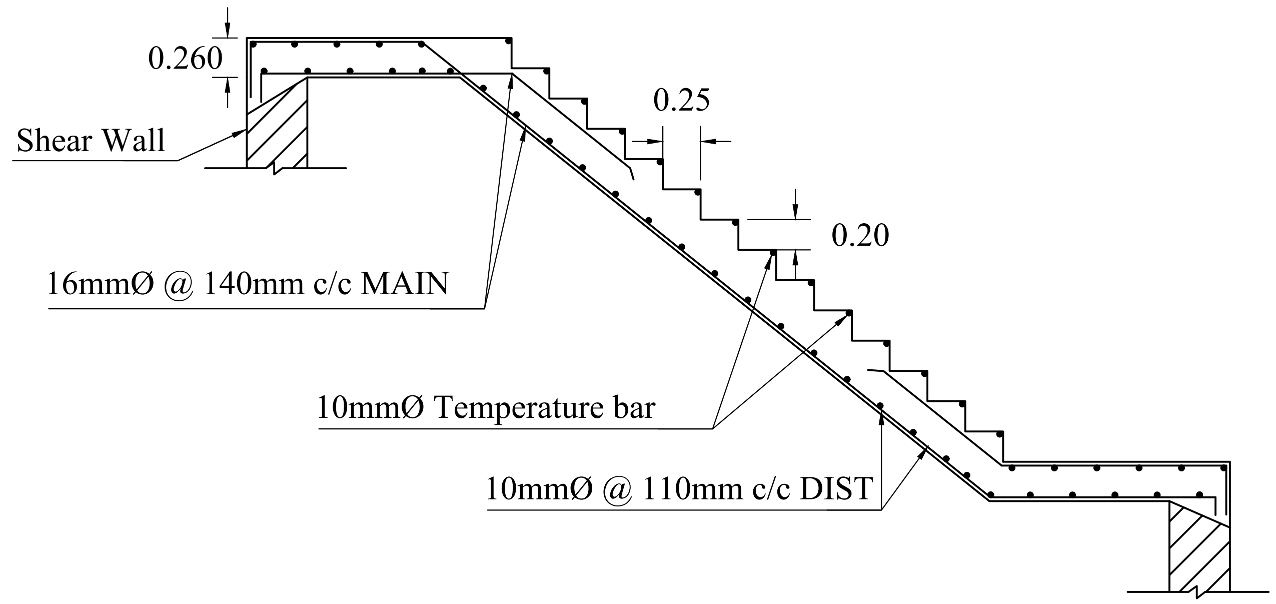
Notes:

1. All the dimensions are measured in mm and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Structures		
Column Sections	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre	Scale : As shown	
	Sheet No: 20	
	Revision	



