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School of Graduate Studies

Faculty of Science

Engineering geological characterization of rock mass for the suitability of Tana Beles hydropower project powerhouse cavity

A Thesis
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Negash Anteneh

July 2007

This work is dedicated tothe memory of:

My mother, Kenubish Biadgie, whom I lost when I was in a way to take this position. I would have been very lucky had I been with her when she was requesting my presence. This occasion made me regret.
Mother was the source of good academic performance and strength of life activities for all of us (males and females). She was the base for what we are in.

Declaration

I, the under signed, declare that this thesis is my work and that all sources of materials used for the thesis have been duly acknowledged.

Name Negash Anteneh

Signature …………………..

The Thesis has been submitted for examination with my approval as university advisor.

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Signature……………………………..

Place and date of submit ion: School of graduate studies
Addis Ababa University
Addis Ababa

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ADDIS ABABA UNIVERSITY
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ENGINEERING GELOGICAL CHARACTERIZATION OF THE ROCK MASS FOR THE TANA BELES HYDRO ELECTIC CITY PROJECT POWERHOUSE CAVITY

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CHAPTER I INTRODUCTION

1.0 General background

Since ages most of the people of Ethiopia has been depending on firewood for different fuel consumptions. This long lasted practice of using wood and plants has resulted in the elimination of forest coverage and scattered trees and bushes in different parts of the country. By the consequence, erosion, degradation and formation of erratic landscape is becoming the current problem for the country. Whether directly or indirectly, this is one of the major factors for the increasing drought and poverty in the country. At present, the population of Ethiopia is increasing at an alarming high rate and with this rate the people cannot further relay on forest and plant products for their fuel consumption. At present only in some bigger towns/ cities electricity is being used for various domestic needs. However, majority of the population in the country still do not have access to the electricity for their daily energy needs. With the modern development pace, need for electric power is increasing rapidly and the efforts are being made by the Government to accelerate the activity of electrical power production and distribution in the country. Electrical power is not only important for the domestic needs but is the key component for the industrialization. For this reason all efforts are being made to harness the untapped resources of the country. The Government vision for power scenario is to electrify the entire country in the coming years and they also have a plan to supply the surplus electrical power to neighboring countries.

At present the country has been getting electric power from the previously built small and medium capacity projects in different parts of the country these are; Melka Wakena (153MW), Fincha (134MW), Awash (43MW), Koqa (43MW), Gilgel Gibe I (184MW) and Tiss Abay I&II (83MW). Presently, relatively larger scale projects such as; Gilgel Gibe II (420MW) Tekeze (300MW) and Tana Beles (460MW) are under construction. In addition, Gilgel Gibe III (1880MW) is to be started soon.

Tana Beles is a multi purpose project which envisages utilizing Lake Tana reservoir for electric power generation and irrigation. Lake Tana, besides its famous name for its larger extent, navigation service, fish resource site, climate regulation for Bahr Dar town and aesthetic value, it is going to serve for hydro electric power generation and irrigation projects. The project will utilize 160 m^3 /s of water and will generate 460 MW power and about half of the water will be utilized for down stream irrigation. The Cherechera regulation wear (dam) constructed at the outlet of Blue Nile from Lake Tana is the prelude for this project. The Tana Beles project is an under ground structure which consists of an intake structure on the SW of the lake shore near the Kunzla town, an 11.5km long and 7.2m diameter headrace tunnel, a

274m vertical penstock shaft, about 7km long tail race tunnel, underground powerhouse complex (powerhouse cavern, transformer cavern, galleries, draft tubes, manifolds, irrigation by pass, different access adits) and the down stream diversion wear. The Tana Beles project was initiated in 1964 as a component of the Blue Nile basin master plan study. The project is owned by EEPCO (Ethiopian Electric Power Corporation) and presently it is being constructed by SALINI Construction Company with close inspection by EEPCO engineers and geologists.

The present study focuses on the characterization of the rock mass and its stability in the powerhouse complex of the Tana Beles project. The general layout of the Tana Beles project is shown as Fig. 1.1 (a & b).

Under ground structures have been constructed all over the world for different purposes (water supply, transportation, irrigation, storage, power stations, military base, and others). However, in Ethiopia, it is not more practiced until very recently. It is obvious that though it could have its own advantages, under ground structures especially power house cavities are hazard bearing and costly. The underground powerhouse cavities, owing to their large dimensions, are more susceptible to failures due to high in situ stresses and discontinuity controlled instabilities. Such excavations often involve problems of unstable rock facing-swelling and collapse of facing, inflow of groundwater and rock collapse, eruption of gas and rock burst due to anisotropic stresses. It needs proper investigation, design and construction techniques. That is why a team of experienced engineers and engineering geologists are always assigned for such projects. Therefore, the role of investigation before construction and during construction becomes important. For this reason the present study focuses on the characterization of the rock mass and its stability for the powerhouse cavity (cavern). The powerhouse cavity has about 41m height, 105m length and 22 meter width. The over all implementation (excavation and support treatment) of this large under ground cavity requires the close inspection of engineering geologist to check suitability of excavation method, to collect data of rock mass on exposed excavation faces, analyze it and to recommend the necessary support measure considering safety and economy.

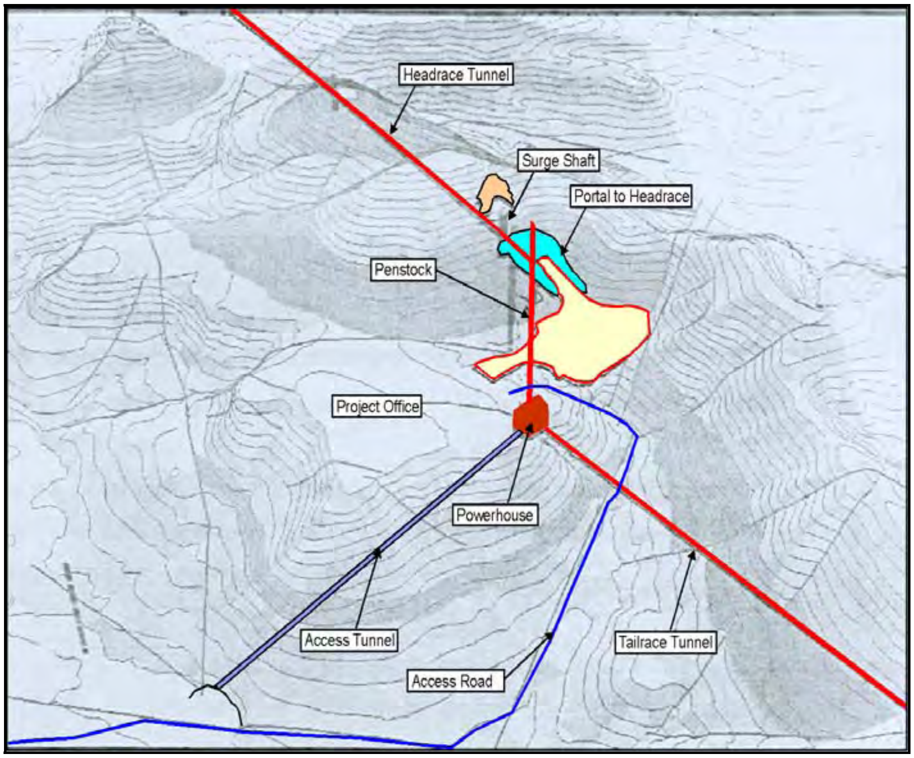


Fig.1.1(a) **General layout of the Tana Beles project and surrounding topography**

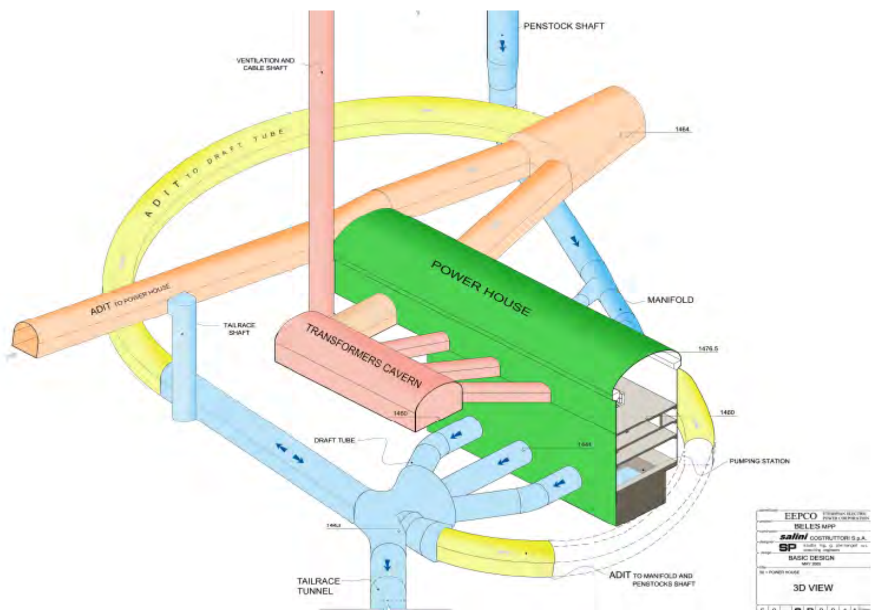


Fig. 1.1 (b) **General layout of Powerhouse cavity and associated structures**

1.1 History and Art of Powerhouse Construction

Under special circumstances, such as when a gorge or a valley forms the site of hydroelectric development, an under ground power station could be an alternative worth considering. An under ground hydro electric development would be an appropriate proposition when there are frequent earth tremors, or other physical surface hazards like rock slide or snow avalanches. An important characteristics of the under ground station is its flexibility of layout.

The shortest possible layout through various feasible alignments can be drown-up with minimum size of pressure conduits and omissions of anchor and synchronous relief valves. The basic requirement for the feasibility of under ground powerhouses is the availability of good sound rock at desired location and depth. Underground powerhouses are also safe as far air attacks during war time are concerned d (M.M. Dandekar and K.N. Sharma.1983).

Power house is that portion of a hydro electric development, where the conversion of energy of water to electrical energy takes place. The power house provides the following main equipments and facilities.

(i) Water conveyance structures –penstock draft tube etc.
(ii) Energy conversion equipments—turbine and generator etc.
(iii) Electrical energy control equipments—switchboard, control equipment etc.
(iv) Mechanical handling equipments ---cranes etc
(v) Office and other facilities
(vi) Distribution system---power sub-station transformer etc

1.1.1 History

Under ground power stations have their origin, in the construction of Verayaz station on the river Salanfe in Switzerland in the year 1897. However; it was built only partly with in the rock. Another development in this connection was Buchberguehle power station in Germany, built during 1904 -1909 which was also partly under ground. The first fully under ground station was constructed in 1911, at Mockfjards in Sweden and operated under a head of 24m, followed by the stations Porjus (1919-1922) in Sweden, Baton (1919-1925) in France and Bjoeskessen (1919-1921) in Norway. After the Second World war, under ground power houses become widely popular and by the end of 1968, there were more than 300 under ground plants with total installed capacity of 32,000 MW in Europe alone. A significant comparable progress has also been made in countries like Canada, Japan, USA, USSR, and India. Some of the world's notable under ground power stations are Portage Mountain, (British Columbia, Canada; 2300MW), Kemano, (Canada; I stage 832 MW, II stage 1670MW), Vianden (Luxembourg; 920 MW), Iddiki, India; 840 MW), etc. Since then, a number of huge underground power stations are being built on the consequence of the demand and sophisticated technologies.

1.1.2 Types of Powerhouse Locations

Depending on the rock quality, tunneling ease and overall economics power houses may be located in different ways (M.M. Dandekar and K.N. Sharma.1983).

(i) The plant totally under ground. Under such a case, the power house has an access through a system of tunnels (Fig. 1.2 (a)).

(ii) The power house located in a pit, the access is directly from the surface (Fig. 1.2 (b)).

(iii) The power house semi-under ground. In this case, some units such as generator may be located on the surface while other units, such as turbine may be under ground. (Fig. 1.2 (c)).

(iv) The power house located in a cut. Where stable rock exists, the units may be placed in a cut in the rock. (Fig. 1.2 (d)).

(v) The buried power house. Here, after locating the units of the power house, the trenches are back-filled. (Fig. 1.2 (e)).

The Tana Beles power station, considered for the present study, is an under ground structure.

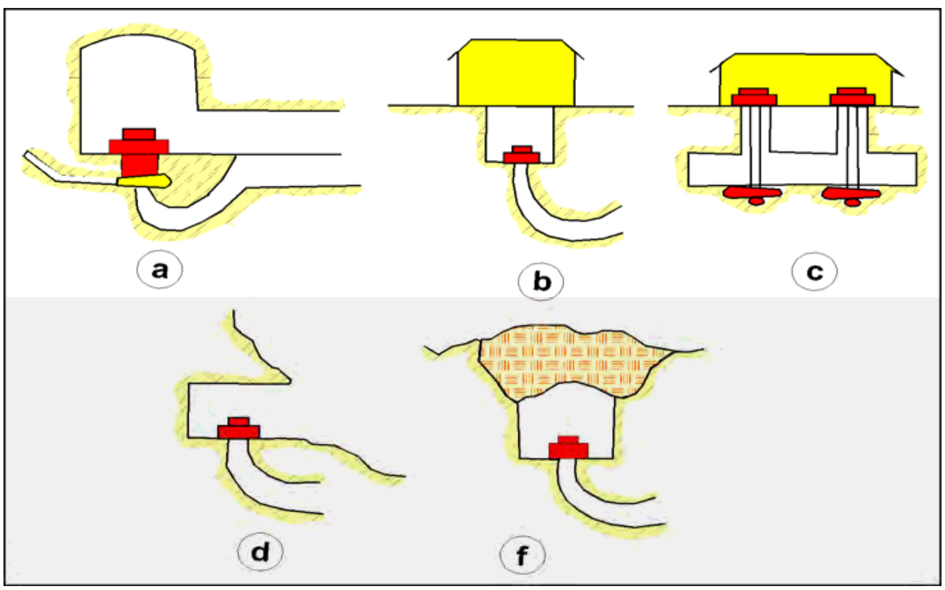


Fig.1.2 Locations of underground power stations.

1.1.3 Types of Underground Power Stations

According to Mosonyi,(as cited in the book of water power engineering by M.M. Dandekar and K.N. Sharama(1983), the various layouts of the power stations could be described with reference to the head and tail ponds as:

(i) Upstream station or head development

In this type, power station is located close to the intake and thus, water is directly fed from the head pond to the generating units. Usually such arrangement is suitable for low head and high discharge conditions in the continuously sloping or mildly rolling terrains. The head utilized in such developments varies generally between 25 and 50 m and in exceptional cases, up to 100m (Fig.1.3 (a)).

(ii) Down stream station or tail development

This type of development has its characteristic in a long and nearly horizontal pressure tunnel together with pressure shafts and a short tail-race tunnel. Such development is most suited for a rugged terrain and high head of the order of several hundreds of meters can be utilized. The Tana Beles power station belongs to this group (Fig.1.3 (b)).

(iii) Intermediate station development

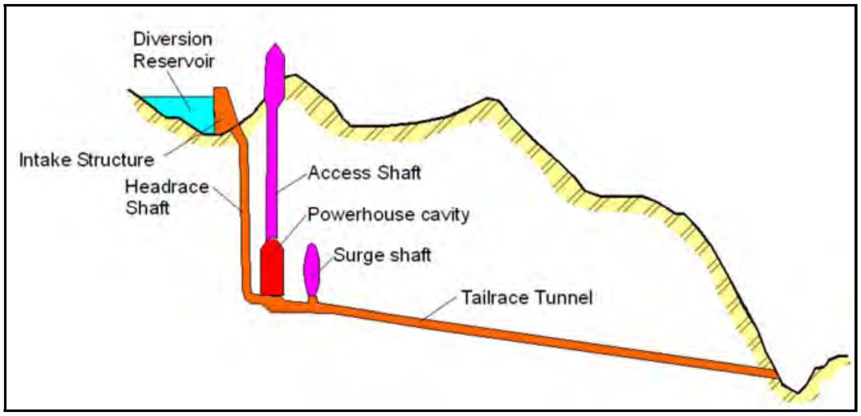
The characteristics of this type of arrangement are a long head–race tunnel and a long tailrace tunnel. The consequent pressure variations due to long tunnels are taken care of by surge tanks, both upstream and down stream of the powerhouse. In such arrangement, good use is made of the topological conditions. For example; a deep saddle dividing two mountain tops can be conveniently utilized in an arrangement of a horizontal tunnel extending up to the saddle followed by a pressure shaft up to the power house which is situated approximately below the saddle point and then as long as tail race tunnel under the second ridge to convey water in to the down stream water carrier (Fig.1.3(c)).

1.1.4 Advantage of Under Ground Power Stations

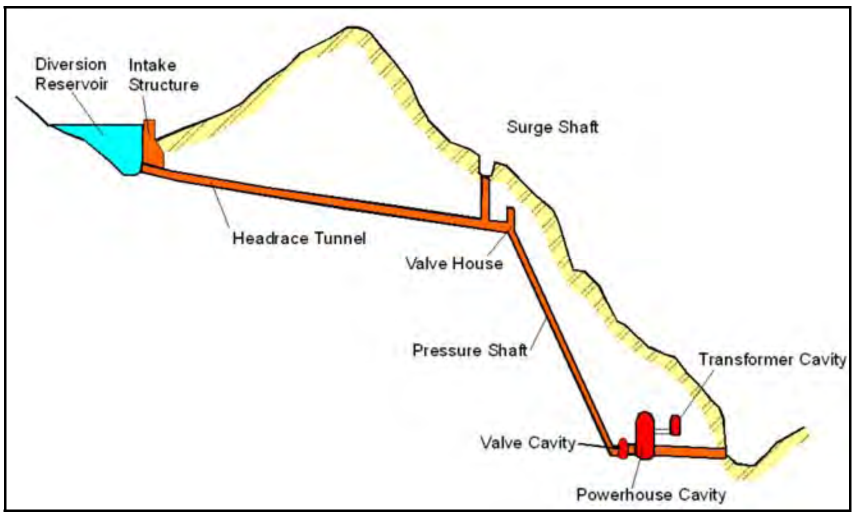
To summarize the advantage from the construction of an under ground power house, the main aspects that could be studied (Mosonyi) are as under:

(i) Economical aspects
(ii) Operational aspects
(iii) Natural conservation and land protection aspects
(iv) Defense requirement aspects

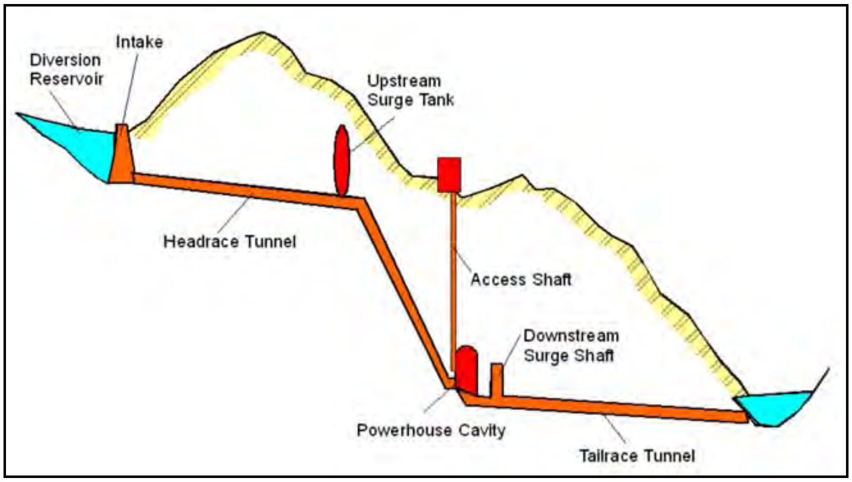
The study and observation of many existing and proposed under ground power houses vis-a vis surface power stations reveal that under suitable geological conditions with complicated sinuous (winding) terrains, it is generally economical to construct under ground stations. This is due to the short length of pressure tunnels, shafts, penstocks which are possible to achieve, because then, it does not become necessary to follow the contours of the terrain while laying the power conduits. The reduced length of pressure tunnels also helps in cutting down the surge–tank dimension and water hammer pressures. Moreover, because of the comparatively smaller lengths of pressure conduits, the operational costs are reduced. This is so, on account of smaller head losses and lower maintenance and repair costs. An aesthetically significant advantage offered by an under ground power house station is the preservation of the natural landscape features.



(a) Upstream type station



(b) Down stream type station



(c) Intermediate type station

Fig. 1.3 Types of under ground power stations

The surface power station would necessitate deforestation, interference with natural landscape on account of the material transportation and constructional activities. Although the biggest single feature in favor of an under ground power station is the economy, yet an extra advantage achieved is the immunity of such construction from air attacks or warfare and from natural damage (snow or rock avalanches, mountain lope sliding). As compared to the surface power stations, the under ground station experience appreciably lesser effect of earthquakes. From some records of such effects, underground structures are found to be affected by half as compared to surface structures. This is because the destructive nature of seismic effects is manifested by surface waves damaging surface structures.

1.2 Locations

The Tana Beles hydro power project site is located in West Gojjam, near Kunzla town (small town where the intake structure is sited). The Tana Beles multi purpose project covers about 20km distance from the intake to the outlet. The powerhouse is located at 2/3 of the total length of the project (12.5km down stream of the intake and 7.5km up stream of the outlet on the Jehana stream) and is an underground about 300m below the surface. The cavity is sited along the Beles valley about 11.5km SW of Lake Tana and at a distance of 20km from Kunzla. The site extends on the left side of Beles valley approximately between elevations 1620m a.s.l. (adit to powerhouse portal) and 1872m a.s.l. (surge shaft portal) and between UTM co-ordinates 1306800-1307200N and 273200-273800E.

To access the project, from Addis Ababa the rout deflects left at Durbete (Woreda town) 64 km short of Bahir Dar town and needs a drive of about 70 km on all weather road (Fig. 1.4).