Plan of Staircase
Scale : 1:50

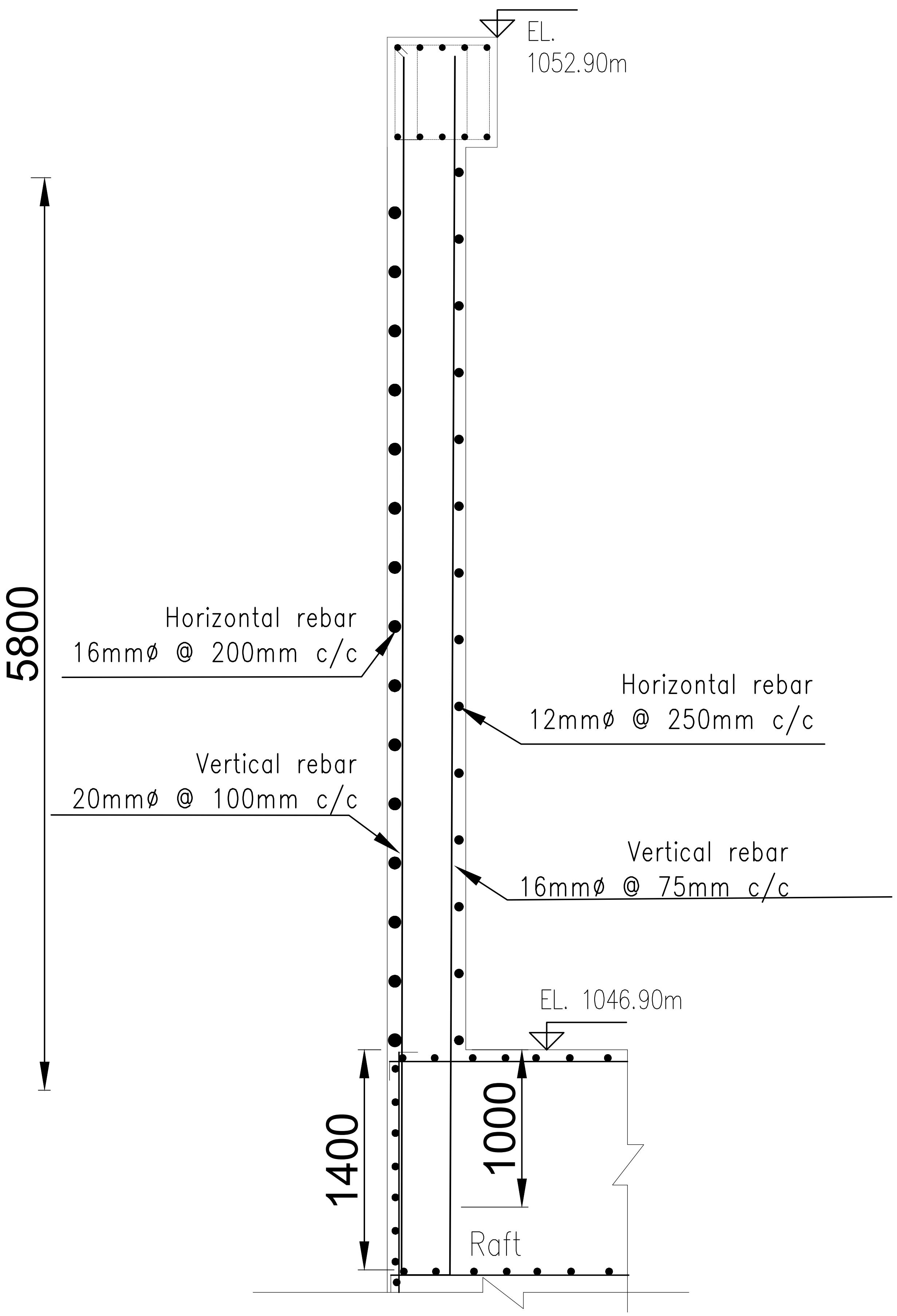


Reinforcement in stair flight
Scale : 1:50

Notes:

1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

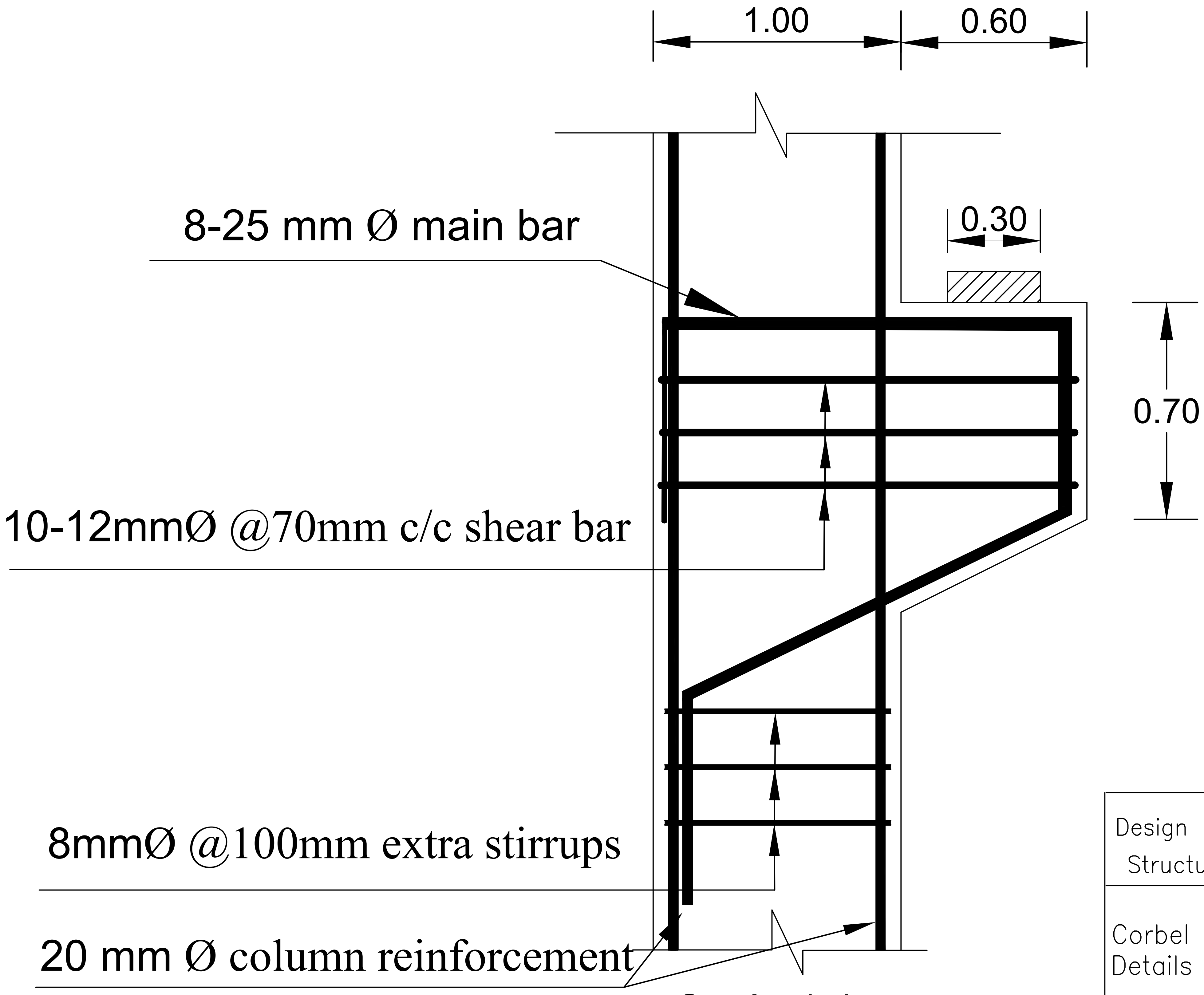
Design of Powerhouse and Analysis of Underground Strucutre		
Staircase Plan and Reinforcement	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre		Scale : As shown
		Sheet No: 21
		Revision



Scale 1:35
Section of Shear Wall

Notes:
1. All the dimensions are measured in mm and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

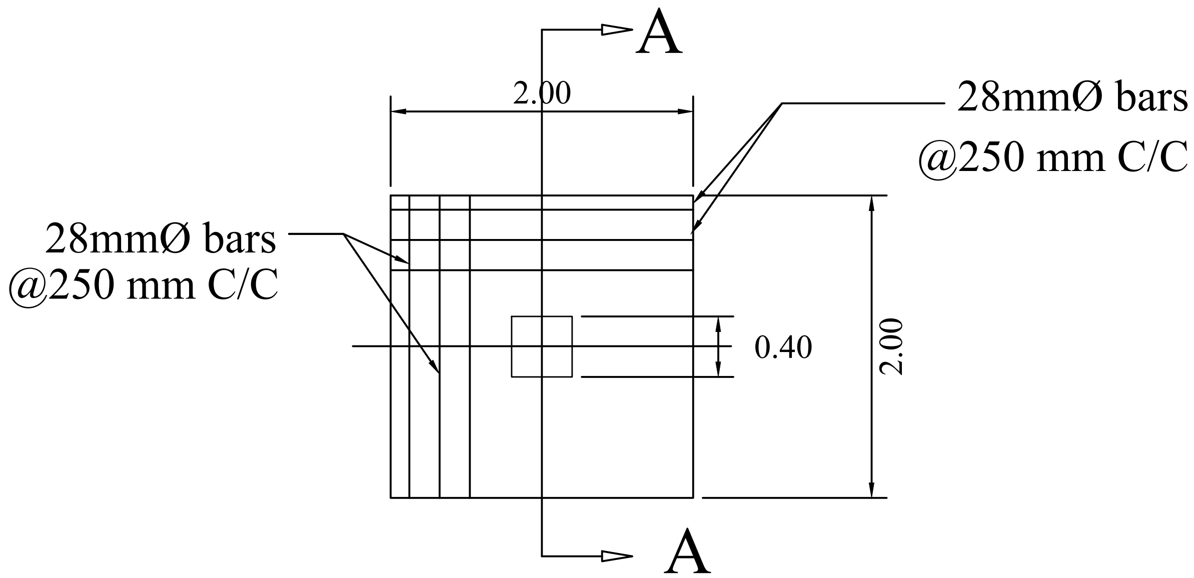
Design of Powerhouse and Analysis of Underground Structures			
Shear Reinforcement Details		Designed	Group 2
		Drawn	Group 2
		Checked	
	KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre	Scale : As shown	
		Sheet No : 22	
		Revision	