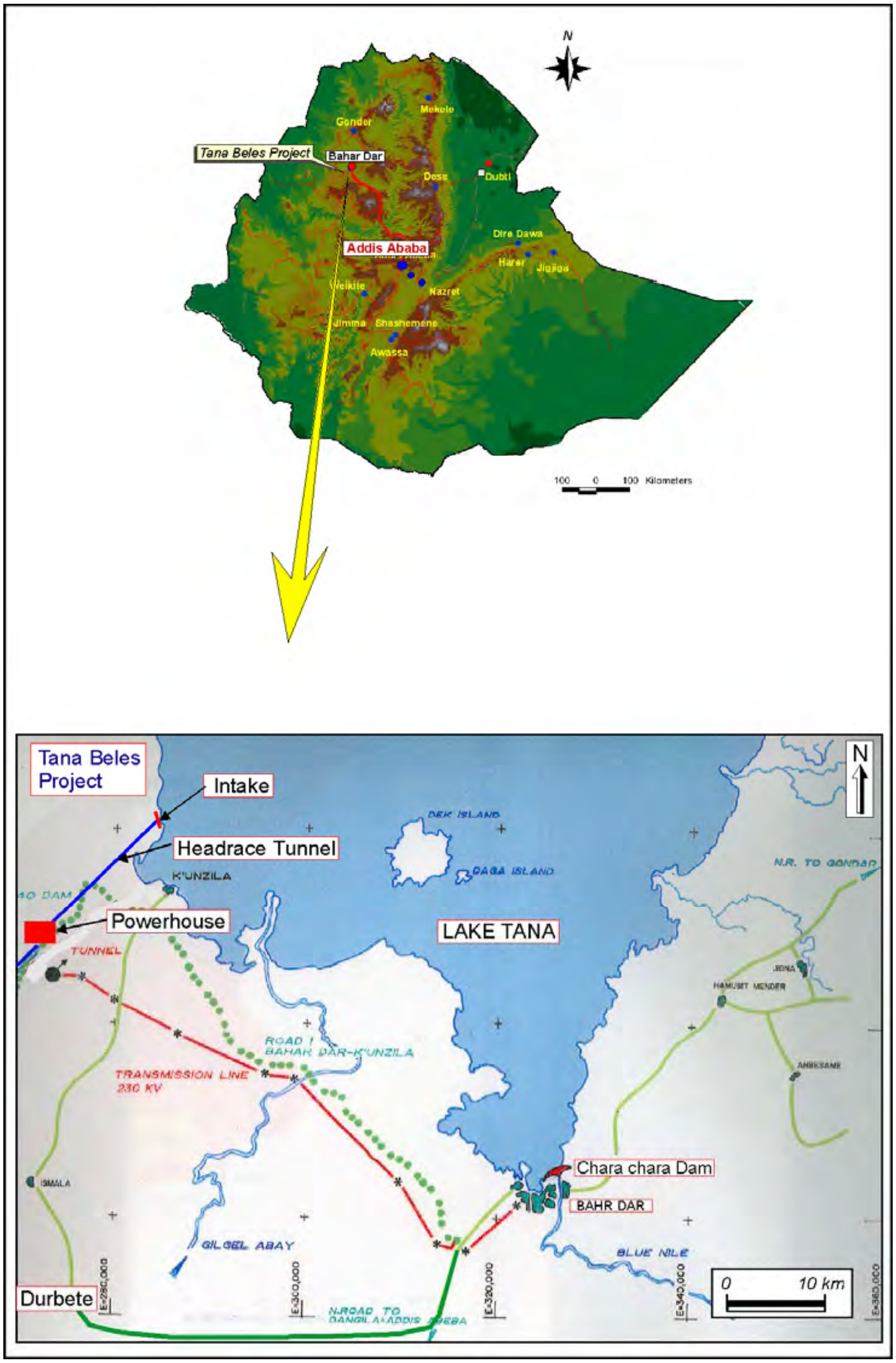


Fig. 1.4 Location Map of the study area

1.3 Climate and vegetation

The project site is located where an abrupt change begins from the upper Tana Beles plain to the Beles Valley. That is where the extended area south of Lake Tana plain abruptly changes to the Beles valley just at the left flank of Jehana River which, is the upper tributary of Beles.

Hence, there is an altitude change (variation) from that of Lake Tana plain area. However, according to the Ethiopian climatological zone classification, the power house site still falls in Woina dega (temperate) division (about 1620m a.s.l. around portal) though the excavation inverts fall in the range of Kola climate (1441m a.s.l). The summer has warm sunny days. It gets highest temperature in the months of March, April, May and June and lowest in the months of July, August, September and November. The average annual temperature is about18˚C (ranges from16˚C to 20˚c). The rainy season extends from end of June to mid of September. The mean annual precipitation is about1500mm. The project area has no vegetation coverage to be mentioned. There are only remenant trees on plots of farmlands and round individual houses. Bushes are scattered as small patches and there protected area with good coverage downstream side of the outlet.

1.4 Previous works

The general geological and structural condition of the region has been studied by many researchers such as Mohir (1971), Kazmin (1975), Mohr and Gounin(1967), Comucci(1850), J.Chorowicz, T.Korme,P.Mohr, B.collet and F.F. Bonavia(1998).

The site is identified as potential hydropower project location during the Blue Nile Basin master plan study in 1964. The geological mapping of the area in a regional and project level is done during the pre-feasibility study of the Tana Beles project as part of the Blue Nile master plan in 1999. This pre-feasibility study has included detail investigations of some parameters using borehole logging, sampling, laboratory tests and analysis of the engineering geological properties of the rock and soil materials.

Next to pre-feasibility study, the Tana Beles hydro power project site is specifically studied from the intake site to the down stream (outlet) portion for the feasibility of the project in 2005 by Salini Constructorri S.P.A. studio. This stage of investigation includes borehole logging, different in-situ and laboratory tests, geological mapping, and geophysical investigation. Engineering geological and hydro geological investigations were carried out along all the components of the Tana Beles hydropower project with rock mass characterization and classification, design preparation, selection of the different excavation methods and with recommendation of proper supporting measures. Especially, the powerhouse cavity is investigated with concentrated boreholes and cross sections. According to this study, location of the powerhouse has been selected in order to optimize the following basic requirements.

- Suitable geological conditions (competent rock and clear from faults of major and major fracture systems), evaluated based on the available geological information and studies.

- Allow an easy access with the shortest possible adit.
- Maintain a short and simple waterway arrangement.

1.5 Objectives

The main objective of this study is to appraise the suitability of powerhouse cavity for powerhouse location and check the appropriateness of the excavation method and employed treatment measures. in accordance with

In addition, the following are the specific objectives as a prelude of the main objectives.

- To characterize the engineering geological properties of the rock mass
- The rock mass classification and characterization of cavity roofs and walls.

- To work out the stability condition of the cavity.

1.6 Methodology

Taking in to consideration the stage of the project, different methodologies are employed based on the intended objectives. Methodologies are devised to gather the maximum amount of information available with in the allocated time budget and resources.

The following are the employed systematic methodology for the present study;

- Review of literature
- Field data collection for the various properties of the rock mass:
  Discontinuity data survey, unconfined compressive strength (UCS), rock quality designation (RQD), rock mass rating (RMR), rock quality index (Q)
- In-situ tests such as Schmidt hammer and point load tests
- Analysis and synthesis of data
- Interpretation of the analyzed data and the suggestions about the over all performance of the power house cavity construction

1.7 Importance of the study

The study is expected to devise mechanisms and suggestions based on the analyzed and interpreted primary and secondary data sources. Under ground excavation (tunneling) is dangerous that it can cause lose of life and property at the first hand. In addition, non functioning (failure) of the project may occur after construction due to improper remedial measures. Such project, which has the probability of causing damage and is also costly, needs proper investigation and construction care to reduce the expected risks and cost of the project. Hence, the study will have the following application or advantages.

- It gives practical experience for the researcher and hence it is great resource for the respective sectors.
- Experience sharing had taken place with the contractor and employer geologists on rock mass characterization and classification
- The material can serve for the department as a source document for the future researchers to continue their research in the similar areas.
- It takes in to application the theoretical concepts of the different subjects taken academically. Hence, it will be an input to review and to see the applicability of the subject matters.

1.8 Limitations

On the process of completing this thesis there have been lots of limitations faced among which the major ones are listed below.

- Insufficient allocated time for proper compilation and completion of the research work.
- Financial limitations to perform activities as desired.
- Non-availability of secondary data (documents) at hand to refer.
- The methodology of underground construction does not give chances to collect primary data on previously excavation faces since they are treated soon. Again it needs waiting for extra excavation faces to collect data and hence needs to stay long during construction activity.

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CHAPTER II GEOMORPHOLOGY AND GEOLOGICAL SET UP

2.1 Geomorphology

The water shed line between Lake Tana and upper Beles valley runs at a distance of 5km from the bank of the Lake Tana. The side facing of the Lake Tana is characterized by a slight inclination in the range of 5^0 and by very gentle sloping morphological inclination (fig. 2.1). On the contrary, the side facing of the Beles valley slopes steeply down to the valley bottom with a sharp morphology, moderately deep incisions of the tributary valleys. Gently sloping portions of the valley sides with rounded forms alternate with steep rock faces generally corresponding to the lava out crops. The valley bottom of Beles is narrow and cut through rocky banks, however, it may open out in some tracts being bordered by alluvial terraces.

The general morphology of the area is characteristic of central Ethiopia and is constituted by a high plain in which Lake Tana is located, with small weir (Cherechera) at the Blue Nile outlet to serve as reservoir regulation and the gully of Beles valley to the west which determines the hydraulic head for the hydroelectric exploitation of the water from Lake Tana. The high plain has an elevation between 1785m and 2300m to the west along the ridge which separates from the Beles valley. The slopes which descend from the high plain are particularly steep and rocky. Along the Beles valley, the land is characterized by ridges that rise 200-400m above the valley bottom with the upper parts rounded or steep depending on the presence or otherwise the emerging lava flows. The ridges divide the network of the various tributaries of the Beles creating dendritic type drainage pattern. Once the watershed dividing the Tana basin from the Beles valley has been crossed, the route of the head race tunnel passes through the hydrographic left ridge of the Beles valley in order to gain height and thus increase the power and energy which may be generated.

The powerhouse area is located on the left (east) side of the Beles valley. The morphology of the area is characterized by a typical “mesas and cuestas” (terraced) profile produced by the intersection of sub horizontal strata with different resistance to erosion (basalt, tuffs). It must be noted also that the site is placed at the junction between the main Beles valley (oriented in the north east – south west direction). The Beles valley floor is flat and wide with gentle slopes at the base that gradually steepens upwards up to elevation 1770m a.s.l. At this elevation, there is a morphological terrace few hundred meters wide characterized by a sub-planar surface. This terrace had used to locate the project offices and main facilities. Further, above the terrace slopes become steeper with inclinations ranging from 30-35^0. Hill- tops around the area are almost planar. The topography of the study area is shown in Fig. 2.2 on

3d elevation with Powerhouse location.

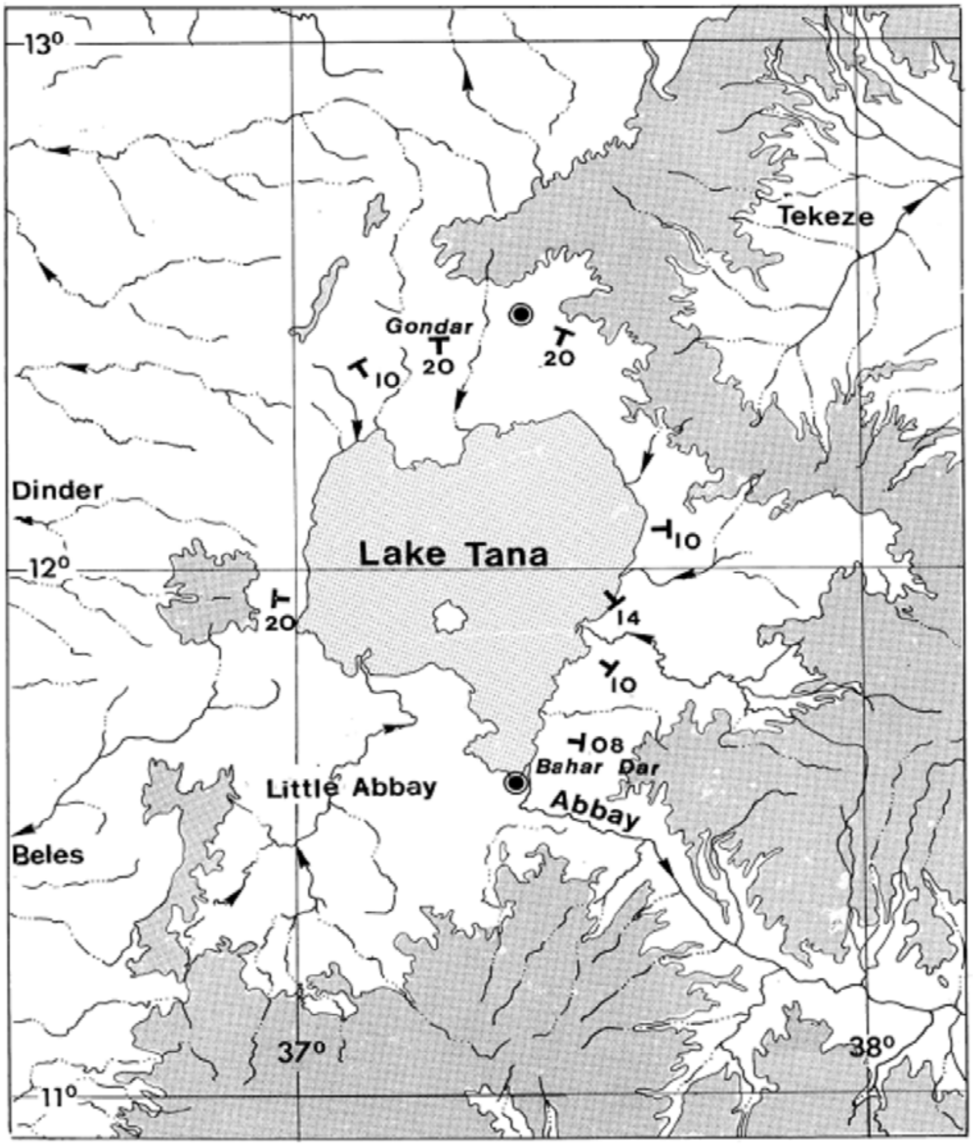


Fig 2.1 Drainage pattern of the Tana Basin

(Source; Chorowicz et al, 1998)

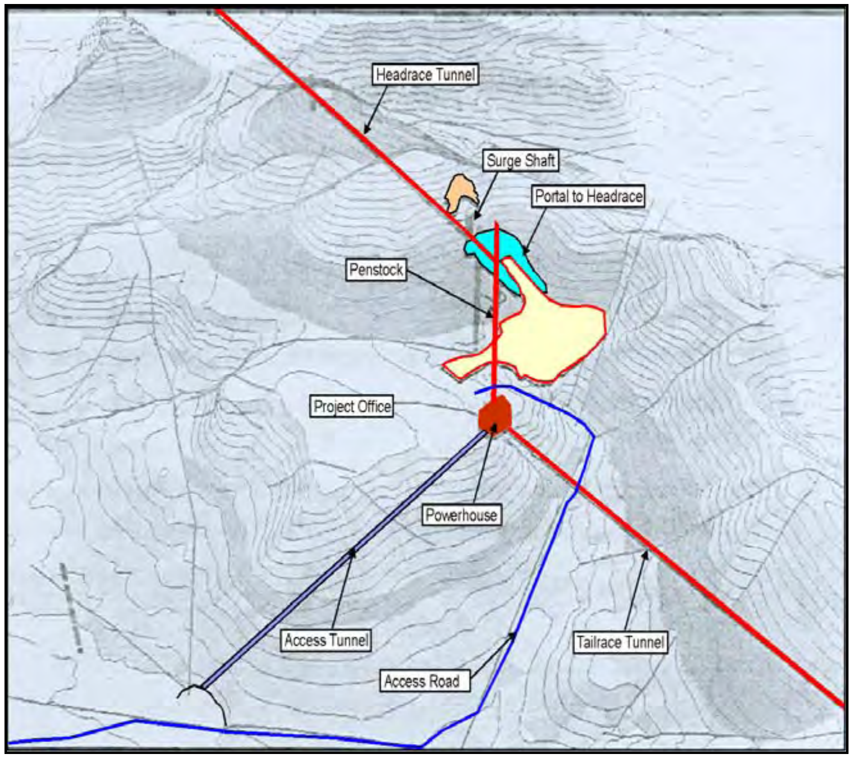


Fig. 2.2 3D model obtained from draping the aerial photo on to digital elevation model

(Source: Feasibility investigation report part 'A')

2.2 General geology

The study of the geology of Ethiopia dates back to 1860 by Blanford, and since then, major advances in the understanding of the geology of Ethiopia have been made from the works of various researchers, e.g. Danieli(1943), Mohr(1971), Kazmin(1972,1974,1978).

At present about 30% of Ethiopia has been mapped at 1:25,000 scale. This distribution of lithologic varieties show that the country is covered by about 18% Proterozoic crystalline basement, 25% Mesozoic sediment and 56% Cenozoic volcanic and sediments (Solomon Gerra, 2000). The generalized geological map of Ethiopia and the region are shown in Fig. 2.3 and Fig. 2.4, respectively.

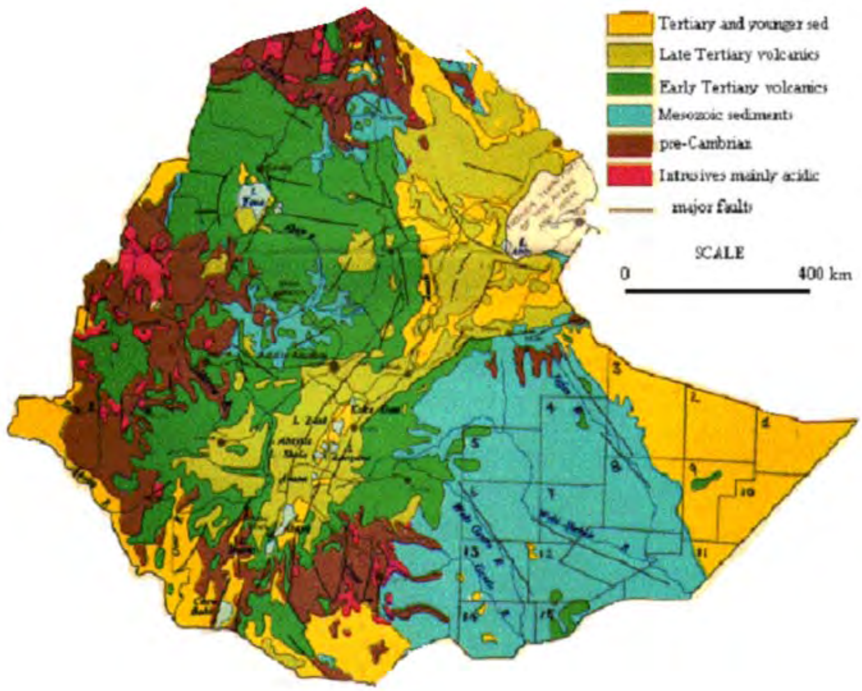


Fig. 2.3 The Generalized Geological Map of Ethiopia

The three major geological terrains recognized in Ethiopia are:

(i) Proterozoic crystalline basement.

(ii) Late Paleozoic to Mesozoic marine and continental sedimentary rocks.

(iii) Cenozoic, basic and felsic volcanic and associated sedimentary rocks.

2.2.1 The Proterozoic crystalline basement

This basement is divided in to three different litho tectonic units.

- The lower complex unit: - consist of high- grade biotite- amphibole gneiss with minor quartzo-feldsphatic gneisses; calcic silicates and amphibolites, affected by faulting and folding.

- The middle complex: - Meta-sediments in which the primary depositional structures are locally preserved. It contains schist, marble, meta-arkoses with patches of ophilites, amphibolites, and other basic meta-volcanites associated with graphitic schist phyllite and quartzite. This unit is also affected by faulting and shearing.

- The upper complex: - this is the youngest and least deformed Precambrian assemblage principally composed of detrital sediments and carbonates.

2.2.2 Late Paleozoic to Mesozoic marine and continental sedimentary rocks

The Paleozoic era in Ethiopia is marked by regional unconformity due to long period of peneplanation. Very few Paleozoic residual deposits containing Precambrian basement (debris and agglomerates) are observed on the peneplained surface in Northern Ethiopia (Enticho Sandstone). The Permo- Carboniferous, glacial deposits of Ediga Arbi, which is about 160m thick, represents of Paleozoic deposits in northern Ethiopia. The Mesozoic sequence in most part of Ethiopia consists of three major sedimentary units. The basal sandstone (Adigrat formation) is a deltaic near shore sand stone, of late Precambrian–early Jurassic age and was deposited during transgression. The basal sand stone unit is overlain by mid-late Jurassic limestone, shale and gypsum of the Antalo formation. Early Cretaceous Sand stone that marked the regression of the sea caps the Mesozoic sequence (Kazmin, 1974).

2.2.3 Cenozoic, basic and felsic volcanic and associated sedimentary rocks

The Cenozoic volcanic sequence rests either directly on Precambrian basement or on Mesozoic sedimentary sequences. The volcanic succession consists of thick (up to 2500m), distinctly bimodal sequences of basalt and rhyolite, underlying low angle shield volcanoes (Tarmaber formation).

2.2.4 The Cenozoic Ethiopian Volcanic Province

Following the late Mesozoic-early Cenozoic regression of the sea to the east and south east an epirogenic uplift of Afro-Arabian (East Africa Penisula and the intervening regions now occupied by the Red Sea and Gulf of Aden) occurred on an immense scale (Cherenet, 1998). According to Mohr, (1963), the magnitude of uplift was such that no where in the world outside the orogenic belts basement rock been uplifted to such an elevation (e.g. 300m Mt. Assimba, Tigray) as that associated with Afro- Arabian swell. The cause of the first eruption of flood basalts, a mantle plume upraised the land mass which fissured under tension and permitted the ascent of magma generated by high degree of decompression melting in the mantle to form the flap series.

2.2.5 Volcanics of North Western Plateau

The plateau volcanics is cut by the rift faulting and are distributed asymmetrically about the Afar and Main Ethiopian rift. That lying to the west of the rift system is designated as the Ethiopian Western Plateau (EWP). The EWP comprises the northern, central and south-western sectors, whereas South-Eastern Plateau (SEP) includes the eastern, south-eastern and southern most part of the Ethiopian flood volcanic province. In the north Western

Ethiopian plateau (WEP), the volcanic succession is emplaced on the sub-horizontal Mesozoic transgressive and regressive sedimentary strata. This volcanic plateau does not fit the popular image of continental flood basalt province in that it is not a thick monotonous, rapidly erupted pile of unreformed flat lying tholeitic basalts. Instead it is made up of several distinct volcanic centers with different magmatic character and with a large range of ages (Kieffer et al., 2004), (Fig. 2.4).