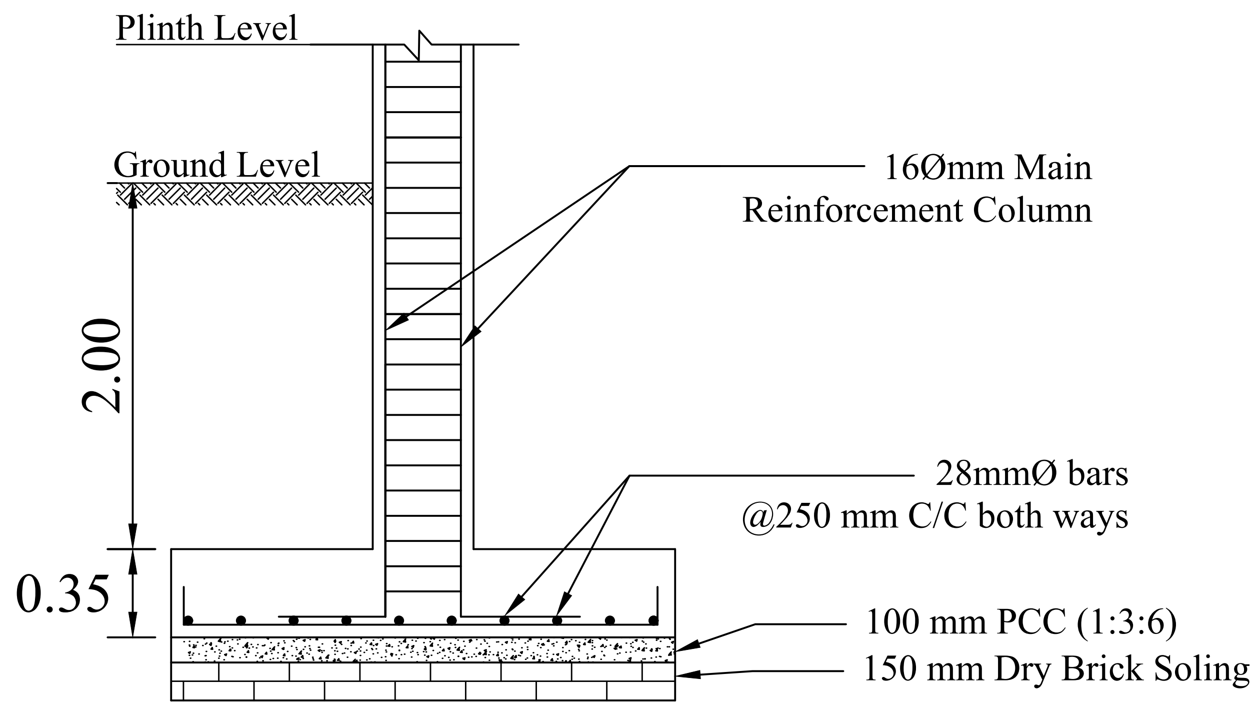
Scale 1:15
Section of Corbel

Notes:
1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

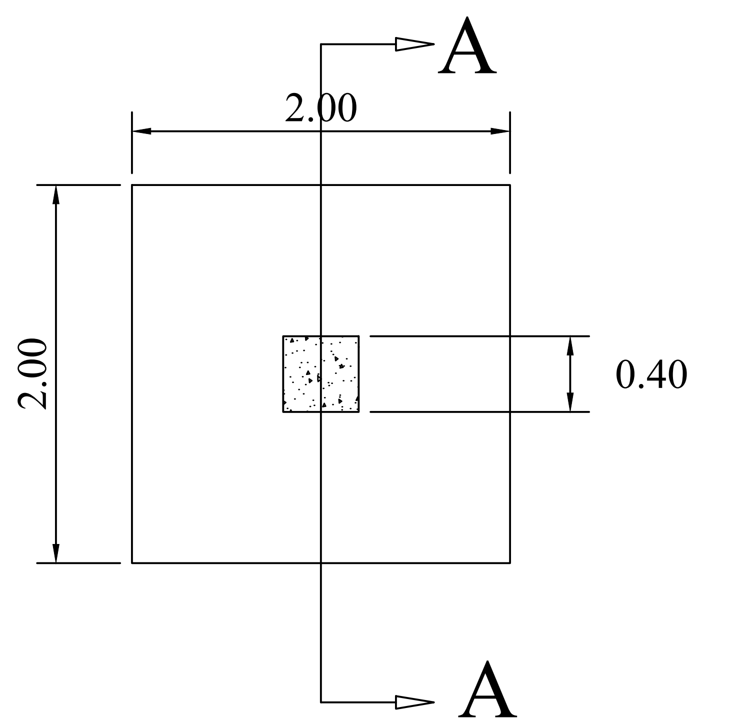
Design of Powerhouse and Analysis of Underground Structures		
Corbel Reinforcement Details	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre		Scale : As shown
		Sheet No : 23
		Revision