The major volcanic unit of Western Ethiopian plateau (WEP) includes;

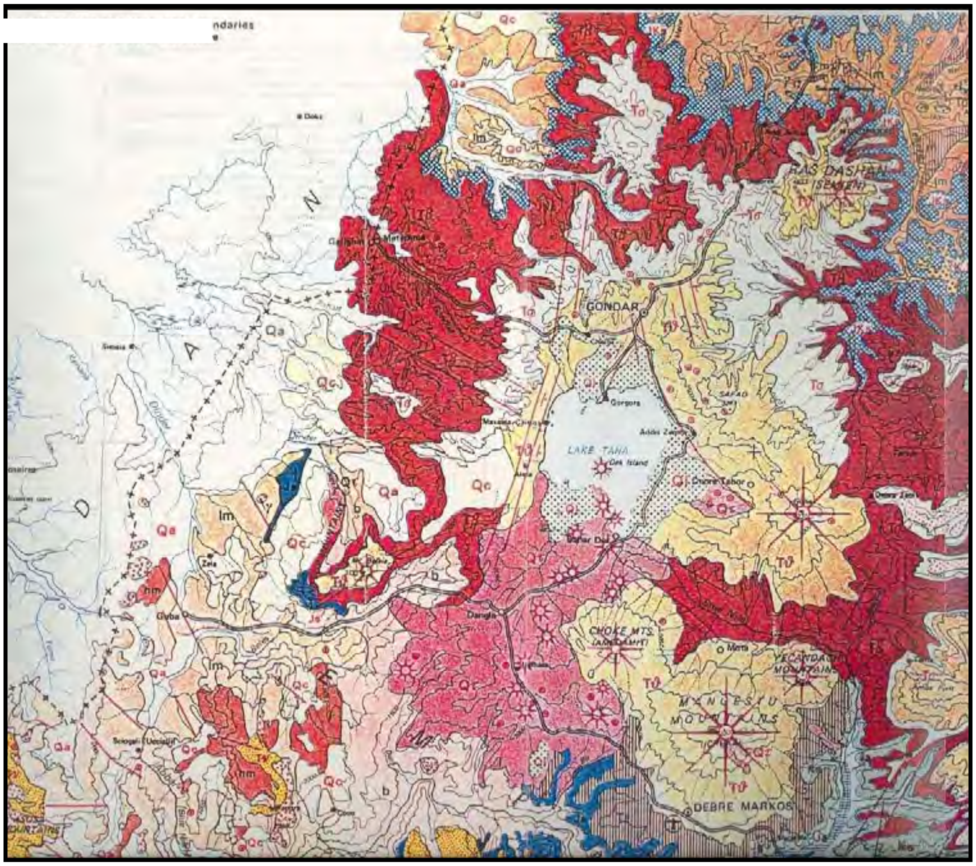
- Oligocene flood volcanics (Trap series) i.e. (Oligocene-Miocene basalts and rhyolites).
- Miocene-Pliocene shield volcanos
- Volcanic plugs and domes
- Quaternary volcanics

2.2.6 Oligocene flood volcanoes (Trap series)

In the Western Ethiopian plateau (WEP) the flood volcanic succession includes basaltic lava flows basaltic tuffs, as well as a considerable volume of rhyolitic and phonolithic products (Mohr and Zenettin, 1988). Intermediate lavas are lacking and the volcanism is of distinctly bimodal basalt, rhyolite type (Chazot and Bertand, 1993) a feature common to most continental flood basalt provinces (e.g. Karoo and Parana).

2.2.7 The Mesozoic- Pliocene shield volcanos (Tarmaber formation)

The flood volcanism was succeeded by emplacement of large shield volcanoes and by continental rifting (Mohr, 1983, Hoffman et al.1997). A number of large shield volcanoes developed on the surface of the volcanic plateau overlying thick sequence of flood basalt. These volcanoes are conspicuous features of the Ethiopian plateau and distinguish it from other well known, but less well preserved flood basalt provinces such as the Decan and Karoo. Like the flood volcanoes the shield volcanoes are bimodal and contain sequences of alternating basalts, rhyolitic and trachytic lava flows, tuffs and ignimbrites, particularly near their summits.



Legend

1. Quaternary – Alluvium (Qa) white; Eluvial and Colluvial deposits (Qc) light yellow; Lacustrine deposits (Qj) black dots on white; Basalt ($Q\tau$) dark pink
2. Pliocene- Miocene- Tarmaber basalts with tuff and Paleosoils ($T\vartheta$) yellow; Wellega Basalts with acidic tuffs ($T\nu$) red dots on yellow
3. Miocene- Amba Alaji basalts($T\sigma$) ($m\alpha$) red dots on white
4. Miocene – Oligocene- AmbaAiba basalts ($T\sigma$) white; Ashangi basalts ($T\beta$) dark reddish
5. Paleocene?- Oligocene?- BlieNile basalts (ε) vertical hatch
6. Cretaceous- Jurassic – Upper sandstone (Ks) green dots; Antalo limestone (Ja) white
7. Cretaceous? - Jurassic? - Tekeze sand stone (JKs) white dots on blue
8. Jurassic- Abay Beds, gypsum and dolomite lime stones (Jg) horizontal hatch
9. Jurassic- Triassic? - Adigrat sandstone (Ts) blue

10. Paleozoic- pre-Cambrian – Granites and quartz- diorites (γ) red dots on pink
11. Precambrian- Undifferentiated metamorphic sediment (b) pink
12. Low-grade metamorphics (lm) light brown; Gneisses, migmatites, amphibolites (hm) brown; marbles and Limestone (m) red vertical hatch
13. Faults in red; Volcanic centers (red rays)
14. Major plugs (red centered rounds)

Fig 2.4 Regional geological map (taken from geological map of Ethiopia and Somalia, by G. Merla and Others (1973))

2.2.8 Quaternary Volcanics

Quaternary alkali basalts (Tana Lava) occur related to local rift structures north–south trending extensional faults, (Chorowicz et al, 1998). Volcanic cones and flows of scoraceous basalts are well preserved in the Lake Tana graven. These basalts are considered

Pleistocene in age. The Volcanic rocks of Lake Tana area are usually described as olivine alkaline basalts (Merla et al., 1979), which may have a thickness of up to 1300m (Mohr, 1971).

In the recently compiled geological map of Ethiopia, (Tefera et al., 1996) described the rock as a plateau basalts, consisting of quaternary alkaline basalts and trachaytes. The region south of Lake Tana exposes quaternary volcanic rocks composed of vesicular alkali basalt and cinder cones indicating the volatile rich nature of the host magma.

2.3 General Geology of the Area (Lake Tana to Upper Beles Valley)

From a geological point of view, Lake Tana represents a natural shallow catchment area formed in the quaternary period due to the combination of tectonic subsidence and volcanic sills (Pre-feasibility report of Tana Beles project, 1990).

In the South-west quadrant, where the present study has been conducted, the Lake Tana Basin is bordered by recent alluvial deposits and further up sloping by Quaternary volcanic – lacustrine deposits (tuffites, and clays with lignite matter) are present. While In the southern portion, recent basaltic volcanics (Quaternary younger volcanics) have covered the area.

The ‘Younger’ volcanics and the volcanic-lacustrine deposits overlay the tertiary basaltic complex (‘older volcanics’) which is present in the western, northern and eastern sides up to about 30km from the lake.

In the present report the term “Older volcanics’ used is inclusive of both the Mio-pliocinic “Tarmaber basalt” and the older “Ashangi basalts” which are mentioned in the preceding geological studies. In particular the “Older volcanics” out crop in the saddle area of the watershed line between Lake Tana and the Beles river as well as in the upper Beles Valley. While at the foot of the side which faces the lake, these are covered by the quaternary volcanic lacustrine deposits on which the more recent coastal alluvial deposits are present. The volcanic lacustrine deposits overlie an ancient topographic surface modeled in the order of volcanics and fill the original depressions. Along the valley of the Tikur wiha river tributary of the lake, the volcanic lacustrine deposits continue to the distance of approximately 3km from lake bank forming part of the left side of the valley up to an elevation of about 1880m above the sea level. On the right bank of the tributary and in the river bed, the tuffites come in to tectonic contact with the basaltic complex through two faults. The first, strike N 70˚E and the second striking NW. Both have covered the block on the lakeside and are linked to the latest stage of the subsidence of the bank.

In the Beles valley, the “Older volcanics” overlies the Precambrian crystalline basement; the contact passes along the valley bottom in the section between the confluence of the Ali and the Buhl. In the lower Beles valley basaltic quaternary lava flows ("Younger volcanics") have in part filled up the valley bottom by overlaying the basement rocks and the "older volcanics". Down stream of the elevation 1450m a.s.l. at the river bed, terraced wide and thick alluvial deposits are present on both the sides of the Beles valley, up to 50km above the present river bed.

2.3.1 Geology along the Project Components (Inlet to Out Let)

In the stretch of open air diversion canal, recent alluvial sediments are present together with quaternary volcanic–lacustrine deposits. These deposits are also present along the short stretch of the tunnel in correspondence with inlet on Lake Tana side. Rest of the tunnel works will be carried out exclusively in the complex of the "older volcanics". Based on the feasibility report of the project (2005), the surface geology is represented by the following units (Fig.2.5).

- ALL - alluvial deposits (gravel, sand and clay) from recent river floods.
- ALT - alluvial terrace deposits (gravel, sand and clay) from ancient deposits.
- RA - sand, silty and clayey fluvial-lacustrine deposits.
- RL - silty and clayey lacustrine deposits.
- B3 - third Quaternary basalt flow
- B2 - second Quaternary basalt flow
- B1 - first Quaternary basalt flow
- QT - Quaternary pyroclastic rocks (ash tuff and lapilli tuff)
- QL - Quaternary lacustrine deposits (layers of silt stone and clay stone, sometimes with coal and lignite elements interbeded with tuff)
- DY - Tertiary dykes (dolerite)
- ABA- Tertiary aphanitic basalt flows
- TT - Tertiary tuffs and lapilli tuffs
- VB - vesicular scoraceous basalt
- BB - brecciated basalt

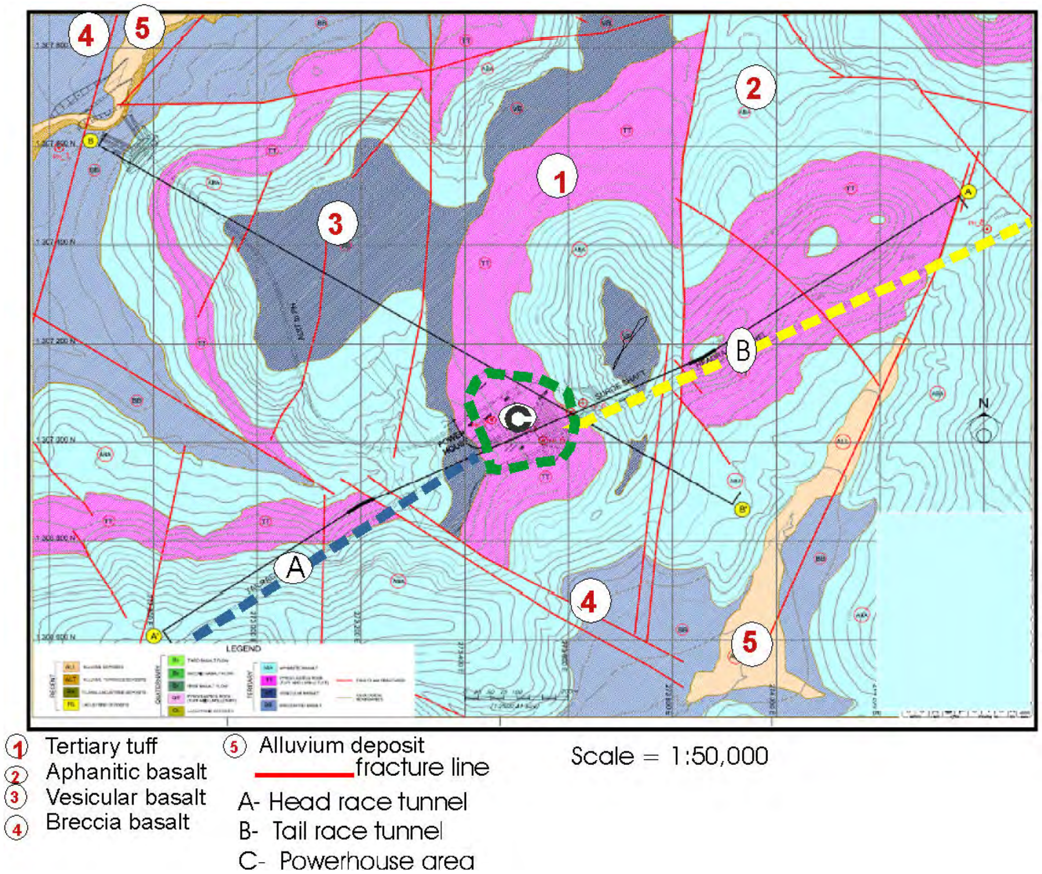


Fig. 2.5 Surface geology of the project site (along the headrace tunnel, powerhouse area and tail race tunnel)

The approach channel of the intake lies above recent lacustrine sediments (RL) and fluvial-lacustrine flood sediments (RA). Such units overlap over 100m thick sedimentary volcanic deposits (QL) which, outcrop at the inlet structure and portal. The entire inlet area is characterized by three successive basalt flows (B1, B2, and B3) preceded by a 10-30m thick tuff layer (QT), due to recent quaternary volcanic activity. While the thickness of B2 and B3 flow varies between 10 and 30m, the thickness of B1 varies according to the paleo-morphology of the underlying Tertiary flows. The inlet area is delimited by Pre-Quaternary N-S and E-W faults (Feasibility investigation report part ‘A’, 2005).

Head race tunnel

The surface geology of head race tunnel can be divided into two main parts:

The first part comprises the inlet area and NE-SW escarpment that intersects the head race tunnel axis and is characterized by Quaternary basalt flows and volcanic lacustrine sediments. The out crops of volcanic lacustrine sediments end in correspondence with a

North-South Pre-Quaternary fault which is visible along the NE-SW valley in proximity of the tunnel at about 2 km from the lake. Volcanic lacustrine sediments lie beneath the Quaternary volcanic rocks which are composed of three basaltic flows and one discontinuity tuff layer. The tuff layer thickness reaches up to maximum of 30m. The Quaternary basalt flows (B1, B2, B3) form the main shield which characterizes the plain leading to the aforementioned escarpment. This area is affected by different fault families:

- Pre-Quaternary N-S and E-W faults intersected and disconnected the tertiary formations. The N-S faults allowed the settlement of the dolerite dyke (DY).
- Quaternary faults cross through the quaternary flow basalts in an approximate NNW-SSE direction.

The Quaternary volcanic rocks are in contact and conformity on the volcano-lacustrine sediment, while the contact with the underlying tertiary formations dipping N $130^0/7\text{-}15^0$ is a discontinuity. The thickness of Quaternary flow basalt varies according to the tertiary paleomorphology.

The second portion, located between the escarpment and the powerhouse, outcrops a tertiary volcanic sequence formed by:

- aphanitic flow basalts with minor tuff bed or paleosoils
- coarse to fine pyroclastic rocks (lapilli tuffs and tuff) with basaltic fragments
- vesicular and scoraceous basalt
- brecciated basalt

Two main tuff layers have been observed alternating with the tertiary lava flows. The top one is 30-80m thick and is fault blocked. The outcrops of such layer are visible from el. 1850-1750m a.s.l., in consequence of the blocks uplift and down lift. The bottom one is 20-30m thick. In this area, the N-S fault trend rotates to a NNE-SSW direction, while the E-W fault trend remains unvaried. These faults dislocate the tertiary rocks with a west-ward down thrown of some 10 m and, occasionally with dextral strike slip component.

Powerhouse area

The exposed rocks in the powerhouse area consist of flow basalts with a dip direction of about N130^0 / $7\text{-}15^0$. The flow basalts consist of aphanitic, vesicular and brecciated textures. Two main tuff layers have been observed alternating with the tertiary lava flows. The above sequences are affected by three main fracture and fault systems, these are; N-S, NNE-SSW and NW-SE systems. The most important structure is located approximately at 1km NE of the powerhouse. It is a NNE-SSW fault zone characterized by cataclastic rock out crops. The west block is the down thrown one. The powerhouse is located in a fault block which is relatively unaffected by geological structures. The powerhouse block is bounded and down faulted for about 80m by two fault systems (Fig. 2.6). The first N-S fault and the second NW-SE fault form the east and the south West sides of the powerhouse block.

Adit to powerhouse

The powerhouse adit axis runs beneath a flat top hill (El. 1760-1770m a.s.l.). The out cropping rocks are;

- a layer of vesicular basalt on the top of the hill.
- aphanitic flow basalts on the hill side alternating with a significant pyroclastic layer (tuff).
- brecciated basalt at the bottom of the hill along the stream.

Tail race tunnel and out let

The surface geology of the 7056m tunnel can be divided into two parts. The main part of the tunnel stretches from the powerhouse area towards the out let for about 6300m. This part mainly runs through the tertiary flow basalts having approximately N120-150^0 dip direction and 10-20^0 dip amounts.

The lithological characteristics of the tertiary flow basalts in this tunnel stretch are very similar to the headrace tunnel and are affected by the same fault systems as discussed in the powerhouse section in the previous paragraph. In comparison to the powerhouse block, the entire tailrace tunnel lies on an uplifted fault block. The last part of the tunnel (about 750m) lies on alluvial terraces with an estimated maximum thickness of 20m overlying the brecciated basalt.

2.4 Geological description of the powerhouse cavity

Generally, the powerhouse cavity and its different accesses consist of volcanic rocks dominantly aphanitic and brecciated basalt units with thin lamination of paleosoils.The powerhouse cavity is fully excavated in the aphanitic basalt and breccia basalt units.

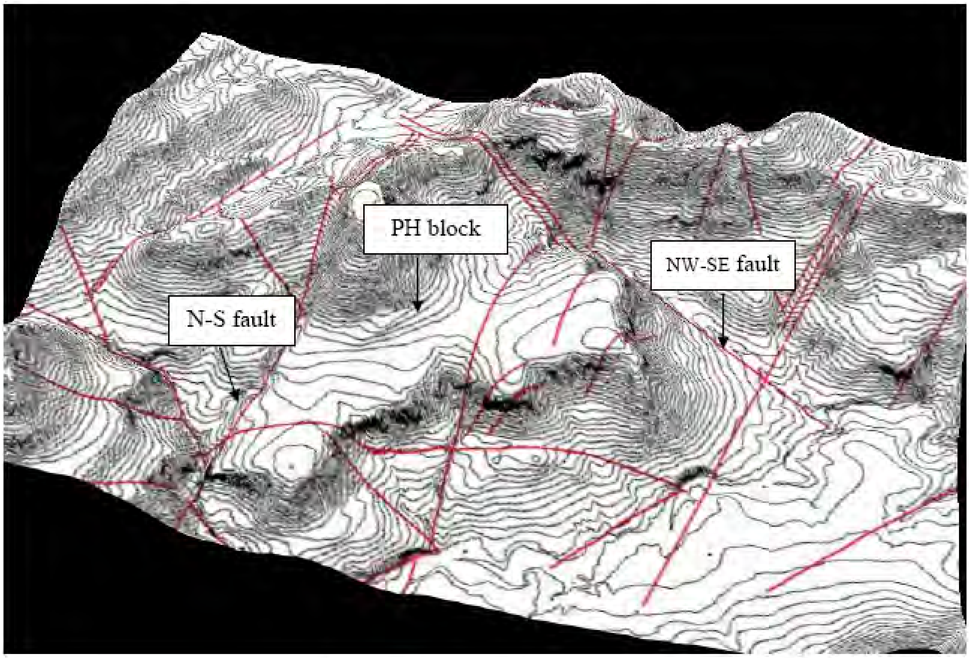


Fig 2.6 structural map of PH area (3D model)
(Source: Feasibility investigation report,2005)

2.5 Structures

2.5.1 Regional structural evolution and tectonics

The area is located close to the edge of Lake Tana depression along the main geological structure named Gondar graven. The structural evolution of the Lake Tana region is described by Chorowicz et al. (1998). The Tana basin is perched on a topographic high located at the junction of 3 gravens: the Dengel Ber (buried by recent lava), Gondar (exposed by erosion) and Debre Tabor (reactivated). This structural complex was active during the build up of the mid tertiary flood basalt pile. Fault slip indicators are consistent with crustal subsidence centered on the present morphologic basin (Lake Tana) which produced fault bounded blocks tilted towards the depression (Fig.2.7).

The tectonics of the area has been described as follows:

On the slope towards Lake Tana, both the basalt complex of the older volcanics and the volcanic lacustrine deposits are found to have horizontal or sub-horizontal attitude that differs from that of the "older volcanics" constituting the western shore of Lake Tana, which dips markedly towards the lake. In the study area, two principal tectonic alignments have been recognized. One alignment comprises a family of faults with a general NE strike that is like that of upper Beles valley; the second family, with a general N-S strike is identifiable only in the out crops of the older volcanics on the north of the area under examination and is almost parallel to the western shore of the lake.

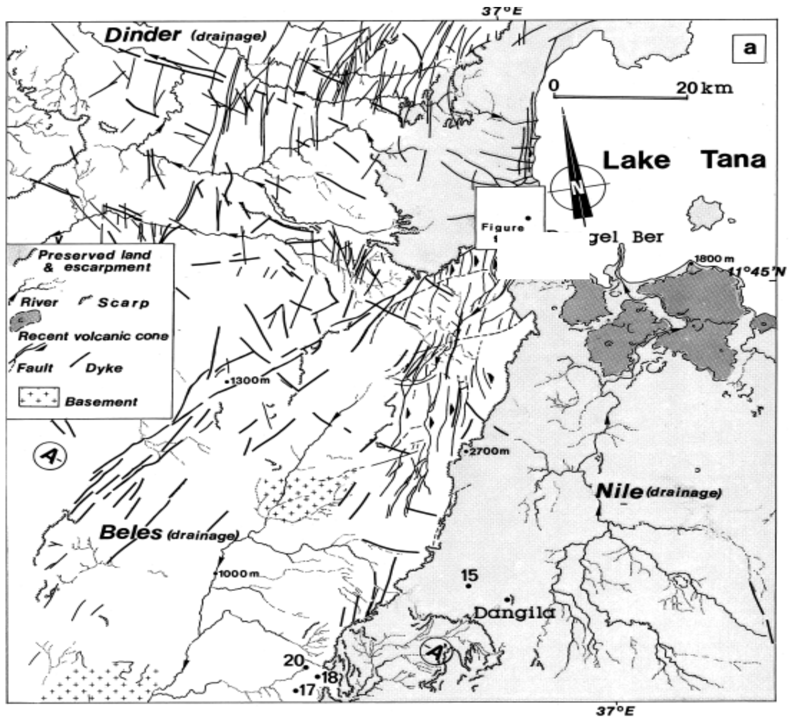


Fig 2.7 Tectonic map of the SW side of Lake Tana basin including upper Beles valley at Scale: 1: 250,000; Source: Chorowicz et al. (1998)

Relatively, subordinate to this, there is a family of faults with NW-SE strike, and which can be identified in the down stream area of the outlet, and which has affected the tuffites of quaternary volcanic- lacustrine complex. Probably the faults of the two principal alignment correspond to an ancient tectonic phases and have affected almost exclusively the bed rock, composed of Miopliocenic basalts and subordinately the successive Volcanic–Lacustrine deposits.

. The attitude of the “Older volcanics”is practically horizontal and cliffs corresponding to extensive basaltic lava flows can be followed with continuity on the slopes of valley for several kilometers. Only a few morphological aspects (example: capture forms in valley incisions, and /or strings of old alluvial terraces at very different elevations on the slopes) may be the evidence of relatively recent tectonic phases. According to Chorowicz et al. (1998) the detected lineaments are not related to the rift system due to its location. Most of them are related to the Lake Tana graven formation and other volcanic processes.

2.5.2 Local structures (Faults)

As discussed in the local geology section, there have been many fault lineaments traced from the intake (Lake Tana), along the head race tunnel, around the powerhouse site and to the down stream portion based on the previous investigations from aerial photo interpretation, geophysical surveys and borehole data analysis. Some of the faults cross the headrace and tailrace tunnels. The powerhouse site has been reported to be free from such major faults. However, one weak structure is identified during this study after successive daily follow up, discontinuity data collection and correlation of the excavation faces.

This fracture zone is a N040^0 trending fault which crosses the power house complex and made the rock mass fractured and disintegrated along its line. The structure is identified to be a normal fault having 310/88^0 orientation. It has developed up to 30 cm wide opening with calcite coated walls on specific locations.

It is really surprising that this structure does not simply touch part of the power house complex; it is aligned crossing many components (adit to penstock, power house cavern, galleries, draft tubes, etc.) Fig. 2.8. This fault is identified to be the main factor for observed instabilities on specific locations during the construction of the cavities.

In general, studies have indicated that probably the NE and N-S strike faults correspond to an ancient tectonic phases and have affected almost all rocks exclusive of bed rock. And this structural complex was active during the build up of the mid flood basalts. While, the NW-SE striking faults have affected the Quaternary volcanic- lacustrine deposits.

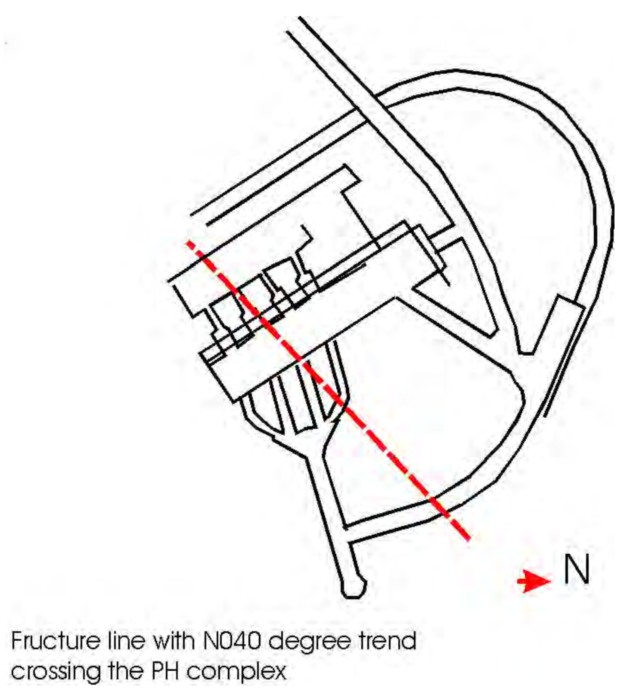


Fig 2.8 Fracture line in Powerhouse cavity

CHAPTER III HYDROGEOLOGY AND SEISMICITY

3.1 Hydrogeology

The flow of ground water into tunnels and caverns has always been a major technical and environmental problem for underground constructions. Water seepage and inflows slow down the construction activity, resulting in an increase of the construction costs (Decenano et al., 2000). Large inflow of water can also endanger the labor force seriously and case histories report loss of human lives (Straskraba, 1998). While this is a very unusual case, typically the working conditions become difficult and are slowed down in the presence of excessive inflows. Ground water inflow is regulated by the parameters: topography, geological and technical variables in the hard crystalline rocks as well as in the over burden. Studies revealed that many factors related to the rock quality, as well as the overburden – such as the number of fractures, the thickness of the overburden, the soil type and the amount of pre-grouting regulate the leakage. The parameters that regulate the minor and major leakage are associated with the drainage of rock mass and the different parameters in the overburden. Unless the rock mass and the overburden are considered as an interrelated system, predictions of ground water inflow are likely to fail.

Determination of ground water condition in the regions of excavations is not to be underestimated at any cost. Ground water is a major governing factor in computation of overhead loads on excavations and also in the choice of method of construction. Ground water is derived from many sources but primarily originates from rain fall. Ground water level is rarely static and it varies with the rate of recharge or discharge of the ground water. Rain fall in a given area rarely have uniform distribution, hence fluctuation in ground water level may vary with geology, topography and proximity to local centers of discharge such as springs, rivers and dams that store water or pump water from the ground.

The infiltrated water can move with ease in soils, rocks and typical fractures of rocks. The extent to which infiltration from rain fall affects underground excavations dependents on a number of factors, such as the original position of ground water table, intensity and duration of the rain fall, the antecedent rain fall within the ground water catchments, the recharge potential of the area, the geology, the degree of saturation and the topography.

Ground water can affect the stability of rock mass in five ways (Abramson et al., 1995);

- It reduces the strength of the rock mass
- It may change the mineral constituents through chemical alteration and solution.

- It may change the bulk density of the rock.
- It may generate pore water pressure.
- It may cause erosion.

According to Garg (2003), there are three general possibilities of relationship between the excavation axis and ground water condition;

(i) The excavation axis may be passing entirely through impervious formations. Such a situation is usually not encountered in natural conditions for long and wide excavations.

(ii) The excavation axis may be located mostly above the water table. Intercepting the aquifer only in some sections. This is one of the commonest situations and involves provisions for special drainage facilities to be located in water bearing zones. The head of water in the zone of intersection has also to be given due consideration, and may necessitate lining for stopping leakage or inrush water.

(iii) Excavation may be located below the water table, which occurs rarely and is the most difficult one.

3.1.1 Powerhouse Cavity Hydrogeology

Site hydrogeology is related to the superficial circulation of ground water of meteoric origin. At the Tana Beles project site there has been a few numbers of local springs that come in to existence during rainy seasons related to percolations along fractures. These springs are usually found on top of tuff layers, which act as an impermeable level and oblige the water to accumulate and emerge along fractures. The water then flows on the tuff following a maximum slope (Feasibility Report part A, 2005).

During the feasibility stage of investigation, the permeability of the site was checked using pressure tests in boreholes and rocks were found to be watertight. Furthermore, during the construction stage there was no groundwater problem that interrupted the construction activity. There were no flowing springs identified on the surface during dry periods; only there has been a few numbers of local springs that come into existence during rainy seasons (Feasibility Report part ‘A’, 2005).

There is no confined water in the cavity. The joints are mostly closely spaced, the rock units are not that much fractured or weathered, and in addition the topography of selected area for powerhouse location does facilitate fast surface drainage of rain water to the surrounding dissected valleys at all sides. It is a sloping hill; hence water easily drains on surface, and does not get sufficient time to infiltrate to the sub surface unless major fractures are extended to the surface to facilitate this. However, this observation is made during the driest month (the present fieldwork was conducted during dry months) therefore there will be possibilities of concentrated seepage and leakage at some locations during the rainy season.

Generally, the hydro geological condition of the site is good that there was no major ground water problem during this huge amount of excavation for two years. The existence of ground water in the powerhouse complex excavation was manifested on excavation faces of only specific locations. The wet and damp features are mainly manifested on powerhouse cavern upstream and on down stream walls, between galleries 2 and 3. Also, traces of water have been noticed around draft tubes, all following the trend of the identified fracture line.
Moreover, during the present investigations at some specific locations wet bands (damp surfaces) were observed. Groundwater is also observed dripping at very slow rate on fresh excavations at few locations.

Along the identified N 040^0 tending fracture line, there was concentrated water, which gushed out with pressure during the opening of the access adit to the penstock bottom. In addition, the water that has been used for drilling and construction purposes; and which was ponded on the excavation floors around the bottom of ventilation shaft was manifested to gush out with pressure for few minutes when the draft tube junction 1 & 2 was excavated below that level. It was also observed that the water accumulated in the powerhouse cavern floor was seeping towards the manifolds, which have lower level of excavation. All these manifestations indicate the presence of some major joints that allow water to circulate through.

In the permanent excavations, the ground water chemistry may have corrosive effect on concrete and steel supports. However, during the present investigation ground water samples were not collected to check the effect of its chemistry on support as there was no flowing water to be sampled during the time of data collection.

Generally the hydro geological condition does not seam to bring major construction and failure problem. But there needs the provision of drainage pipes and if possible grouting of the identified fracture line. Because the water that seeps through any weak access can accumulate to this zone and other possible locations producing large pressure on surface supports which may result to failure.

In addition, water can act as the main agents of chemical weathering and decomposition to take place through out the life of the project (50 years) contributing for lose of the structural strength through time.

3.2 Seismicity

Earthquake, resulting Seismicity (dynamic loading) is one of the major natural hazards known for its destructive records on natural and artificial features on landscapes in many different parts of the world since long. Natural hazard is defined as "the probability of failure occurrence within a specified period of time, within a given area of potentially damaging phenomena "(Johnson et al, 1988). The damage can be manifested as ground failure in the form of landslides, ground cracking, subsidence and differential settlement. Natural hazard analysis is part of the project study to determine whether the natural risk is at acceptable range or not.

Determination of natural hazard requires a statement of probability of occurrence. The prediction of future events of a given magnitude usually depends on the basic geologic and seismic studies. The usually relate present and past conditions in effect to some empirical relationship, which may serve to predict future events based on existing information (Johnson et al,1988).

When large engineering structures are to be constructed in seismic risk areas, the peak earthquake ground acceleration will be a useful variable to be considered in the evaluation of the response of the structure to the earthquake motions.

3.2.1 Seismicity of the Project Area

Gouin (1979) delineated the seismic zone of Ethiopia, which was latter modified by Laikemariam Asfaw (1979). According to Pierre Gouin (1976), the distribution of the earth quake, known or recorded, starting from 1400 shows that only the faults of the rift system influence the Seismicity of the Lake Tana region. It derives from the fact that the Tana area runs the moderate 100-year seismic risk of 6^{th} degree intensity in the modified Mercalli (MM) scale. The 300-year of the 7^{th} MM degree, corresponding to acceleration comprised between 0.05 g at Gondar and 0.06 g at Debremarkos. In general, one cannot exclude the local structural conditions are not seismogenetic, because of the neotectonic evidences.

According to the work of Likemaryam Asfaw (1979), strain release and seismic risk maps has been produced for earthquakes from year 1400 to 1985 and 1900 to 1985 and the probable return period of destructive earthquakes has been considered.

The seismic risk map produced by Laikemaria Asfaw for a hundred years return period and 0.99 probability shows that the project area falls within Zone 5 M.M. intensity scale. The map showing the seismic risk zones of Ethiopia and the location of the project area is shown in Fig. 3.1.

Based on the chart presented by Johnson et al., (1988), the ground acceleration ranges from 0.03 to 0.06 g for M.M. intensity level 5.

As already mentioned in chapter I, underground structures are less susceptible to seismic effects as compared to surface structures. In addition, there have never been considerable seismic effects recorded around the project area. Therefore, the project is most probably safe from the effect of such natural hazards. However, safe seismic factor is good to be considered in the design of any underground structures and supports in that area.

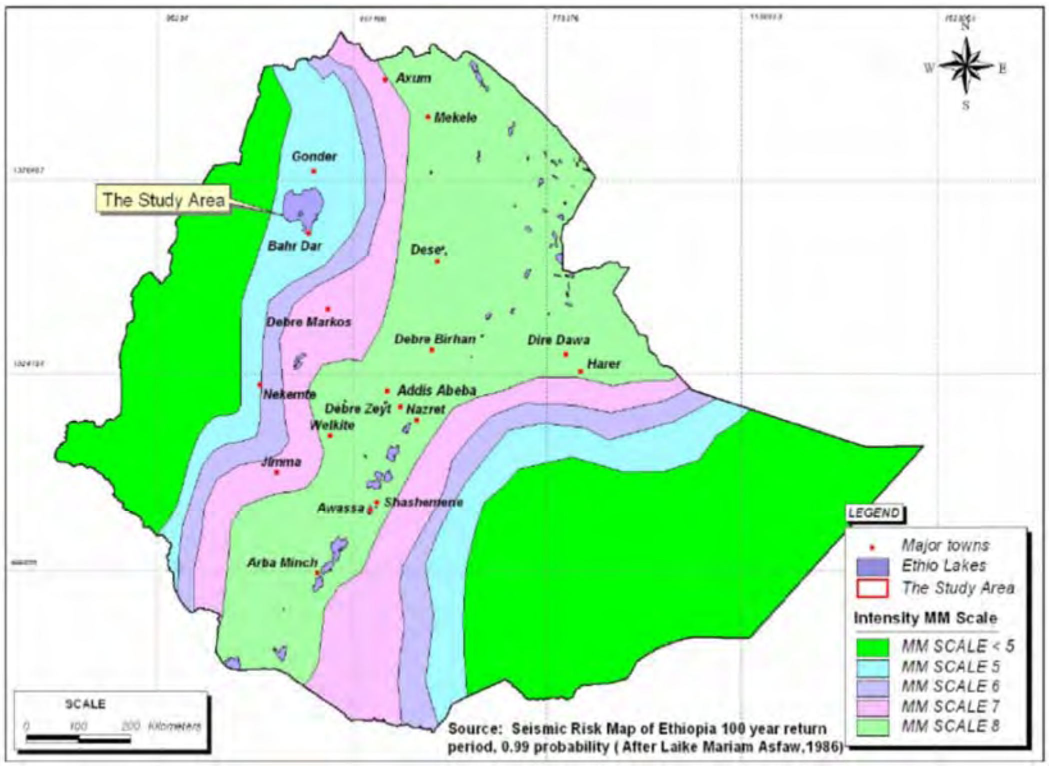


Fig. 3.1 Seismic risk map of Ethiopia 100 years return period, 0.99 probabilities by Laike Mariam Asfaw, (1986)

CHAPTER IV REVIEW OF PREVIOUS SITE INVESTIGATIONS

4.0 Preamble

Reviewing documents is an important primary component of any study to have clear idea about the project, to acquire the basic understanding of the problems and to choose the appropriate methodology in solving the problems. It also helps to know the scope of the study and limitations which are to be faced by a researcher. Reviewing of the engineering geological investigation enables to have a general overview about the engineering geological, geotechnical and hydro geological properties of rock materials. Moreover, it also helps to think and formulate the design, construction and support requirements for an underground structure. For the present study, an attempt is made to review the pre-feasibility and feasibility investigations in detail. For this review more emphasis was given to the feasibility investigations of the powerhouse area, as the present study focus on the rock characterization for the suitability of the powerhouse cavity.

After the proposal of the Tana Beles hydropower project, geological and geotechnical investigations were carried out by different groups at different stages for the feasibility of the project. The investigations were carried out to characterize the rock mass for the suitability of the powerhouse construction and to workout the necessary support measures for the excavated cavity. The main investigations were geophysical survey, borehole drilling, water pressure tests, in situ and laboratory tests. The review of these investigations is summarized in the following paragraphs;

4.1 Geophysical investigations

Geophysical surveys are always conducted on project sites since they have wide application, simple to implement and are not expensive. In order to explore the powerhouse cavity of the Tana Beles project site, geophysical investigations involving Refraction Seismic survey, Vertical electrical sounding (VES) and electrical well logging have been carried out.

4.1.1 Surveys Findings

The integrated geophysical surveys carried out on selected stations and different survey lines have identified the subsurface conditions interims of layer resistivity and compress-ional wave velocities. A generalized table of refraction wave velocities and layer resistivities which have been used for the construction and interpretation of paleoelectrical, peleoseismic sections is presented through Table 4.1. This table is prepared on the basis of the available geo-log of wells (PH4 & PH5) along the surveyed lines.

Table 4.1 Generalized table of compressional wave velocities and layer resistivities with the Lithologic units of the powerhouse area.

No	Rock type	Range of resistivity (ohmmeter)	Range of velocities (cm/s)
1	Colluvium highly weathered tuff	75-165	<1000
2	Highly weathered tuff with little colluvium	4-20	
3	Weathered brecciated tuff	450-750	500-1500
4	Highly to moderately weathered volcanic ash/tuff	8-85	1000-2000
5	Moderately weathered fine tuff, low breccia	650-950	1500-2500
6	Slightly to moderately weathered tuff	330-664	2000-3000
7	Slightly fractured fresh tuff	>200	2500-3500
8	Moderately to slightly weathered basalt	100-300	2500-3500
9	Fresh basalt	>300	>4500

(Source: Beles Geophysical investigation report, 2006).

The integrated analysis of resistivity and seismic refraction investigations carried out on different survey lines (10 survey lines) shows the increase of compactness of the rock units with depth with some lateral variations.

The report summarizes that from engineering point of view, units with very high resistivity (> 120m ohm-m) and greater than 4000 cm/s may generally be considered as stable, water tight and high bearing capacity , RQD > 80%. According to the bed rock velocity map powerhouse may be classified as:

A: 3500 – 4500 m/s very good foundation
B: 3000 – 3500 m/s good foundation
C: 2500 – 3500 m/s weak to good foundation
D: 1500 – 2500 m/s poor (unfavorable)

However, as it is tired to check from the study report, the depth of geophysical investigations are above 1720m (shallow level) and hence, can not be used for the comparison and analysis purpose of the powerhouse cavity and its components which are located below 1484 m a.s.l.

In addition, the investigations have identified some discontinuities (probably contacts) and also lineaments inferred from the combined refraction seismic and resistivity interpretations trending in the W-E and SSW- NNE directions. So, this is extra information to confirm the weak structure identified during field data collection in the powerhouse area.

4.2 Borehole drilling

Drilling is the most important and expensive method of investigation, which is done to know the subsurface geological and hydro-geological conditions at a project site. The exploratory drilling at the project site is conducted mainly; (i) to establish correlation of lithounits, (ii) to evaluate the physical conditions of rocks such as rock quality designation, (iii) to collect samples of various rock units for laboratory tests and (iv) to carry out water pressure tests.

In order to investigate the subsurface engineering geological and hydro geological condition from core samples, observations and then to conduct water pressure tests a total number of 8 boreholes have been drilled close to the powerhouse and surge shaft. All the drillings are in vertical position and their depths range from 42.5m to 356.22m from their respective ground positions (Table 4.2). The surface locations of some boreholes are shown in Fig. 4.1.

The boreholes are logged and analyzed with the associated descriptions of litho logical units, total core recovery (TCR), strength estimation (UCS), fracture frequency, rock quality designation (RQD). The details of lithological descriptions and engineering geological parameters are discussed in the following sub-sections. (Tana Beles feasibility reports, 2005).

Table 4.2 Location and depth of boreholes in powerhouse location

Name	Location	Top elevation (m)	UTM		Depth (m)
			Easting	Northing	
PH-1	Some 80m right of PH adit portal	1626	273619	1307598	200.58
PH-2	Some 200m left of PH adit portal	1633	273023	1307941	200.08
PH-3	Left side of LPHT portal (on the power house room from top surface)	1752.5	273495	1307033	328.10
PH-4	On the penstock shaft (from top surface)	1806	273604	1307061	356.22
PH-5	On the LPHT portal (on the first or second bench)	1762.5	273551	1307004	42.50
PH-6	On the top of surge shaft	1829	273628	1307080	105.07
PH-7	On top of cable shaft (near to PH-3)	1746	273452	1307045	307
PH-8	On the head race tunnel (back of surge shaft)	1822	273406	1307432	102.88

(Source: Feasibility report of Tana Beles project part “A”, 2005)

4.2.1 Lithological description of borehole logs in powerhouse location

In order to know the lithology in the powerhouse cavity and the surge shaft area geo-logs of boreholes PH 3, PH 4, PH 5, PH 6 and PH 7 were reviewed. The lithological description as reviewed from the core recovery from the boreholes in the powerhouse and surge area is presented in Tables 4.3 to 4.7.

4.2.2 Engineering geological properties of logged samples

During the feasibility investigation, drill-cores were the only source of information for engineering geological condition of the rock mass at powerhouse cavity. Cores have been logged and preserved by the project authorities. From these cores the intact rock and rock mass properties have been determined through visual observations and laboratory tests.

In this sub-section attempt is made to review the rock mass properties present in the powerhouse cavity. For this purpose fracture index, rock quality designation (RQD), unit weight and uniaxial compressive strength (UCS) has been analyzed. The descriptions of the parameters are presented in the Tables 4.8 to 4.14 and the geological cross section between the three boreholes logs is shown in fig. 4.2.