Reinforcement Details
of Isolated Foundation
Scale : 1:50



Section A-A
Scale : 1:30



Plan of Isolated Foundation
Scale : 1:40

Notes:

1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Strucutre

Isolated Footing Plan and Reinforcement	Designed	Group 2
	Drawn	Group 2
	Checked	

KATHMANDU UNIVERSITY
School of Engineering
Department of Civil Engineering
Dhulikhel,Kavre

Scale : As shown
Sheet No: 24
Revision

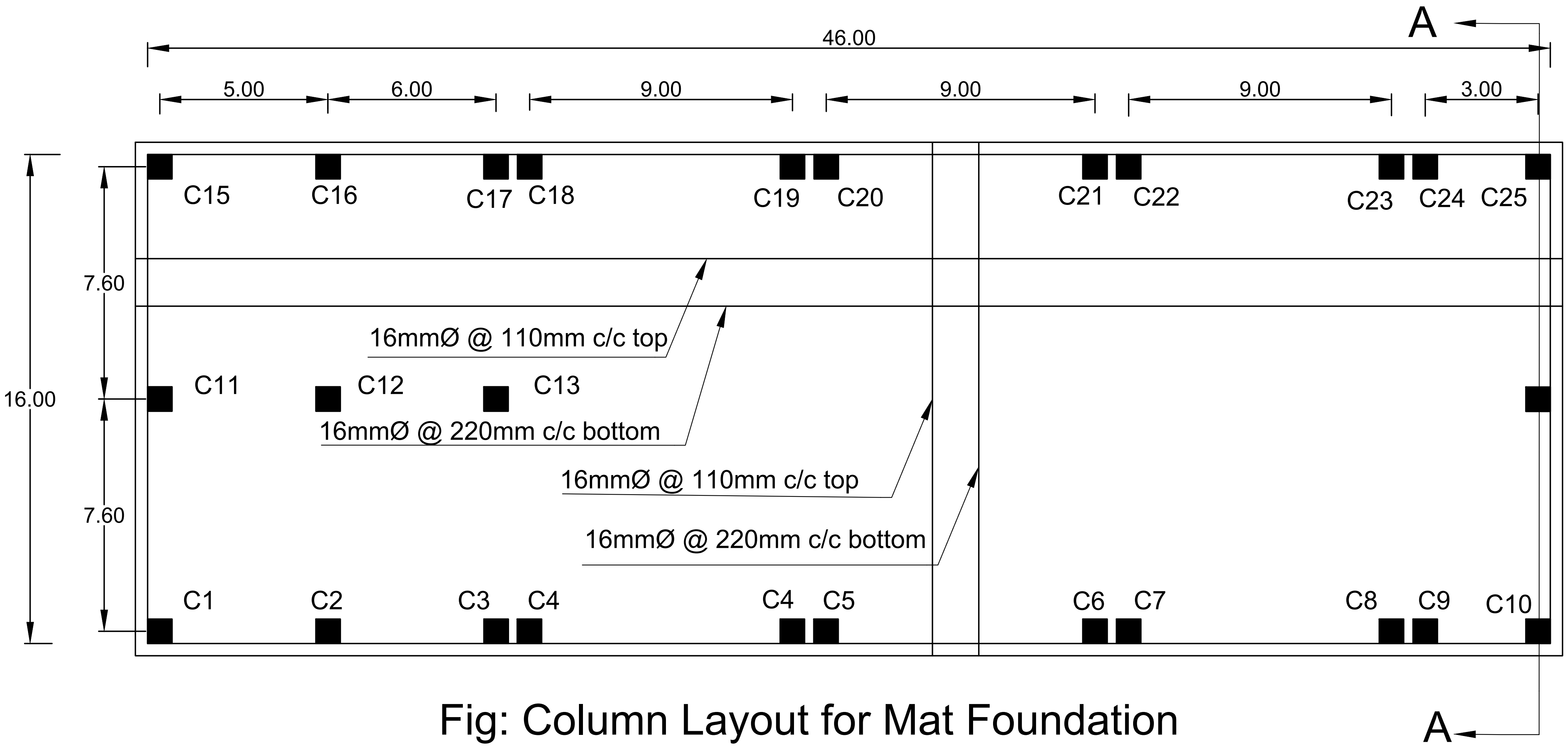
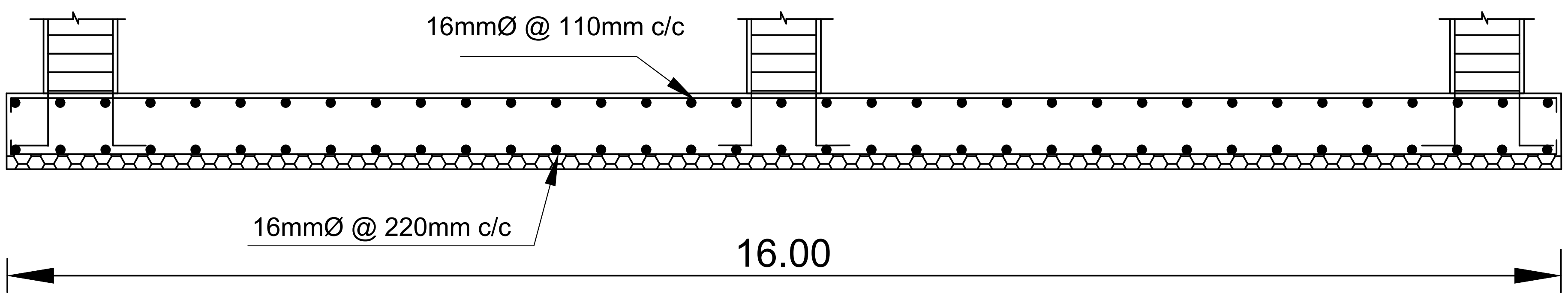