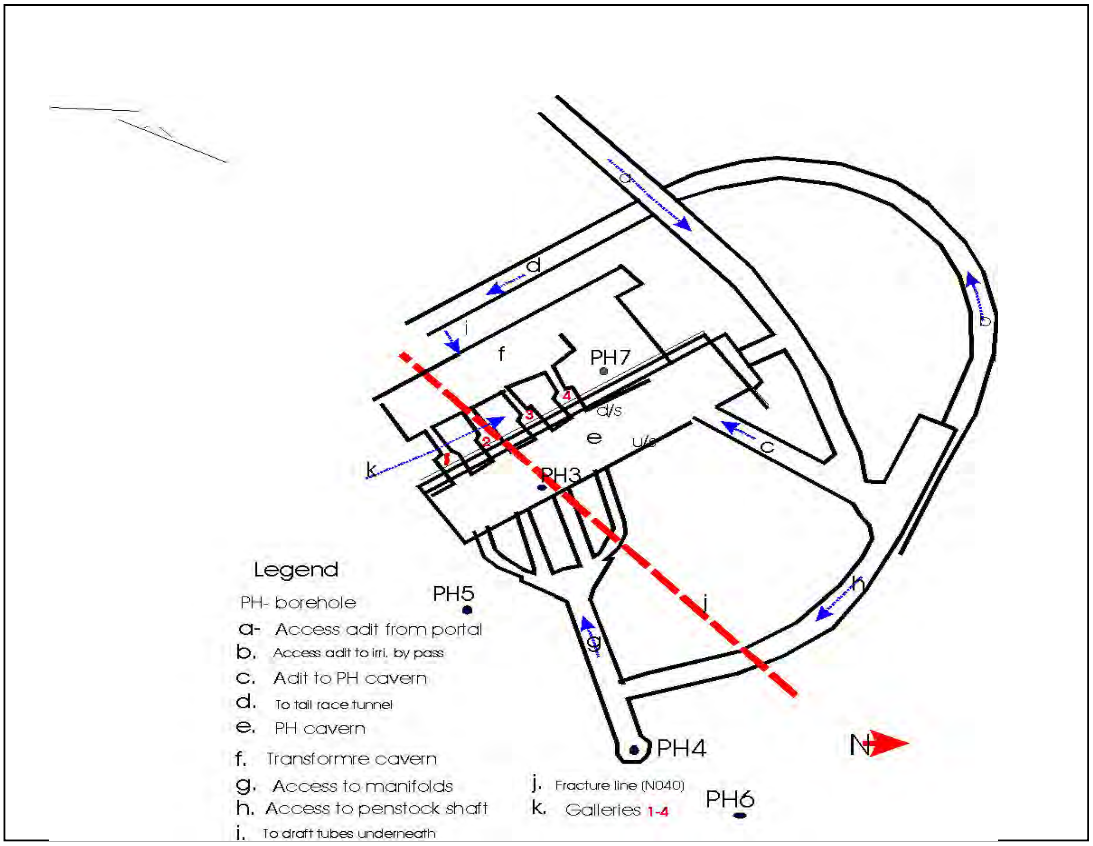


Fig. 4.1 Surface locations of boreholes above powerhouse

From the review of Tables 4.8 to 4.12 it can be concluded that the two rock units (aphanitic basalt ‘ABA’ and brecciated basalt ’BB’) behave differently for RQD and UCS parameters. Mostly the aphanitic basalt is found to have lower RQD as compared to brecciated basalt due to fracturing and jointing effects. On the other hand, the strength of the aphanitic basalt is much higher than the brecciated basalt due to the difference on intact rock properties of the two rock units.

Table 4.3 Lithological description from Borehole PH 3

(Location; E: 273495, N: 1307033, Z: 1752.5m)

Depth(m)	Litho logical description
0-5	Colluvial material: inorganic silt and sand with sand fragments
5-7.48	Colluvial material: clay sand with tuff fragments
7.48-8.95	Tuff fragments
8.95-9.46	Highly weathered tuff with layered structure
9.46-25.30	Greenish tuff with layered structure
25.30-26.40	Pyroclastic layer: fine ash (base surge?)
26.40-35.00	Vesicular basalt
35.00-53.00	Fine grained aphanitic basalt
53.00-200.34	No recovery
200.34-200.7	Fresh massive basalt
200.7-201.5	Brecciated basalt
201.5-202.85	Soft friable, highly weathered basalt
202.85-208.3	Brecciated basalt
208.3-215.84	Dull black or black grey, massive hard basalt, highly jointed (213.5-215.84)
215.84-216.82	Highly weathered basalt
216.82-217.12	Paleosoil
217.12-219.2	Brecciated basalt
219.2-221.10	Vesicular basalt
221.10-222.44	Brecciated basalt
222.44-223.7	Vesicular basalt
223.7-224.04	Brecciated basalt
224.04-225.09	Highly weathered basalt
222.09-229.15	Brecciated basalt
229.15-240.7	Dull black or black fresh, massive hard aphanitic basalt
240.7-244.68	Slightly to moderately weathered basalt
244.68-250.61	Dull black or black fresh, massive hard aphanitic basalt
250.61-251.12	Highly weathered material with relicts of vesicular basalt
251.12-253.52	Brecciated basalt
253.52-254	Dull black or black fresh, massive hard aphanitic basalt
254-254.28	Brecciated basalt
254.28-259.4	Dull black or black fresh, massive hard aphanitic basalt
259.4-267.30	Brecciated basalt
267.30-273.30	Dull black or black grey, massive hard basalt, highly jointed basalt
273.30-279.77	Brecciated basalt
279.77-284.83	Dull black or black grey, massive hard basalt, highly jointed basalt
284.83-304.36	Brecciated basalt
304.36-314.20	Dull black or black fresh, massive hard aphanitic basalt
314.20-324.59	Brecciated basalt
324.59-328.10	Dull black or black fresh, massive hard aphanitic basalt

Table 4.4 Lithological description from Borehole PH 4 (Penstock Shaft)

(Location; E: 273604, N: 1 307 061, Z: 1806m, Inclination Vertical)

Depth (m)	Litho logical description
0—3.55	Light colored brecciated tuff
3.55--8.07	Light colored moderately weathered tuff
8.07--25.02	Light gray colored, moderately weathered lapilli tuff
25.02—32.00	Light colored, slightly weathered brecciated tuff
32.00—40.72	Dark gray colored, moderately weathered tuff
40.72—47.30	altered zone, highly weathered friable tuff

Table 4.4Cont….

Depth (m)	Litho logical description	*Table 4.4 Cont….*
47.30—53.00	slightly to moderately weathered acidic tuff (brecciated)	
53.00—66.50	slightly weathered to fresh gray colored tuff	
66.50—76.10	light gray to white, slightly weathered brecciated tuff	
76.10—82.00	light greenish to gray, moderately weathered tuff	
82.00—90.60	dark gray, fresh vesicular basalt	
90.60—148.20	Dull black, dark gray, fresh massive aphanitic basalt	
148.20—150.46	Vesicular basalt with some vesicles filled with calcite	
150.46—155.10	Light to dark gray lapilli tuff	
155.10—155.20	Paleosoil	
155.20—169.40	Light gray to light green brecciated tuff	
169.40—171.80	Altered zone, highly weathered friable tuff	
171.80—175.20	Brecciated basalt	
175.20—177.20	weak zone	
177.20—179.10	Fine grained basalt	
179.10—181.75	Amygdaloidal vesicular basalt	
181.75—183.00	Fine grained basalt	
183.00—183.90	Weak zone, paleosoil	
183.90—184.22	Dark gray, reddish brecciated basalt	
184.22—189.00	Dark gray, vesicular basalt	
189.00—192.00	weak zone, weathered pyroclastic layer	
192.00—195.00	Dark gray, vesicular basalt	
195.00—196.45	Aphanitic basalt	
196.45—197.30	weak zone, paleosoil	
197.30—200.75	Dark grey, vesicular basalt	
200.75—201.00	Brecciated basalt	
201.00—202.50	Dark grey vesicular basalt	
202.50—202.87	Pyroclastic layer, tuff	
202.87—209.30	Dark grey vesicular basalt	
209.30—210.20	Brecciated basalt	
210.20—217.80	Dark grey vesiculated basalt	
217.80—220.00	Pyroclastic layer, tuff	
220.00—221.06	paleosoil	
221.06—225.30	pyroclastic layer, tuff	
225.30—225.58	paleosoil	
225.58—226.10	Moderately weathered basalt	
226.10—227.30	Dark grey, vesicular basalt	
227.30—247.30	Brecciated basalt	
247.30—253.97	Fine grained basalt	
253.97—254.15	Brecciated basalt	
254.15—255.20	paleosoil	
255.20—260.10	Brecciated basalt	
260.10—265.57	Fine grained basalt	
265.57—265.80	Brecciated basalt	
265.80—266.82	Dark brown clay, paleosoil	
266.82—273.87	Dark grey vesicular basalt	
273.87—276.90	Brecciated basalt	
276.90—281.05	Aphanitic basalt	
281.05—281.40	vesicular basalt	
281.40—283.70	Paleosoil	
283.70—291.04	Brecciated basalt	
291.04—303.70	Aphanitic basalt	
303.70—304.10	paleosoil	
304.10—305.60	Brecciated basalt	
305.60—310.70	Fine grained basalt	
310.70—311.20	Brecciated basalt	
311.20—311.30	paleosoil	
311.30—316.20	Brecciated basalt	
316.20—328.10	Aphanitic basalt	
328.10—331.00	Brecciated basalt	
331.00—344.00	Aphanitic basalt	
344.00—356.20	Brecciated basalt	

Table 4.5 Lithological description from Borehole PH 5 (Penstock Shaft)

(Location; E: 273355, N: 1 307 003, Z: 1762 m, Inclination Vertical)

Depth (m)	Litho logical description
0.00—0.60	Colluvium material mixed with dark brown soil
0.60—1.15	Light colored, loose fine material, acidic volcanic ash, highly weathered tuff
1.15—1.40	Light yellowish, fragmented loose weathered tuff
1.40—1.91	Light colored acidic volcanic ejecta compacted (weathered tuff breccia)
1.91—3.51	Light yellowish, fragmented loose weathered tuff
3.51—4.06	Light gray, fragments of rock composed of ash (tuff)
4.06—20.00	Light gray, brecciate welded tuff composed of volcanic ash, rock fragments, bombs, slightly to moderately weathered
20.00—28.60	Light greenish gray to white brecciated weathered tuff
28.60—30.70	Black to light gray, slightly to moderately weathered welded tuff
30.70—35.35	Light gray to white, brrecciated tuff composed of volcanic ash dominant and minor breccia
35.35—42.50	Dark gray, slightly weathered to fresh brrecciated tuff

Table 4.6 Lithological description from Borehole PH 6

(Location; E: 273628, N: 1 307 080, Z: 1829 m, Inclination Vertical)

Depth (m)	**Litho logical description**
0.00—1.38	Dark gray, moderately weathered vesicular basalt
1.38—19.03	Dark black, dark gray, fresh, massive hard aphanitic basalt
19.03—20.80	Dark gray, moderately weathered basalt
20.80—20.94	Paleosoil
20.94—21.54	Dark gray, moderately weathered vesicular basalt
21.54—21.85	Paleosoil
21.85—24.00	Dark gray, moderately weathered vesicular basalt
24.00—24.53	Highly weathered tuff
24.53—26.70	Light gray, slightly weathered fine tuff
26.70—27.35	Light gray, slightly weathered coarse grained tuff
27.35—27.80	Light gray, slightly weathered fine grained tuff
27.80—28.00	Light gray, slightly weathered coarse grained tuff
28.00—29.56	Light gray, slightly weathered fine grained tuff
29.56—29.84	Light gray, slightly weathered brecciated tuff
29.84—31.90	Light gray, slightly weathered coarse grained tuff
31.90—32.90	Light gray slightly weathered fine tuff
32.90—33.50	Light green slightly weathered fine tuff
33.50—34.00	Light gray, slightly weathered brecciated tuff
34.00—48.00	Light gray, slightly weathered brecciated tuff with rock rock fragments
48.00—48.20	Light gray, slightly weathered brrecciated tuff
48.20—49.30	Light gray, slightly weathered fine tuff
49.30—51.85	Light gray, slightly weathered fine to coarse tuff
51.85—62.00	White greenish, slightly weathered coarse tuff
62.00—65.60	Gray greenish, moderately weathered coarse tuff intercalated with weak zone
65.60—65.80	Gray greenish, highly weathered tuff
65.80—66.40	Gray greenish, moderately weathered layered fine / coarse tuff
66.40—67.30	Light gray, moderately weathered fine to coarse tuff
67.30—87.80	Light gray, slightly weathered brrecciated tuff
87.80—92.40	Light to whitish, slightly weathered coarse tuff

92.40—102.40	Light gray greenish, slightly weathered brrecciated tuff
102.40—105.07	Gray, slightly weathered coarse tuff

Table 4.7 Lithological description from Borehole PH 7 (ventilation and cable shaft)

(Location; E: 273452, N: 1 307 045, Z: 1746 m, Inclination Vertical)

Depth (m)	Litho logical description
0.00—9.69	Whitish gray, highly weathered, fractured and moderately hard welded fine tuff
9.69—13.25	Dark gray greenish, moderately weathered, weak fine to coarse
13.25—14.93	Dark gray, vesiculated basalt
14.93—15.34	Weak tuff layer
15.34—19.32	Dark gray, vesiculated basalt
19.32—83.77	Dark gray, moderately weathered, fractured, hard aphanic basalt
83.77—84.85	Dark gray, vesiculated basalt
48.77—106.24	Greenish gray and whitish, slightly weathered, moderately fractured, medium hard coarse tuff (weak layer from 86 to 86.17)
106.24—107.18	Paleosoil
107.18—120.40	Dark gray, slightly weathered, medium hard brrecciated basalt
120.40—127.64	Dark gray, slightly weathered, moderately fractured, fine grained to aphanitic basalt
127.64—131.37	Light gray, moderately weathered, coarse-grained weak tuff
131.37—134.34	Dark gray, fresh and hard vesicular basalt
134.34—135.20	Weak tuff layer
135.20—144.24	Dark gray, fresh and hard vesicular basalt
144.24—148.85	dark reddish gray, moderately to highly weathered brecciated basalt
148.85—150.31	Reddish brown layer, paleosoil
150.34—164.26	Dark gray, coarse grained, moderately fractured, medium hard, welded tuff (very weak from 155.70 to 164.26)
164.26—178.06	Dark gray, slightly weathered brrecciated basalt
178.06—192.70	Dark gray, fine grained, highly fractured, fresh and hard basalt
192.70—195.17	White to reddish gray, highly weathered coarse grained weak tuff
195.17—203.6	Dark to reddish gray, slightly weathered, medium hard brecciated basalt
203.17—210.60	Dark gray, fine grained, highly fractured, fresh and hard
210.60—216.37	Dark to reddish gray, slightly weathered, medium hard brecciated basalt
216.37—217.83	Highly weathered tuff
217.37—224.80	Dark to reddish gray, slightly weathered, medium hard breciated basalt
224.80—233.17	Dark gray fresh, fractured, and aphanitic basalt, it is highly fractured at the interval 224.80-226.05
233.17—243.29	Dark reddish gray, slightly weathered, medium hard brecciated basalt
243.29—243.72	Highly weathered weak tuff
243.72—245.94	Dark to reddish gray, slightly weathered, medium hard brecciated basalt
245.94—251.74	Dark gray, fresh, fractured, hard aphanitic basalt
251.74—252.00	Dark to reddish gray, slightly weathered medium hard brecciated basalt
252.00—256.77	Dark to reddish gray, slightly weathered medium hard brecciated basalt
256.77—266.29	Dark gray, fresh, fractured, hard aphanitic basalt
266.29—272.40	Dark to reddish gray, slightly weathered medium hard basalt
272.40—280.03	Dark gray, fresh, fractured, hard aphanitic basalt
280.03—292.10	Dark to reddish gray, slightly weathered medium hard brecciated basalt
292.10—307.00	Dark gray, fresh, fractured, hard aphanitic basalt

Further, the two parameters, RQD and UCS, will not have similar nature for the same rock unit. In addition, the low RQD and UCS values are not related to the same elevation units on the three boreholes indicating that the effects are local or specific or may be by lack horizontal bedding. The strength of the breccia basalt is very low to low. The possibility of having poor core sampling may also be one factor for the low value of RQD at some locations where the fracturing index is low and the rock is less weathered. Table 4.13

demonstrates the percentage values of the rock quality and strength parameters as concluded from the borehole data and table 4.14 summarizes the range and average values of these parameters.

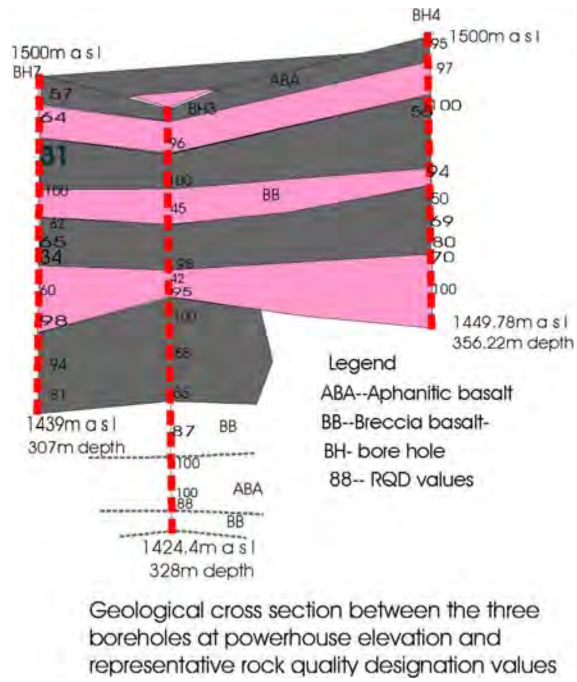


Fig. 4.2 Geological cross section

Table 4.8 Engineering rock parameters as determined from core samples of Borehole PH3

Depth (m)	Rock unit ***	RQD (%)	Fracture index	Quality Based on RQD
251-252.52	BB	77-78	4	Good
252.52-254	ABA	95-97	3-5	Excellent
254-254.28	BB	96-100	2-3	Excellent
254.28-259.4	ABA	95-100	2-3	Excellent
259.4-263.30	BB	100	3	Excellent
263.30-266.30	BB	88	4	Good
266.30-268	BB+ABA	100	3	Excellent
268-270.60	ABA	45-49	5-7	Poor **
270.6-272.2	ABA	76	8	Good
273.30-279.77	BB	90-100	3-4	Excellent
279.77-281	ABA	96-100	4	Excellent
281-283	ABA	42	9	Poor **
283-298	BB	95-100	2-3	Excellent
298-299.2	BB	80	4	Good
299.2-303	BB	97-100	0	Excellent
303-305.4	BB+ABA	53	4	Fair
305.4-306.2	ABA	61	6	Fair
306.2-308.3	ABA	86-100	0-3	Good-Excel
308.3-313	ABA	76-87	5-7	Good
313-315	ABA+BB	66	8	Fair
315-317.3	BB	100	8	Excellent
317.3-318	BB	76	2	Good
318-323	BB	100	2-4	Excellent

323-325	BB+ABA	88	3	Good
325.2-328	ABA	76	3	Good
** High jointing *** ABA - Aphanitic basalt and BB - Brecciated basalt				

Table 4.9 Engineering rock parameters as determined from core samples of Borehole PH3

Depth (m)	Rock unit *	Schmidt Hammer value	Unit weight (γ) (kg/m^3)	UCS (Mpa)	Strength based on UCS
256.6	ABA	48	28	152	High
260.4	BB	20	23	24	Very low
264.7	BB	36	21	50	Low
268.1	ABA	50	28	162	High
274.4	BB	40	23	64	Medium
279.6	BB	20	23	26	Very low
283.7	ABA	48	28	152	High
286.4	BB	32	23	33	Low
302.7	BB	28	23	39	Low
308	ABA	44	28	121	High
315.6	BB	14	23	19	Very low
319.5	BB	24	26	36	Low
322.1	BB	25	23	32	Low
326.5	ABA	34	26	57	Medium
* ABA - Aphanitic basalt and BB - Brecciated basalt					

Table 4.10 Engineering rock parameters as determined from core samples of Borehole PH4

Depth (m)	Rock unit ***	RQD (%)	Fracture index	Quality based on RQD
305.6-310.7	ABA	92-100	0-4	Excellent
310.7-316.2	BB+P.S	97-100	0-2	Excellent
316-317.2	ABA	100	0	Excellent
317.2-319.5	ABA *	55	2	Fair
319.5-322	ABA	78	5	Good
322-323	ABA	88	4	Good
323-325.7	ABA	76	6	Good
325.7-332	BB	92-100	0-3	Excellent
332-333.6	ABA	72	5	Fair
333.6-335.2	ABA *	50	8	Poor
335.2-339.5	ABA	69-70	5	Good
339.5-341	ABA *	84	1	Good
341-342.2	ABA *	36	4	Poor
342.2-343.7	ABA	80	9	Good
343.7-346	BB *	70	0	Fair
346-356	BB	100	0	Excellent
* Improper sampling or wrong fracture index ** Schmidt value, unit weight and UCS are not available on this borehole logs. *** ABA - Aphanitic basalt and BB - Brecciated basalt				

Table 4.11 Engineering rock parameters as determined from core samples of Borehole PH7

Depth (m)	Rock type ***	RQD (%)	Fracture index	Rock quality
247-248	ABA (fractured)	76-79	5-6	Good
248-251.74	ABA (fractured)*	57	8	Fair
251.74-254	BB	64	7	Fair

254-256.77	BB	90-100	1-3	Excellent
256.77-260.2	ABA	89-100	4-5	Excellent
260.2-261	ABA *	31	8	Poor
261-266.29	ABA	78-90	3-5	Good
266.29-271.5	BB	93-100	2-4	Excellent
271.5-274.2	BB+ABA	62-65	7-8	Fair
274.2-280	ABA	89-91	4-8	Good
280-285	BB	82-100	2-4	Good-exe
285-286	BB	60	10	Fair
286-291.4	BB	92-98	4	excellent
291.4-293	BB+ABA	67	7	Fair
293-301	ABA	88-94	4-6	Good-exe
301-302	ABA	71	11	Fair
302-307	ABA	81	7	Good
Quality class according to Deer, 1968 *** ABA - Aphanitic basalt and BB - Brecciated basalt				

Table 4.12 Engineering rock parameters as determined from core samples of Borehole PH7

Depth (m)	**Rock unit *****	**Schmidt value**	**Unit Weight 'γ' (kg/m^3)**	**UCS (Mpa)**	**Strength based on UCS**
247.8	ABA	35	26	63	Medium
255.5	BB	20	23	28	Low
257.5	ABA	55	26	171	High
261.5	ABA	55	26	171	High
267.3	BB	45	26	166	High
270	BB	17	23	22	Very low
272.5	BB	28	26	40	Low
278.5	ABA	50	26	138	High
281	BB	22	23	28	Low
284.5	BB	14	23	19	Very low
288.2	BB	25	23	32	Low
292.2	ABA	50	26	138	High
296.2	ABA	50	26	321	Very high
302.6	ABA	43	26	100	Medium
306.5	ABA		26	43	Low

Table 4.13 Percentage values of RQD and UCS as concluded from the borehole data

Rock Property	**Quality**	**Borehole Number**					
		PH 7		**PH 4**		**PH 3**	
		Aphanitic basalt ABA	**Brecciated basalt BB**	**Aphanitic basalt ABA**	**Brecciated basalt BB**	**Aphanitic basalt ABA**	**Brecciated basalt BB**
Rock Quality Designation (RQD)	**Excellent**	10	67	25	66.67	27.27	63.63
	Good	50	-	50	-	36.36	36.36
	Fair	30	33	16.67	33.33	18.18	-
	Poor	10	-	8.33	-	18.18	-
Uni-axial compressive strength (UCS)	**Very high**	12.5	-	-	-	-	-
	High	50	12.5	-	-	80	-
	Medium	25	-	-	-	20	11.11
	Low	12.5	50	-	-	-	55.55
	Very low	-	12.5	-	-	-	33.33

Table 4.14 Range and average values of the RQD (%) and UCS (Mpa)

Parameter		PH3		PH4		PH7	
		ABA	BB	ABA	BB	ABA	BB
RQD	Range	42 - 97	76 - 100	50 - 100	70 - 100	57- 93	64 - 96

	Average	76	92	77	90	81	81
UCS	Range	57 - 162	19 - 64	no data	no data	43 - 321	19 - 66
	Average	128.8	36			139	35

4.3 Review of Water pressure (Packer) tests

In addition to the parameters discussed in the previous section, boreholes were also utilized

to conduct the water pressure tests. Water pressure tests were mainly conducted to know the in-situ permeability condition of the lithological units at different positions. For this purpose packer test were conducted in the boreholes. The packer test is carried out by adding pressurized water to the selected borehole section. The rate at which water flows in to the surrounding ground through a response section sealed by a packer is measured. For this investigation the project authorities performed both single and double packer tests.