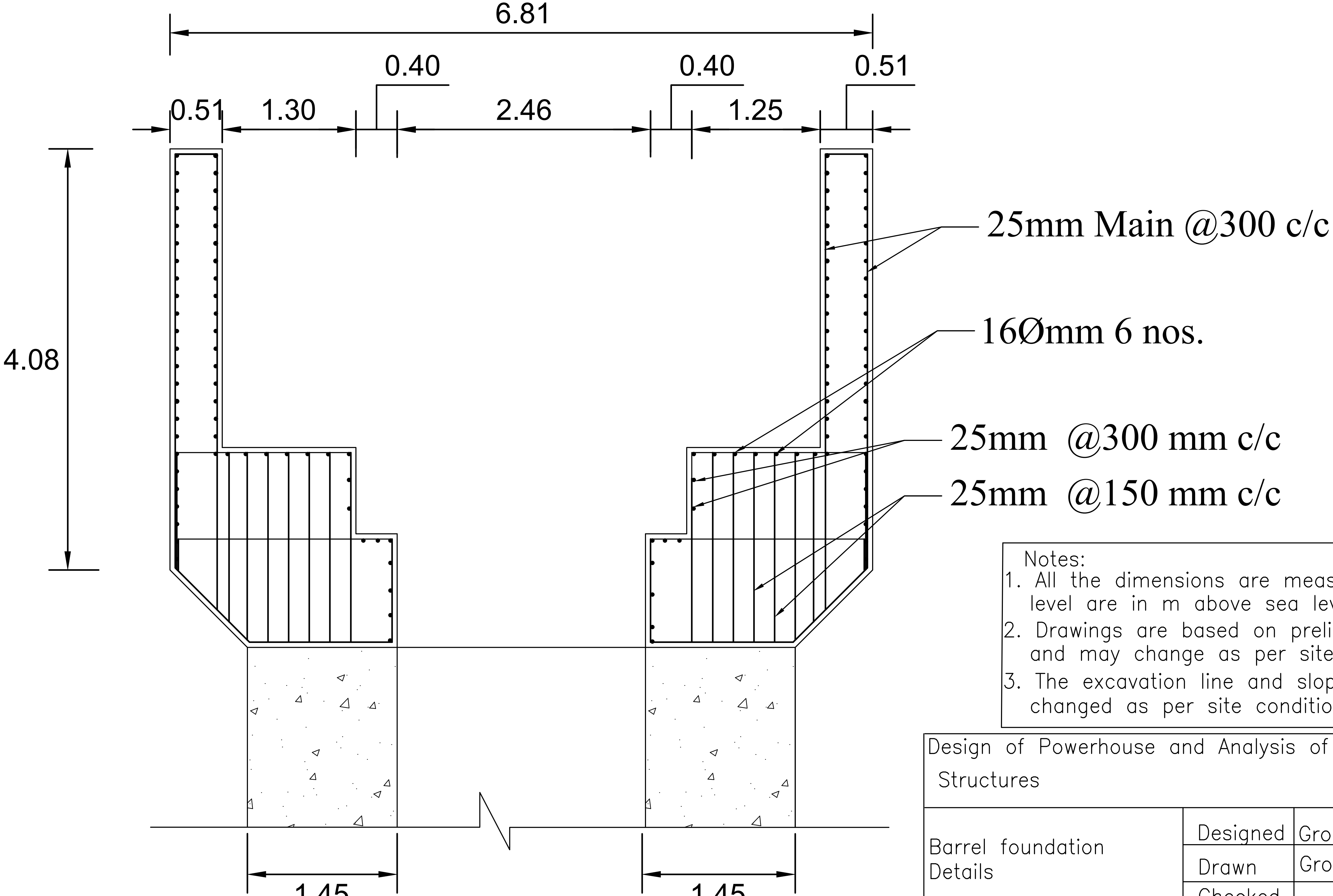
Fig: Column Layout for Mat Foundation
Scale 1:180



Section A-A
Scale 1:70

Notes:
1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Structures			
Mat Foundation Details		Designed	Group 2
		Drawn	Group 2
		Checked	
	KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre		Scale : As shown Sheet No: 25 Revision



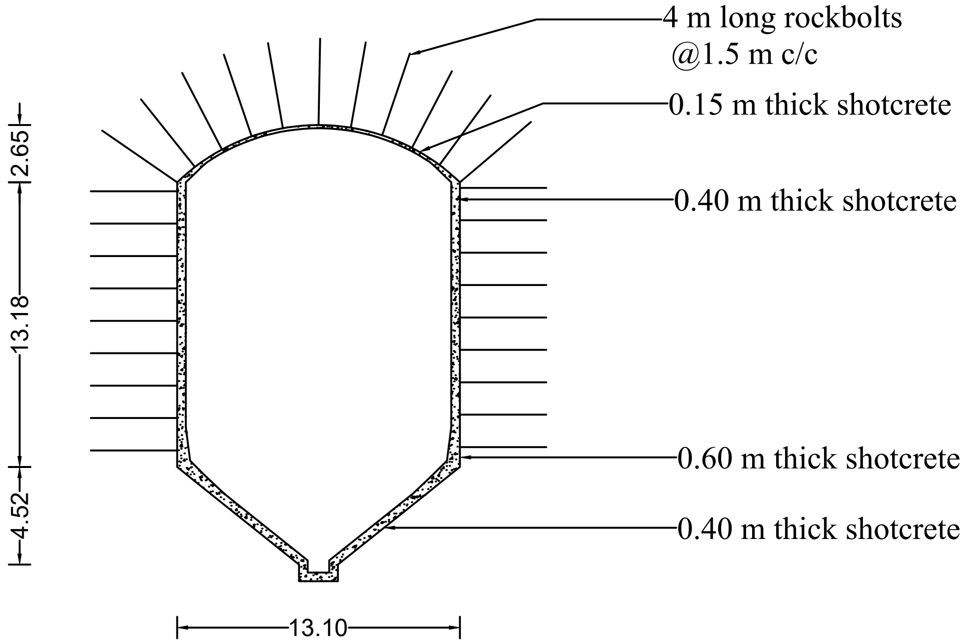
Scale 1 : 40

Section of Barrel Foundation

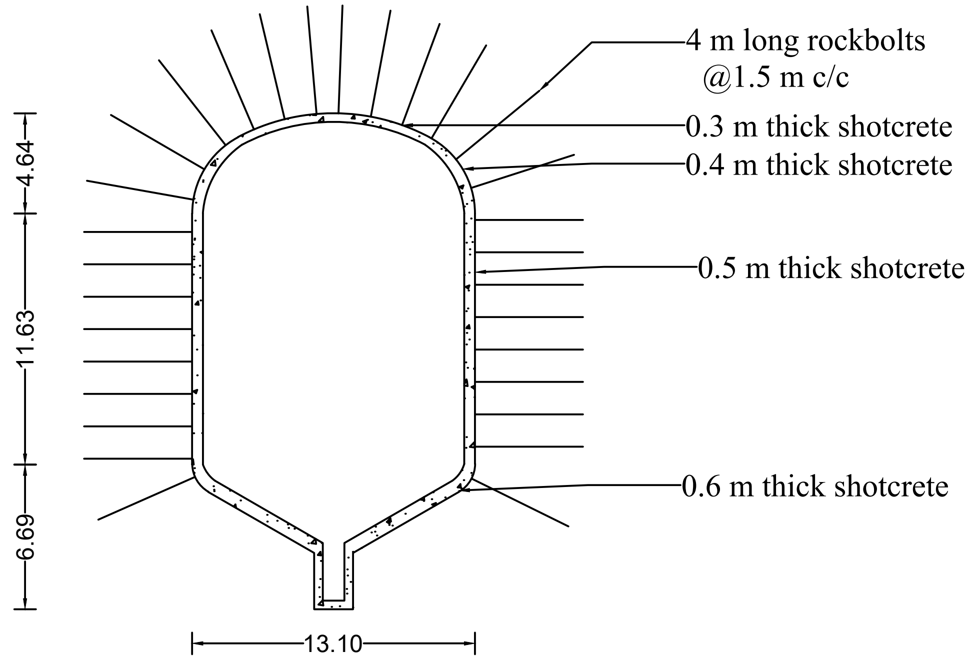
Notes:

1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

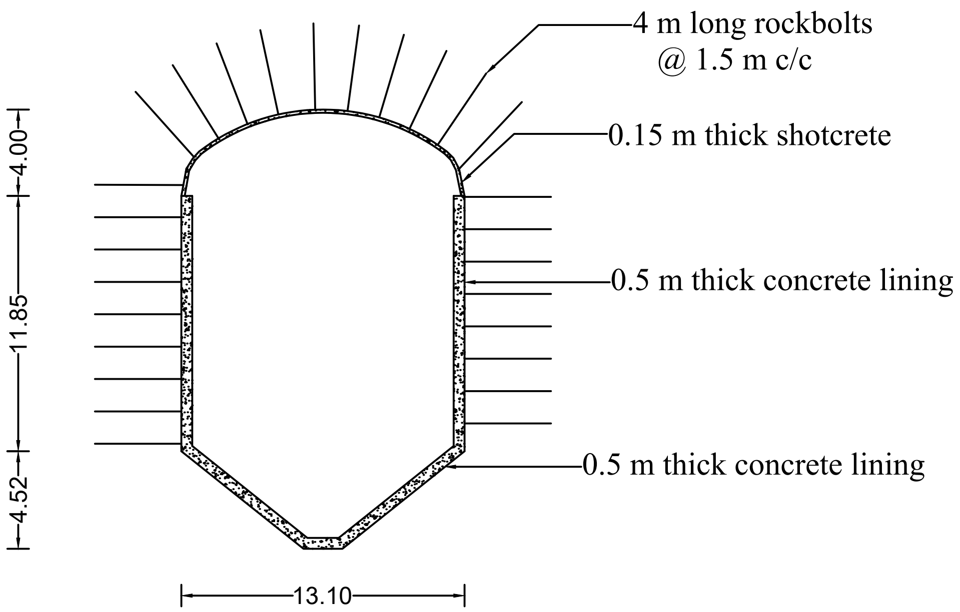
Design of Powerhouse and Analysis of Underground Structures		
Barrel foundation Details	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel, Kavre		Scale : As shown
		Sheet No: 26
		Revision



Chainage 0+104 m
Scale : 1:350



Chainage 0+190 m
Scale : 1:350



Chainage 0+089.20 m
Scale : 1:350

Notes:

1. All the dimensions are measured in m and level are in m above sea level.
2. Drawings are based on preliminary design and may change as per site conditions.
3. The excavation line and slope may be changed as per site conditions.

Design of Powerhouse and Analysis of Underground Strucutre		
Settiling Basin Underground Cavern	Designed	Group 2
	Drawn	Group 2
	Checked	
KATHMANDU UNIVERSITY School of Engineering Department of Civil Engineering Dhulikhel,Kavre		Scale : As shown
		Sheet No: 27
		Revision