For water pressure tests, there is a pattern to be followed as proposed by Houlsby (1977). According to this approach, five consecutive tests are performed for 10 minutes, each at pumped pressures in sequence A, B, C, B, A, i.e. pressure increasing from A to C and then decreasing from C to A step wise. According to Houlsby (1976), the description of flow regimes and Lugeon units of the tests are shown through Fig. 4.2 and are summarized as;

(i) Laminar flow: same flow condition for all steps

(ii) Turbulent flow: the amount of inflow into rocks decreases as pressure increases

(iii) Void filling: inflow continuously decreases from the firs step to the last step

(iv) Dilatational flow condition: high inflow at high pressure

(v) Wash out flow condition: inflow goes increasing from the first step up to the last step.

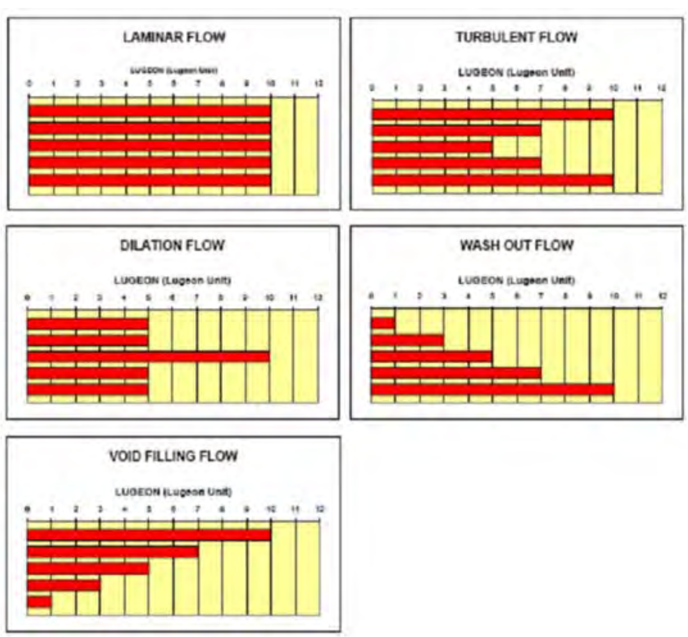
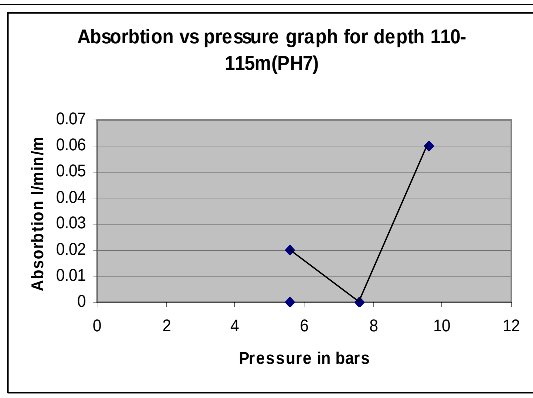


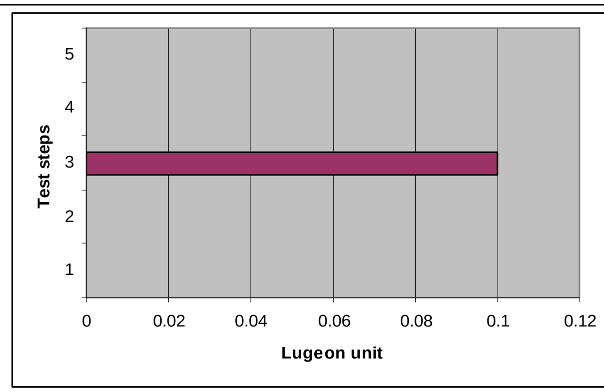
Fig. 4.3 Lugeon test pattern and the associated flow conditions

In total 73 in-situ permeability tests has been conducted in the boreholes around the project area. Lugeon test has been used to define the permeability of rock mass found at the power house area. Test results for each unit expressed as Lugeon units for borehole No. PH 3 and PH 7 are summarized in annexure "A" and the flow regime and lugeon values are shown in fig.4.4 and 4.5.

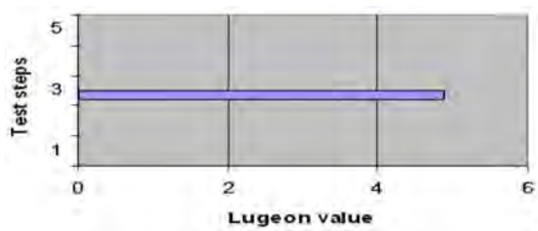
According to Lugeon (1933), a rock absorbing less than one lugeon unit can be considered watertight. Perusal of results (annexure 'A') clearly indicates that all rock types are almost impermeable with Lugeon units lower than 0.2, which is equal to 2.00 E^{-}8 m/s.

The mean in situ permeability for different rock units varies from 0.00 to 0.2 Lu. The minimum and maximum values are 0.00 Lu and 0.2 Lu for tertiary tuff (TT) and Aphanitic basalt (ABA) units respectively.





Flow regime: Void Filling at depth 110 – 115m



Flow regime: Dilation at depth 280-285 m

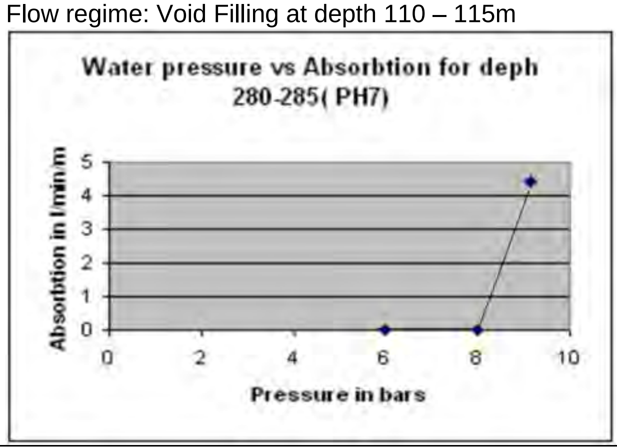


Fig. 4.4 Flow regime and Lugeon value, depths 110–115m and 280 – 285 m

The results presented in annexure 'A' clearly indicates that the permeability of the rock mass in the powerhouse cavity area are far below the limits and does not show any seriousness for the powerhouse cavity for the construction and performance stages. The limitation is that the result is only for specific locations. Nevertheless, this analysis is very help full to compare and contrast the pressure test results with the actual field observation as discussed in chapter III.

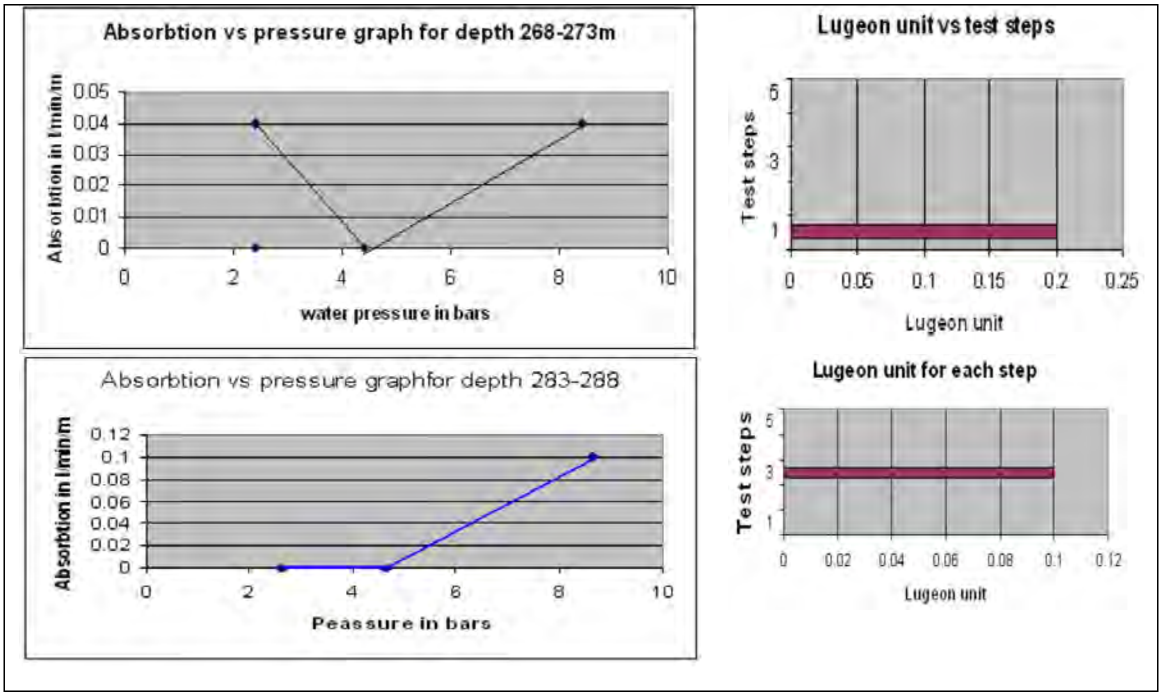


Fig. 4.5 Flow regime and Lugeon value, depths 268–273m and 283 – 288 m

Table 4.15 Summary of in situ mean permeability values in different rock units in the project area

Rock unit ***	Number of test	Minimum Value	Mean Value	Maximum Value	Standard deviation
		All units are in Lugion			
ABA	32	0	0.006	0.2	0.035
BB	31	0	0.006	0.1	0.025
VB	5	0	0.02	0.1	0.045
TT	5	0	0	0	0
*** ABA - Aphanitic basalt, BB - Brecciated basalt, VB – vesicular basalt and TT – Tertiary tuff					

(*Source: Powerhouse area final geological report by Salini Constructtori.(S.P.A), 2007*)

The review and analysis of the chapter from the secondary data (geophysical, borehole log and water pressure test data) confirms that the rock units in which the excavations are to take place are in general in good strength, quality and also water tight. However, the strength of the breccia unit is in low to moderate strength range. Any way, the general condition assures the suitability for the construction of the powerhouse cavern and access adits.

CHAPTER V ROCK MASS CLASSIFICATION

5.1 General Background

The need for a suitable classification in the field of rock mechanics has long been recognized and, in fact, numerous proposals have been made. Rock mass classification schemes have been developing for over 125 years since Ritter (1879) attempted to formalize an empirical approach for tunnel design, in particular for determining support requirements. While the classification schemes are appropriate for their original application, especially if used within the bounds of the case histories from which they were developed, considerable caution must be exercised in applying rock mass classifications to other rock engineering problems. Some of the popular rock classification systems developed so far are (i) Terzaghi's Rock load Classification (Terzaghi, 1946), (ii) Lauffer's Stand up time Classification (Lauffer, 1958), (iii) Rock quality designation or RQD Classification (Deer et al, 1967), (iv) Rock structure rating or RSR Classification (Wickham, et al, 1972), (v) Bieniawiski's Rock mass rating – RMR or Geomechanics Classification (Bieniawski, 1973) and Barton et al Q-system (Barton et al, 1974).

These schemes were developed from civil engineering case histories in which all of the components of the engineering geological character of the rock mass were included. Different classification systems place different emphases on the various parameters, and it is recommended that at least two methods be used at any site during the early stages of a project.

Classification system for rock mass is essential to ensure understanding, and communication among the concerned bodies. These aims should be fulfilled by ensuring that a classification system has the following attributes (Beniawski,1989).

(i) is simple and understandable.
(ii) each term is clear and the terminology used is widely applicable.
(iii) only the most significant properties of the rock masses are included.
(iv) is based on measurable parameters, which can be determined relevant tests quickly and cheaply in the field.
(v) is based on rating system, which can weight the relative importance of classification parameters.
(vi) is general enough so that the same rock will posses the same classification regardless of how it is being used.

In general, a rock mass classification has the following purposes:

(i) To divide a particular rock mass into groups of similar behavior.

(ii) To provide a basis for the understanding the characteristics of each group.

(iii) To yield quantitative data for the design of supports.

(iv) To provide a common basis or communication.

Nevertheless, although some of the existing classifications have a considerable potential, some are fully satisfactory in terms of the above pre-requisites (Bieniawski, 1979).Most of the above-mentioned classifications systems are based on rock material characteristics only and do not fully explain the properties of rocks for tunneling purposes. However, two classification systems widely used are the RMR and Q- systems.

Bieniawski's (1973) Geomechanics classification evolved from several earlier systems and has undergone several changes (1974, 1975, 1976, 1979 and 1989) since its first introduction in 1973. Bieniawski (1989) rates the six parameters in his RMR system: Uniaxial compressive strength (UCS), Rock quality designation (RQD), and spacing of joints, condition of joints, orientation of joints and ground water condition.

The common parameter used in both systems is RQD. Bianiawski includes joint spacing and orientation, while the Q-system includes the number of joint sets. Orientation is included implicitly in the Q-system by classifying the joint roughness and alteration of the most unfavorably oriented joint sets or discontinuities (Barton, 1988).

Some times, it is reported that from the two systems, the Q-system was found to be applicable to the worst conditions encountered during excavation. When considering stand up time versus unsupported span, the RMR system still seem to be conservative compared with the Q-system (Einstein et al, 1983).

The Q-system is more detailed than any other methods due to the factors such as joint roughness (degree of planarity), joint alteration (filling) and relative orientation. The use of joint roughness and joint alteration represents one of the strongest features of the method. It also seems to be a factor that is virtually ignored in the other classification systems. For example, in the RMR system, although data for all joint set and discontinuities are collected, only the average data are incorporated in the numerical ratings. Furthermore, in the RMR, it is impossible to separately vary the degree of joint roughness and the degree of infillings as obviously may occur in practice. The Q-system is essentially a weighting process in which the positive and negative aspects of a rock mass are assessed (Beniawiski, 1989).

Much of the problem in proposing a classification system is to select a set of parameters of greatest significance. It is believed that there is no single parameter or index which can fully and quantitatively describe a jointed rock mass. Various factors have different significance and only if taken together can they describe a given rock mass very satisfactorily. Although the significance of some factors may be different in different cases, for example, in roof stability and in blastability or drillability, certain parameters are related to one another and can be used in different applications. Thus for practical applications, it is advisable to use both RMR and Q- systems considering the most important and significant parameters (Beniawski, 1989)

5.2 Rock mass rating classification system (RMR)

In applying classification for heterogeneous and isotropic assemblage, it was necessary and convenient to distinguish in a number of structural uniform features and similar characteristics. Hence, the rock mass is divided into a number of structural regions usually coincide with a major structural features or with the change in the rock type. In some cases, significant change in discontinuity spacing or characteristics, within same rock type necessitates the division of the rock mass in to a number of small structural regions or domains.

For the present study all six parameters used in this classification system were measured in the field.

5.2.1 Uniaxial Compressive Strength (UCS)

Uniaxial compressive strength of intact rock material (UCS): is included for a number of reasons. If the discontinuities are widely spaced and the rock material is weak, the rock material properties will influence the behavior of the rock mass. The determination of UCS of rock material is a simple process for which standard techniques are available. Hence, Schmidt hammer was used on the excavation faces and UCS was determined by the empirical relation proposed by Barton and Choubey (1977). The Uniaxial compressive strength of the rock mass thus determined is presented in Table 5.1.

$\log_{10} \delta c = 0.00088 * \gamma R + 0.01$ ……eq.5.1

where; ‘δc’is the uniaxial compressive strength in Mpa, ‘γ’ is the unit weight in KN/M^3 and ‘R’ is the Schmidt hammer rebound number

Perusal of Table 5.1 indicates that UCS of the rock mass in the underground excavation for the powerhouse cavity varies from 51Mpa to 167.7Mpa with an average value of 101pa for aphanitic basalt unit and 40-48Mpa, average 43.2Mpa for breccia.

5.2.2 Rock Quality Designation

Rock quality designation (RQD) is a measure of drill hole (log) quality as obtained from borehole logs (Deer et al.,1967) or in other words RQD can be defined as a ratio of sum of core length (greater than 10 cm) to the total core run, represented as percent.

$$RQD = \frac{\text{sum of core pieces} > 10\text{ cm}}{\text{Total core run length}} * 100 \text{ --- from core logs} \qquad \text{……eq.5.2}$$

For the first case (Deer et al) RQD is a quantitative index based on modified core recovery procedure which incorporates only three pieces of hard sound core which are 100mm or greater in length. The core should be at least NW size (54.7 mm or 2.15 inches in diameter) and should be drilled with a double-tube core barrel. Shorter lengths of cores are ignored as they are considered to be due to close shearing, jointing or weathering in the rock mass (Bieniawski, 1979). RQD can vary from 0 -100, but in actual case RQD values of 0 and 100 are uncommon. The average range of RQD determined from logs for each borehole in the powerhouse cavity is given in Table 4.8 in chapter 4.

RQD can also be determine by empirical relations proposed by Palmstorm relation (Palmstorm, 1982) or by Scanline method (Priest and Hudson).

Palmstorm's Relation

$$RQD = 115-3.3Jv, \qquad \text{……..eq.5.3}$$

Where; 'Jv'is the volumetric count of joints > 10 cm on 1m * 1m exposure

Scanline Method of Priest and Hudson

$$RQD = 100\, e^{-0.1\lambda} (0.1\lambda+1) \qquad \text{……..eq.5.4}$$

Where; 'λ' is the mean number of discontinuities per meter.

However, for the present study RQD is determined using Palmstorm's relation. *RQD* is a directionally dependent parameter and its value may change significantly, depending upon the borehole orientation. The use of the volumetric joint count can be quite useful in reducing this directional dependence. *RQD* is intended to represent the rock mass quality in-situ. When using diamond drill core, care must be taken to ensure that fractures, which have been caused by handling or the drilling process are identified and ignored when determining the value of *RQD*. When using Palmstorm's relationship for exposure mapping, blast induced fractures should not be included when estimating *Jv*. Palmström (1982) suggested that when no core is available but discontinuity traces are visible in surface exposures or exploration adits, the *RQD* may be estimated from the number of discontinuities per unit volume. The results for RQD are shown in Table 5.1.

Perusal of Table 5.1 indicates that the RQD for the rock mass in the underground excavation for the powerhouse and other cavities varies from 72 % to 95% and90-98% with an average values of 83.5 and 95.3 % for aphanitic and breccia units respectively..

5.2.3 Spacing and orientation of joints

These parameters are of paramount importance for the stability of structures in jointed rock mass. The presence and spacing as well as the dip and strike of joints reduce the strength of rocks. The data on spacing and orientation of joints for classification purpose is obtained from a joint survey on exposures.

5.2.4 Condition of joints

The term includes spacing of joints (distance between joint surfaces), continuity and roughness of joints as well as gouge material. Tight joints with rough surfaces and no gouge have high strength. On the other hand open continuous joints will facilitate unrestricted inflow of ground water. The continuity of joints influences the extent which the rock materials and the joints separately affect the behavior of the rock mass.

5.2.5 Ground water condition

Ground water is known to have an important effect on the behavior of jointed rock mass and is one main component for rock mass classification.

5.2.6 RMR determination

Data collection for the above parameters was one of the main tasks during the present field work. Field data was collected from the exposed excavated face where the support treatment measures were not applied. Therefore, the locations from where the RMR data was collected were randomly located and it was not feasible to collect data on systematic basis.

Finally, based on above mentioned parameters RMR values for the rock mass in the powerhouse cavity has been determined and is presented in Table 5.1. Thus, from the results (Table 5.1) the range and average RMR values are 48-81, 66.23 and 57-69, 65.5 for aphanitic basalt (ABA) and Brecciated basalt (BB) rock units, respectively having equal average values.

Table 5.1 Rock Mass Rating (RMR) for Rock mass in Underground excavation

Location	UCS (Mpa)	RQD	J.spacing	Disco.length	Separation	Infilling	Roughness	weathering	G.w.condition	Orie. Of diccontinuties	RMR	Rock class
Irrigation by pass audit right wall	141.41	75.4										
20m onwards from bifurcation	12	17	9	3	5	1	6	5	15	-5	68	II
Same location on 10m length(ABA)	12	13	8	3	4	6	1	5	15	-5	62	II
Draft tube junction 1&2(ABA+BB)	68.2	88.6										
	7	17	10	3	1	5	2	4	14	-2	61	II
Face to irrigation by	68.2	92										
ABA	7	20	10	4	1	5	1	5	15	-2	66	II
BB	35.65	95										
	4	20	12	4	1	4	5	4	14	-2	66	II
D/s PH pillar left of gallery -1(BB)	48.7	98										
	8	20	16	2	1	4	5	5	10	-2	69	II
Pillar b/n galleries 1&2 (BB)	47.28	98										
	4	20	10	3	1	6	2	5	12	-6	57	III
Pillar b/n galleries 2&3(ABA)	141.41	88.6										
	12	17	12	4	0	2	1	5	15	-2	66	II
Gallery 1right wall(ABA)	85.85	92										
	7	20	12	5	1	6	1	6	15	-6	67	II
Gallery 1left wall(BB)	42.61	95										
	4	20	12	4	4	6	5	5	15	-6	69	II
Pillar b/n galleries 3&4(ABA)	153.97	95										
	12	20	12	4	4	6	1	5	15	-3	76	II
Face on draft tube unit 3(ABA)	167.72	95										
	15	20	14	4	1	6	2	6	15	-2	81	II
Face on draft tube unit 4	103.92	90										
	12	18	17	2	0	2	1	5	15	-5	67	II
Face on draft tube junction 1&2(ABA)	77.46	74.3										
	7	13	10	3	1	6	1	6	15	-12	50	III
Face on draft tube junction 1&2(BB)	38.97	98										
	4	20	13	4	1	4	5	14	5	-10	60	III
Pillar b/n galleries(ABA)	146.66	72.1										
	12	13	10	3	2	6	1	5	15	-12	55	III
PH cavern front face (BB)-17m	46.82	98										
	4	20	18	2	4	6	5	5	15	-2	77	II
U/s PH.C wall on BB-12m	64.58	97										
	7	20	11	3.5	4	5	3	5	12	-10	60.5	II
Gallery unit 1(ABA)	51	90										
	7	20	10	4	0	2	4	5	15	-5	62	II
PH adit right wall(ABA)	109.55	91										
	12	18	12	4	4	6	1	5.5	15	-8	69.5	II
PH adit left wall (ABA)	108.73	88.6										
	12	17	10	2	4	6	1	5	10	-2	65	II
Draft tube unit 4(ABA)	76	88										
	7	17	17	3	4	6	1	5	15	-2	73	II
BB	40	90										
	4	20	20	4	1	4	5	3	5	-2	64	II
Middle of irri. by pass wall(ABA)	97	93										
case-1(for one joint set)	7	20	16	2	0	2	3	6	15	-2	69	II
case -2(for other joint sets)	90	93										
	7	20	17	4	4	6	1	6	15	-2	78	II
Draft tube unit 4(ABA)	76	90										
	7	20	8	3.3	4	4	3	5	4	-10	48.3	III
BB	42	93										
	4	20	12	3	4	3	5	3	5	-5	54	III
Draft tube unit 3(ABA)	77	90										
	7	20	17.5	3	1	4	3	5	12	-10	62.5	II
BB	45	92										
	4	20	20	4	1	6	5	4	7	-2	69	II
U/s junction of draft tube junc.(1&2)	84.48	92										
ABA	7	20	9	3	4	6	1	6	15	-5	66	II
BB	44.95	96										
	4	20	18	4	4	5	5	4	5	-2	67	II
U/s junction of draft tube junc.(1&2)	7	13	8	4	1	6	1	4	15	-10	49	III

right wall (ABA)- estimation												

Based on the RMR values (Table 5.1) the rock mass exposed in the underground excavations are grouped into two classes; Class II –Good; and class III-Fair quality rock mass. The engineering performance of rock mass falling in 'Good' and 'Fair' class is presented in Table 5.2.

Table 5.2 Engineering performance of rock mass

RMR Rating	**61-80**	**41-60**
Rock Mass Class	II	III
Description	Good Rock	Fair Rock
Average Stand-up Time	6 months for 8 m span	1 week for 5 m span
Rock Mass Cohesion (Kpa)	300 - 400	200 - 300
Rock Mass Friction Angle	$35^o – 45^o$	$25^o – 35^o$

Further secondary data on RMR has also been utilized to assess the rock mass quality in the underground cavity. Table 5.3 presents the RMR data for the rock mass from the secondary data (Source; construction follow up report of EEPCO, 2006-2007)

Table 5.3 Secondary data on RMR rating for the two units (combined)

Location	**Spacing**	**RQD**	**UCS**	**Joint leng.**	**Opening**	**Roughness**	**Infilling**	**Weathering**	**GWC**	**Orient.of dis.**	**RMR**	**Rock class (Beniawski)**	**Rock class (agreement)**
Audit to draft tube	15	17	12	5	5	3	4	6	14	-9	72	II	A
	14	16	12	4	4	4	4	5	15	-5	73	II	A
	16	17	11	4	5	4	5	5	13	-5	75	II	A
	15	18	10	4	4	5	5	5	14		75	II	A
	18	18	12	2	6	3	5	5	15	-8	76	II	A
	15	18	11	5	5	4	4	5	14	-5	76	II	A
	16	18	11	5	5	4	5	5	14	-9	74	II	A
By pass adit to TRT	16	18	10	4	5	4	4	5	13	-8	71	II	A
	18	17	10	5	5	5	4	4	14	0	82	I	A
	18	18	10	6	6	4	6	4	13	0	85	I	A
	15	19	10	6	5	3	4	5	14	-5	76	II	A
	19	18	12	4	5	4	4	5	14	-5	80	II	A
	18	17	10	5	5	4	4	5	14	-5	77	II	A
	16	18	10	4	5	4	4	5	13	-8	71	II	A
Draft tube unit 4	15	17	11	3	5	3	4	5	5	-5	63	II	B
	13	17	11	3	4	3	4	5	8	-3	65	II	B
	17	19	11	3	5	3	4	4	15	-7	74	II	A
Audit to ventilation shaft	15	18	11	4	5	5	4	5	14	-12	69	II	B
Draft tube to u/s unit 1&2	12	18	10	3	2	3	4	4	10	-6	60	II	B
	10	17	11	3	4	2	4	5	13	-8	61	II	B
	10	18	10	3	4	3	5	4	15	-5	67	II	B
Gallery unit 2 face	10	17	11	3	1	3	2	4	13	-6	58	III	B

The RMR values presented in Table 5.3 are based on Rock mass classification adjustment on the agreement document for excavation. For this purpose the construction company (Salini Constracttori S.P.A) has developed its own rock class adjustment from RMR values for the price quotation of excavation activities (table 5.4). Moreover, the rating of the secondary data listed in Table 5.3 is not done separately for each rock unit. It shows the combined effect; it does not give clear image of each rock unit with its respective quality.

Table 5.4 Rock mass classification adjustment

Rock mass classification adjustment based on Beniawski (1989) from RMR value				
100-81	80-61	60-41	40-21	<21
I	II	III	IV	V
Very good	Good	Fair	Poor	Very poor
Rock mass classification adjustment on the agreement document for excavation				
100-71	70-36	35-21	<21	
A	B	C	D	

(a) Five basic rock mass classification parameters and their ratings

1. Strength of intact rock material	Point load strength index (MPa)	> 10	4 – 10	2 – 4	1 – 2			
	Uniaxial compressive strength (MPa)	> 250	100 – 250	50 – 100	25 – 50	5 – 25	1 – 5	< 1
Rating		*15*	*12*	*7*	*4*	*2*	*1*	*0*

2. RQD (%)	90 – 100	75 – 90	50 – 75	25 – 50	< 25
Rating	*20*	*17*	*13*	*8*	*3*
3. Joint spacing (m)	> 2	0.6 – 2	0.2 – 0.6	0.06 – 0.2	< 0.06
Rating	*20*	*15*	*10*	*8*	*5*
4. Condition of joints	not continuous, very rough surfaces, unweathered, no separation	slightly rough surfaces, slightly weathered, separation <1 mm	slightly rough surfaces, highly weathered, separation <1 mm	continuous, slickensided surfaces, or gouge <5 mm thick, or separation 1–5 mm	continuous joints, soft gouge >5 mm thick, or separation >5 mm
Rating	*30*	*25*	*20*	*10*	*0*

5. Groundwater	inflow per 10 m tunnel length (l /min), or joint water pressure/major in situ stress, or general conditions at excavation surface	none 0 completely dry	< 10 0 – 0.1 damp	10 – 25 0.1 – 0.2 wet	25 – 125 0.2 – 0.5 dripping	> 125 > 0.5 flowing
Rating		*15*	*10*	*7*	*4*	*0*

(b) Rating adjustment for joint orientations

Strike and dip orientation of joints		very favourable	favourable	fair	unfavourable	very unfavourable
Rating	tunnels	0	– 2	– 5	– 10	– 12
	foundations	0	– 2	– 7	– 15	– 25
	slopes	0	– 5	– 25	– 50	– 60

(c) Effects of joint orientation in tunnelling

Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		Dip 0° – 20°
Drive with dip		Drive against dip				
Dip 45° – 90° very favourable	Dip 20° – 45° favourable	Dip 45° – 90° fair	Dip 20° – 45° unfavourable	Dip 45° – 90° very unfavourable	Dip 20° – 45° fair	irrespective of strike fair

Perusal of Table 5.3 indicates that the rock mass as per rock mass classification adjustment on the agreement document for excavation falls into ‘A’ and ‘B’ which implies that rock mass is of ‘Very Good’ and ‘Good’ class.

5.3 The rock mass Quality index (Q) system

The concept of rock mass quality index (Q) system that is introduced by Barton et al (1974) is used for determination of rock mass characteristics and support requirements. This method is considered to provide the best approximation to the actual conditions to be encountered. The numerical values of Q range from 0.001 in case of very poor and very weak rock to 1000 for very competent massive rock (Barton et al, 1974).

The six parameters taken into consideration for classifying the rock mass quality are;

(i) Rock quality designation (RQD)
(ii) Number of joint sets (Jn)
(iii) Joint roughness value (Jr)
(iv) Joint alteration number (Ja)
(v) Joint water reduction factor (Jw)
(vi) Stress reduction factor (SRF)

Taking in to account the sum total of the above mentioned observations according to the rating table in annexure 'B', the rock mass quality classification system is given by:

Q = RQD/Ja*Jr/Ja*Jw/SRF …….eq. 5.5

For the present study data pertaining to above mentioned 6 parameters were collected from the exposed rock face in the underground excavation. The 'Q' value determined is presented in Table 5.5.

Table 5.5 Ratings for 'Q' system and computed quality of the rock mass

S.N	Location	RQD	Jn	Jr	Ja	Jw	SRF	Q	Quality
1	PH C. access adit Right wall (ABA)	90	9	1	1	1	**2.5**	4.00	Fair
2	PH C. access adit Left wall (ABA)	88.6	9	1	1	1	**2.5**	3.94	poor
3	Right side of irri-by access adit(ABA)	72.1	9	1.5	2	1	**2.5**	2.40	poor
4	Additional on same wall	75.4	9	1	1.5	1	**2.5**	2.23	poor
5	Draft tube junction 1&2 face(ABA+BB)	88.6	9	3	1.5	1	**2.5**	7.88	Fair
6	Face on irri by pass (ABA)	91	9	2	1	1	**2.5**	8.09	Fair
7	Face on irri by pass (BB)	95	6	3	2	1	**2.5**	9.50	Fair
8	D/s PHc left of bus duct gallery 1(BB)	98	6	1.5	1	1	**2.5**	9.80	Fair
9	D/s PHc pillar face b/n galleries 1&2(BB)	98	9	1.5	1	1	**2.5**	6.53	Fair
10	D/s PHc pillar face b/n galleries 2&3(ABA)	88.6	6	1.5	0.75	1	**2.5**	11.81	Good
11	Right wall of bus duct gallery unit 1(ABA)	92	6	1	1	1	**2.5**	6.13	Fair
12	D/s PHc pillar face b/n galleries 3&4(ABA)	95	6	1	1	1	**2.5**	6.33	fair
13	Face of draft tube unit 3(ABA)	95	9	2	1	1	**2.5**	8.44	Fair
14	Face of draft tube unit 4(ABA)	90	12	2	1.5	1	**2.5**	4	Fair
15	Draft tube junction 1&2 face(ABA)	74.3	13	1	1	1	**2.5**	2.29	poor
16	Draft tube junction 1&2 face(BB)	98	12	3	2	1	**2.5**	4.90	Fair
17	D/s PHc pillar face b/n two galleries (ABA)	72.1	12	1	1	1	**2.5**	2.4	poor
18	Front face of PH cavern (BB)	98	6	1.5	0.75	1	**2.5**	13.07	good
19	U/S wall of PH cavern (BB)	97	9	1.5	1	1	**2.5**	6.47	Fair
20	Middle wall of irr. By pass(ABA)	93	9	1	1	1	**2.5**	4.13	Fair
21	Draft tube unit 4(ABA)	92	9	1.5	1	1	**2.5**	6.13	Fair

22	Draft tube unit 3(ABA)	90	6	1.5	1	1	**2.5**	9.00	Fair
23	U/S draft tube junci1&2(ABA)	90	6	2	2	1	**2.5**	6.00	Fair

From Table 5.5 the average value of ‘Q’ for Aphanitic and brecciated rock mass is 5 and 8, respectively. Accordingly the rock mass for both the rock units falls into Class-C which designates ‘Fair quality rock mass’.

5.4 Support Requirement Based on Rock Mass Classification

Design and construction of tunnels and shafts in rock require thought processes and procedures that are in many ways different from other design and construction projects, because the principal construction material is the rock mass itself rather than an engineered material. Uncertainties persist in the properties of the rock materials and in the way the rock mass and the groundwater will behave. These uncertainties must be overcome by sound, flexible design and redundancies and safeguards during construction.

In underground excavations in hard rock, failure is frequently controlled by the presence of discontinuities such as faults, shear zones, bedding planes and joints. The intersection of these structural features can release blocks or wedges which can fall or slide from the surface of the excavation. Failure of the intact rock is seldom a problem in these cases where deformation and failure are caused by sliding along individual discontinuity surfaces or along lines of intersection of surfaces. Separation of planes and rotation of blocks and wedges can also play a role in the deformation and failure process. An analysis of the stability of these excavations depends primarily upon a correct interpretation of the structural geological conditions in the rock mass followed by a study of the blocks and wedges, which can be released by the creation of the excavation. Identification and visualization of these blocks and wedges is by far the most important part of an investigation for support designs.

Judgment on the adequacy of a support design has to be based upon an evaluation of a number of factors such as the magnitude and distribution of deformations in the rock and the stresses induced in support elements such as grouted cables, steel sets or concrete linings. (Hoek, 1999).

From the above discussion it may easily be understood that no analytical method is capable of defining the exact rock mass quality and its stability particularly in underground openings. Experience has shown that ‘Rock mass classification systems’ are being utilized successfully to work out the support design for the underground openings. For this purpose both ‘RMR’ and ‘Q’ system are widely employed world over.

5.4.1 Support by RMR System

Bieniawiski (1989) has suggested Guidelines for excavation and support of 10 m span rock tunnels. For the present study these guidelines may not be adopted as the Underground cavity for the present case has span greater than 10m. However, the guidelines suggested by Bieniawiski may be followed for the associated powerhouse structures (penstock, draft tube, galleries, etc.), which have a span less than 10m. Table 5.6 shows the Guidelines for excavation and support up to 10 m span rock tunnels for Fair and Good quality rock mass.

Table 5.6 Guidelines for excavation and support of 10 m span rock tunnels in accordance with the RMR system (After Bieniawski 1989).

Rock Mass Class	Excavation	Rock bolts (20 mm dia, fully grouted)	Shotcrete	Steel sets
II – Good Rock RMR: 61-80	Full face, 1-1.5 m 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
III – Fair Rock RMR: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast complete support 10 m from face	Systematic bolts 4 m long spaced 1.5 – 2m in crown & walls with wire mesh in crown	50-100 mm in crown and 30 mm in sides	None

5.4.2 Support by Barton et al 'Q' System

The Q value is related to tunnel support requirements by defining the equivalent dimensions of the excavation. This equivalent dimension, which is a function of both the size and the purpose of excavation, is obtained by dividing the span, diameter or the wall height of the excavation by a quantity called the excavation support ratio (ESR).

$$De = \frac{\text{Excavation span, diameter or height (M)}}{\text{Excavation support ratio (ESR)}} \qquad \text{.......eq.5.6}$$

The ESR is related with the use of the excavation and the security and safety of the excavation and is given in Table 5.7.

Table 5.7 Excavation support ratio (ESR) for various Excavation Categories

	Excavation Category	ESR
A.	Temporary mine openings	3-5
B.	Permanent mine openings, water tunnel for hydro power (excluding high pressure penstocks). Pilot tunnel drifts and heading for large excavation.	1.6
C.	Storage rooms, water treatment plants minor road and railway tunnels, surge chambers, access tunnels.	1.3
D.	Power stations, major road and railway tunnels, civil defense	1.0

	chambers, portal intersections	
E.	Underground nuclear power stations, railway station and sports and public facility, factories	0.8

Grimstad and Barton (1993) proposed Estimated support categories based on the tunneling quality index *Q.* (Annexure 'C'**).** For the present powerhouse the value of Excavation support ratio (ESR) is 1. Thus based of 'Q' values attempt has been made to workout the supports for the underground excavation (Table 5.8).

Table 5.8 Proposed Estimated support for rock mass in Underground cavity

S.N	Location	Q	Quality	De	Support Requirement		
					Rock Bolt Length (m)-calculated	Thickness of Shotcrete Layer (mm)	Bolt Spacing (m)
1	PH C. access adit Right wall (ABA)	4.00	Fair	8	3.2	50	2.2
2	PH C. access adit Left wall (ABA)	3.94	poor	8	3.2	45	2.2
3	Right side of irri-by access adit(ABA)	2.40	poor	6	2.9	50	2.1
4	Additional on same wall	2.23	poor	7	3.05	50	2
5	Draft tube junction 1&2 face(ABA+BB)	7.88	Fair	6	2.9	42	2
6	Face on irri by pass (ABA)	8.09	Fair	7	3.05	40	1.8
7	Face on irri by pass (BB)	9.50	Fair	6	2.9	40	2
8	D/s PHc left of bus duct gallery 1(BB)	9.80	Fair	7	3.05	40	2.4
9	D/s PHc pillar face b/n galleries 1&2(BB)	6.53	Fair	22	5.3	80	2.3
10	D/s PHc pillar face b/n galleries 2&3(ABA)	11.81	Good	22	5.3	60	2.4
11	Right wall of bus duct gallery unit 1(ABA)	6.13	Fair	7	3.05	42	2.3
12	D/s PHc pillar face b/n galleries 3&4(ABA)	6.33	fair	22	5.3	80	2.3
13	Face of draft tube unit 3(ABA)	8.44	Fair	7	3.05	40	2.4
14	Face of draft tube unit 4(ABA)	4	Fair	6	2.9	50	2.1
15	Draft tube junction 1&2 face(ABA)	2.29	poor	6	2.9	50	2.1
16	Draft tube junction 1&2 face(BB)	4.90	Fair	6	2.9	50	2.2
17	D/s PHc pillar face b/n two galleries (ABA)	2.4	poor	22	5.3	110	2
18	Front face of PH cavern (BB)	13.07	good	60	14	110	2.5
19	U/S wall of PH cavern (BB)	6.47	Fair	22	5.3	80	2.3
20	Middle wall of irr. By pass(ABA)	4.13	Fair	7	3.05	50	1.6
21	Draft tube unit 4(ABA)	6.13	Fair	6	2.9	40	2.3
22	Draft tube unit 3(ABA)	9.00	Fair	6	2.9	40	2.4
23	U/S draft tube junci1&2(ABA)	6.00	Fair	6	2.9	40	2.3

Barton et al. (1980) provided a relation through which, length of the rock bolts can be estimated. Accordingly;

$L = \underline{2+0.15B}$ ……eq. 5.7

ESR

Where; ‘B’ is the excavation width and ESR is the Excavation support ratio.
Thus calculated rock bolts length for stabilizing rock mass at various locations in the underground excavation is presented in Table 5.8. Perusal of Table 5.8 indicates that, in general, the Rock Bolt length varies from 2.9 to 5.3 m. with bolt spacing in a range of 1.6 to 2.4 m. Also the thickness of Shotcrete Layer varies in the range of 40 to 110 mm. However, if the cavern roof height and wall length are considered the bolt lengths can be as long as about 12m.

The maximum unsupported span can also be determined by using ‘Q’ and ‘ESR’ values. According to Barton et al the maximum unsupported span in underground excavation can be estimated as;

$$\text{Maximum span unsupported} = 2ESRQ^{0.4} \qquad \text{……eq. 5.7}$$

According to Barton et al the Permanent roof support pressure (proof) in underground excavation can be estimated as;

$$\text{proof} = \frac{2\sqrt{Jn}\,Q^{-1/3}}{3Jr} \qquad \text{……eq. 5.8}$$

Table 5.9 presents the ‘Maximum un-supported span’ and Permanent roof support pressure at different locations in the underground excavation.

Thus, from the results (Table 5.9) it may be concluded that the maximum un-supported span in the underground excavation varies from 2.84 to 5.6 m for the powerhouse cavern walls and roofs and 3.6 to 6.5 for accessory tunnels. Also the permanent roof support pressure varies from 0.46 to 1.72 and 0.26 to 1.82m for powerhouse cavern and access tunnels respectively.

Table 5.9 'Maximum un-supported span' and Permanent roof support pressure at different locations in the underground excavation

S.N	Location	Q	Quality	Maximum Un-supported span (m)	Permanent roof support pressure (Mpa)
1	PH C. access adit Right wall (ABA)	4.00	Fair	4.52	1.26
2	PH C. access adit Left wall (ABA)	3.94	poor	4.50	1.27
3	Right side of irri-by access adit(ABA)	2.40	poor	3.69	1.00
4	Additional on same wall	2.23	poor	3.59	1.53
5	Draft tube junction 1&2 face(ABA+BB)	7.88	Fair	5.93	0.33
6	Face on irri by pass (ABA)	8.09	Fair	5.98	0.50
7	Face on irri by pass (BB)	9.50	Fair	6.40	0.26
8	D/s PHc left of bus duct gallery 1(BB)	9.80	Fair	6.50	0.51
9	D/s PHc pillar face b/n galleries 1&2(BB)	6.53	Fair	4.20	0.71
10	D/s PHc pillar face b/n galleries 2&3(ABA)	11.81	Good	5.40	0.48
11	Right wall of bus duct gallery unit 1(ABA)	6.13	Fair	5.20	0.89
12	D/s PHc pillar face b/n galleries 3&4(ABA)	6.33	fair	4.18	0.88
13	Face of draft tube unit 3(ABA)	8.44	Fair	6.11	0.49
14	Face of draft tube unit 4(ABA)	4.00	Fair	4.52	0.73
15	Draft tube junction 1&2 face(ABA)	2.29	poor	3.61	1.82
16	Draft tube junction 1&2 face(BB)	4.90	Fair	4.89	0.45
17	D/s PHc pillar face b/n two galleries (ABA)	2.40	poor	2.84	1.72
18	Front face of PH cavern (BB)	13.07	good	5.60	0.46
19	U/S wall of PH cavern (BB)	6.47	Fair	4.22	0.72
20	Middle wall of irr. By pass(ABA)	4.13	Fair	4.58	1.25
21	Draft tube unit 4(ABA)	6.13	Fair	5.36	0.73
22	Draft tube unit 3(ABA)	9.00	Fair	6.24	0.52
23	U/S draft tube junci1&2(ABA)	6.00	Fair	5.30	0.45

5.5 Comparison between RMR and Q system

Both RMR and Q system are based on a rating of three principal properties of rock mass. These are; (i) Intact rock strength, (ii) The principal properties of discontinuities and (iii) Geometry of intact blocks of rock defined by discontinuity. For the Q system the intact rock strength is only a factor in the content of the induced stress by the rock as defined by the SRF term. Variation in values of important parameter of RMR and Q system is shown through Table 5.10 (Milne, 1988).

Table 5.10 Variation in important parameters in RMR and Q system

	Q	RMR
Basic range in values	0.001 to 1000	8 to 100
Strength as % of total range	19%	16%
Block size as % of total range	44%	54%
Discontinuity friction as % of the total range	39%	27%

Table 5.10, shows the similarity between the weightings given to the three basic rock mass properties. Despite this, there is no basis for assuming the two systems should be directly related. The assessment for intact rock strength and stress is significantly different in the two

systems. In spite of their differences, it is common practice to use the rating from one system to estimate the rating value of the other. The following equation proposed by Bieniawski (1976) is the most popular, relation between Q and RMR:

RMR = 9 ln Q + 44 ……eq. 5.9

Based on Beniawski's RMR and Q relation, different countries modified the above relation to suite the overall intact rock and discontinuity properties of their respective local condition. Here it is worth mentioning that both the rating systems, RMR and Q, are based on case histories and may differ in certain situations when used in different conditions. Many attempts has been made by many workers to establish correlations between RMR and Q system. Choquet and Hadjigeorgiou ((Miline et al., 1995) compiled these correlation and are presented in Table 5.11.

During the present study also attempt has been made to correlate RMR with Q system. For this purpose data pertaining to RMR and Q system has been collected from the underground powerhouse cavity. Care was taken to collect the data from the identical locations. RQD being a common parameter for both RMR and Q system, therefore data for it was collected only once at respective locality. In order to make correlation between RMR and Q system data from each lithology was collected separately. The correlations thus established are presented through Table 5.11 and Fig. 5.1.

Table 5.11 Correlation between RMR and Q system world wide.

Correlation	Source	Comments
RMR=13.5 log Q + 43	New Zealand	Tunnels
RMR=9ln Q + 44	Diverse Origin	Tunnels
RMR=12.5 log Q + 55.2	Spain	Tunnels
RMR=5 ln Q + 60.8	S.Africa	Tunnels
RMR=43.89-9.19ln Q	Spain	Mining soft rock
RMR=10.5 ln Q + 41.8	Spain	Mining soft rock
RMR=12.11 log Q + 50.81	Canada	Mining hard rock
RMR= 8.7 ln Q + 38	Canada	Tunnels, sed. rock
RMR=10 ln Q +39	Canada	Mining hard rock

Table 5.12 Correlation between RMR and Q as established during the present study

Rock Type	Correlation Established During Present Study
Aphanitic rock	RMR = 3.23 ln Q + 61
Brecciated rock	RMR = 15.44 ln Q+ 34.27
Combined rock units	RMR = 4.07 ln Q + 59

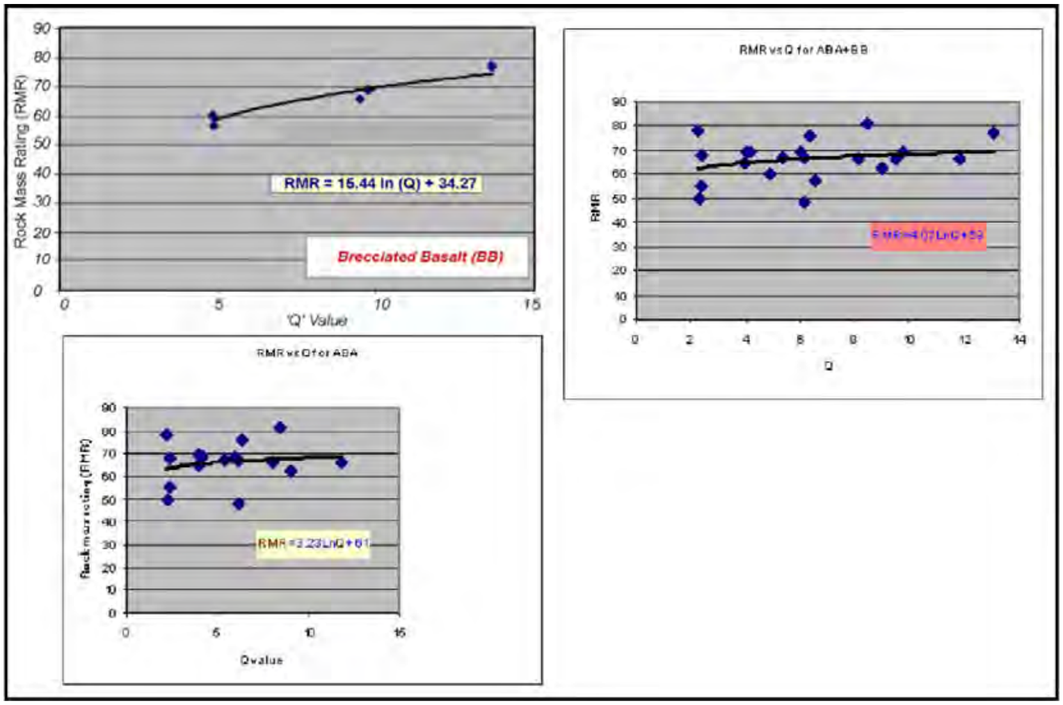


Fig. 5.1 Correlation between RMR and Q as established during the present study

Moreover, from the classification of the rock masses using RMR and Q systems the summarized table 5.13 is prepared with fig 5.2 for check up of the general correlations among the parameters UCS, RQD, RMR and Q values for ABA and BB units left and right table. Note that the average RMR values of the two rock units is the same (65) but considerable variation is there on UCS and Q values. The average RQD values are almost comparable being 90 and 95 though it is 83.5 in table 5.13 for ABA unit since there are some data not included.

The charts (Fig. 5.2) indicate that there is well defined concentration (clusters) of parameters indicating the quality of the data collected for analysis. It will also be very important to compare the RQD and UCS values determined from the core samples in chapter 4, Tables 4.14 and those determined from field data in Table 5.13 to see the variations and justification for the same (Table 5.13).

Table 5.13 Summarized results of rock mass classifications

UCS	RQD	RMR	Q	UCS	RQD	RMR	Q
141	75.4	68	2.4	35.65	95	66	9.5
68.2	92	66	8.09	48.7	98	69	9.8
141	88.6	66	11.81	47.28	98	57	6.53
85.5	92	67	6.13	42.61	95	69	
153.97	95	76	6.33	38.97	98	60	4.9
167.72	95	81	8.44	46.82	98	77	13.1
103.9	90	67	5.33	40	90	64	
77.43	74.3	50	2.29	42	93	54	
146.66	72	55	2.4	45	92	66	
51	90	62		44.95	96	67	
109.55	91	69.5	4	**R=40-48.97**	**90-98**	**54-77**	**4.9-13.07**
108.73	88.6	65	3.94	**Ave=43.2**	**95**	**64.9**	**8.76**
76	88	73					
97	93	69	4.13				
90	93	78	2.23				
76	90	48.3	6.13				
84.48	90	69	6				
77	92	62.5	9				
70	72	49	0				
R=51-167.7	**72-95**	**49-81**	**2.23-11.81**				
Ave=101.32	**83.5**	**65.33**	**4.67**				

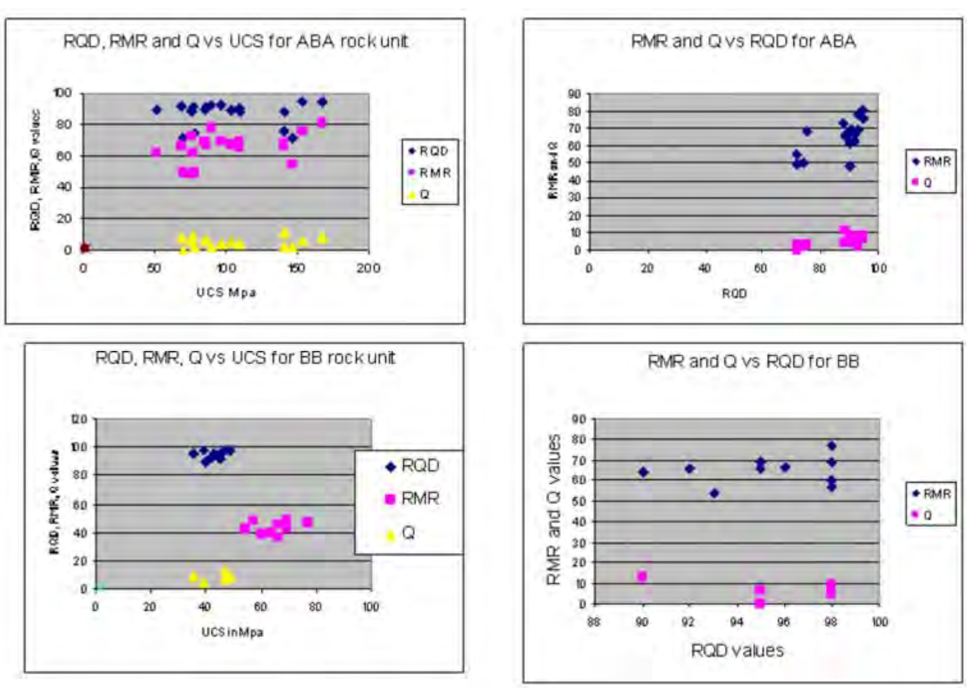


Fig 5.2 Correlations among the rock mass characterization parameters

Table 5.14 Summarized RQD and UCS values of log and field data

Borehole data							
Parameter		**PH3**		**PH4**		**PH7**	
		ABA	BB	ABA	BB	ABA	BB
RQD	Range	42 - 97	76 - 100	50 - 100	70 - 100	57- 93	64 - 96
	Average	**76**	**92**	**77**	**90**	**81**	**81**
UCS	Range	57 - 162	19 - 64	no data	no data	43 - 321	19 - 66
	Average	**128.8**	**36**			**139**	**35**
Field data		ABA	BB				
RQD	Range	72-95	90-98				
	Average	**83.5**	**95**				
UCS	Range	51-167.7	40-48.97				
	Average	**101**	**43**				

The average RQD values of the units from the summary Table 5.13 are comparable with the average values summarized in Chapter 4, Tables 4.14 from core log samples but show some degree of reduction. In addition, the average UCS values for the breccia rock unit are in the same range. However, the average UCS values of aphanitic basalt show higher variation as compared to the UCS values determined from the core log samples. In general, the comparison requires primary data collection and ground verification to work out the values of parameters for analysis and design purposes.

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CHAPTER VI ENGINEERING GEOLOGICAL CHARACTERIZATION OF ROCKS

6.1 General

Design and construction of tunnels and shafts in rock require thought processes and procedures that are in many ways different from other design and construction projects, because the principal construction material is the rock mass itself rather than an engineered material. Uncertainties persist in the properties of the rock materials and in the way the rock mass and the groundwater will behave. These uncertainties must be overcome by sound, flexible design and redundancies and safeguards during construction. More than for any other type of structure, the design of underground structure must involve selection or anticipation of methods of construction. When an underground structure or shaft is excavated, the rock stresses are disturbed around the opening and displacements will occur. The rock mass is often able to accommodate these stresses with acceptable displacements. The stable rock mass around the opening in the ground, often reinforced with rock bolts, shotcrete, or other components, is an underground structure, but a definition of the degree of stability or safety factor of the structure is elusive.

Unfortunately, geologic materials are inherently variable, and it is difficult to define their properties with any certainty along a length of underground structure. In fact, most underground structures must traverse a variety of geologic materials, the character of which may be disclosed only upon exposure during construction. Thus, ground reinforcement and lining must be selected with adaptability and redundant characteristics, and details of construction must remain adaptable or insensitive to variations in the ground (Hoek, 1983).

6.2 Intact Rock and Rock Mass Properties

6.2.1 Intact rock properties

Understanding the properties of rocks is an important part of exploration, design and construction of an engineering project. Based on their physical and chemical properties, rocks may give different responses when civil engineering structures are interfered with them. For an engineering use rocks are divided into two terms as intact rock and rock mass. Intact rock is the term applied to a rock material with out discontinuities while rock mass is a rock containing discontinuities such as joints and bedding planes

The important engineering geological properties of intact rocks according to the Geological Society Engineering Group Working party (1977) are given in Table 6.1.

Table 6.1 Engineering properties of intact rocks

(i) Rock type	(vii) Hardness	(xiii) Young's Modulus
(ii) Color	(viii) Durability	(xiv) Poisson's ratio
(iii) Grain size	(ix) Porosity	(xv) Primary permeability
(iv) Texture and fabric	(x) Density	
(v) Weathering	(xi) Strength	
(vi) Alteration	(xii) Sonic velocity	

For an intact rock characterization of the rock units of the powerhouse area, the construction company has conducted different laboratory tests. These tests are unit weight, unconfined compressive strength, young's modulus, tensile strength, triaxial test and sonic velocities. The test results for these properties are presented in Tables 6.2, 6.3, 6.4(Final Geological report of the powerhouse area, by Salini constructtori. S.P.A., 2007).

Table 6.2 Unit weight of different rock units exposed in Underground cavity

Unit	No of tests (#)	Min (KN/m^3)	Mean (KN/ m^3)	Max (KN/ m^3)	ST.dev. (KN/ m^3)	Remark
ABA	49	20.6.	25.752	28.17	2.16	Max. unit weight
VB	9	20.60	22.50	24.60	1.35	-
BB	37	20.00	21.90	24.80	1.09	-
TT	28	13.05	15.36	18.24	1.45	Min. unit weight
ABA Aphanitiv basalt VB – Vesicular basalt			BB – Breccia basalt TT – Tertiary tuff			ST.dev= standard deviation

Table 6.3 Uniaxial compressive strength results of different rock units exposed in Underground cavity

Unit	No of tests (#)	Min (Mpa)	Mean (Mpa)	Max (Mpa)	ST.dev. (Mpa)	Remark (Strength range)
ABA	18	61.7	183.60	364.40	84.90	Strong-Very strong
VB	7	17.8	34.30	52.90	13.20	Low-Moderately strong
BB	22	11.4	47.0	105.80	26.90	Low - Moderately strong
TT	17	6.8	17.90	39.60	9.80	Low strength
ABA Aphanitic basalt VB – Vesicular basalt				BB – Breccia basalt TT – Tertiary tuff		

Table 6.4 Young's Modulus of different rock units exposed in Underground cavity

Unit	No of tests (#)	Min (Kpa)	Mean (Kpa)	Max (Kpa)	ST.dev. (Kpa)
ABA	17	24,480	67,755	91,968	17,929
VB	7	5,397	28,050	65,380	23,065
BB	22	3,834	19,959	54,552	13,254
TT	17	895	2,964	6,426	1,958
ABA Aphanitic basalt VB – Vesicular basalt			BB – Breccia basalt TT – Tertiary tuff		

The results when compared with the bi-logarithmic Deer-Miller chart and with intact rock strength classification, table 6.5, the aphanitic basalt unit falls in to the category of strong to very strong rock (UCS 50-200Mpa) with high stiffness; brecciated and vesicular basalt units are low- moderately to strong rock with medium stiffness and tertiary tuffs low strength rocks with low stiffness.

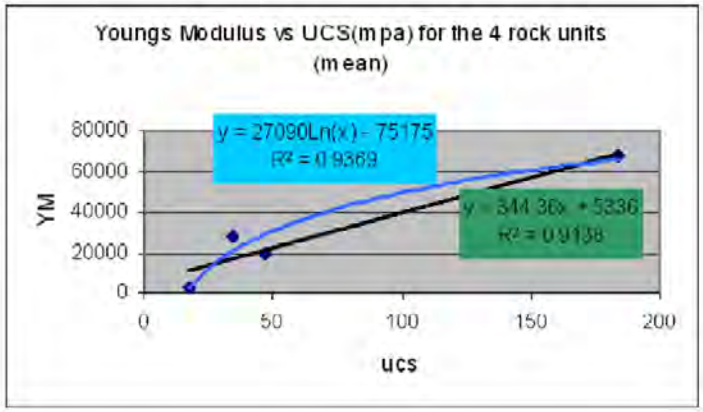
Table 6.5 Deere and Miller's classification of intact rock strength

Description	Strength MPa	Examples of rock types
Very low strength	1 - 25	Chalk, Rock salt
Low strength	25 - 50	Coal, Siltstone, Schist
Medium strength	50 - 100	Sandstone, Slate, Shale
High strength	100 – 200	Marble ,Granite, Gneiss
Very high strength	> 200	Quartz, Dolerite, Gabbro, Basalt

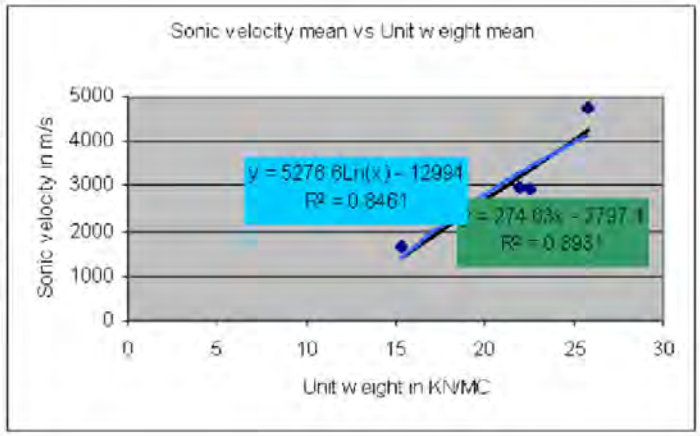
The plots (Fig. 6.1(a) and (b)) show the correlations of the parameters from the summary result (min, mean and max) of Tables (6.2, 6.3 & 6.4). The plots show good correlation among the different parameters from the summary results. However, if considered from the total laboratory values, there are no such good correlations because there are variation depending on the degree of weathering and fracturing.

The charts (Fig. 6.2) demonstrates the relations between different parameters worked out considering total number of laboratory results listed in annex annexure 'D'. The charts show that there is well defined cluster of parameters for each rock unit.

	UCS	YM
ABA	183.6	67755
VB	34.3	28050
BB	47	19959
TT	17.9	2964



	UW(mean)	SV(mean)
ABA	25.75	4732
VB	22.5	2952
BB	21.9	2960
TT	15.36	1651



UCS	YM
61.7	24480
183.6	67755
346.4	91968

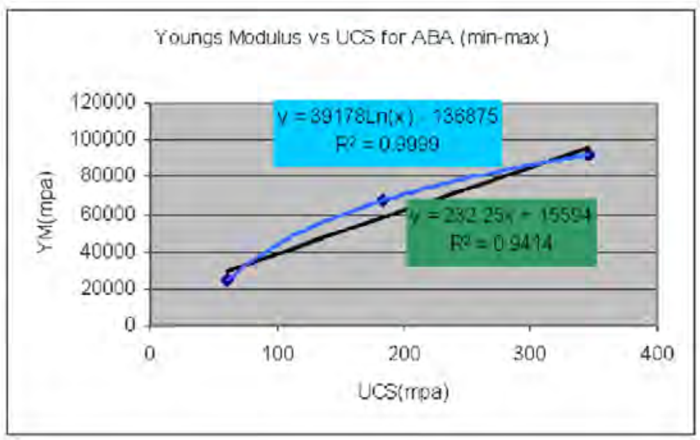
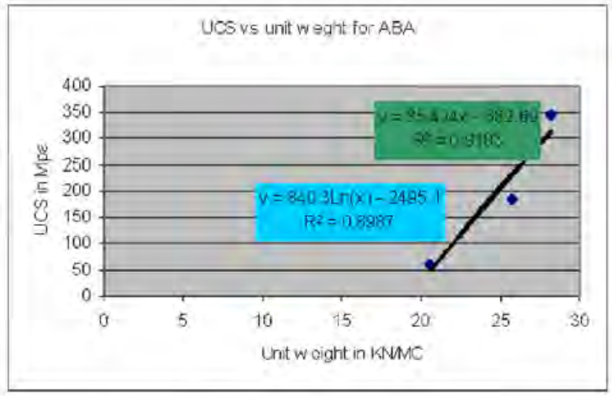
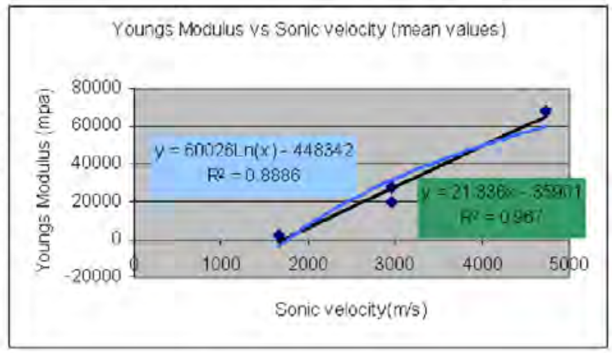


Fig. 6.1(a) Correlation between intact rock properties

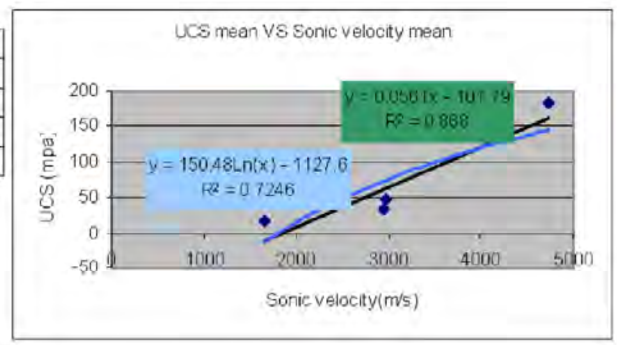
	UW	UCS
ABA	20.6	61.7
	25.75	183.6
	28.17	346.4



	SV(mean)	YM(mean)
ABA	4732	67755
VB	2952	28050
BB	2960	19959
TT	1651	2964



	SV(mean)	UCS(mean)
ABA	4732	183.6
VB	2952	34.3
BB	2960	47
TT	1651	17.9



	sv(max)	ucs(max)
ABA	5550	346.4
VB	3849	52.9
BB	4340	105.8
TT	2383	39.6

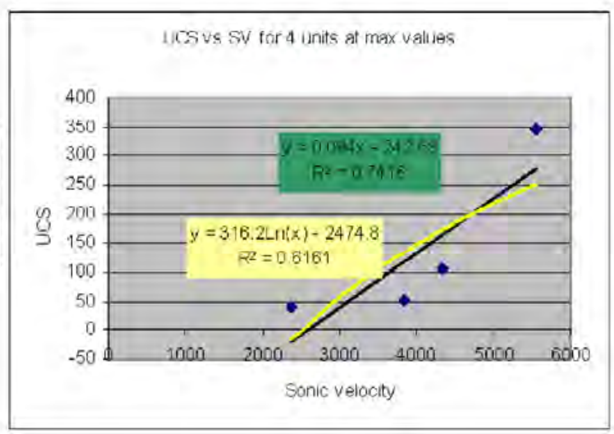


Fig. 6.1(b) Correlation between intact rock properties

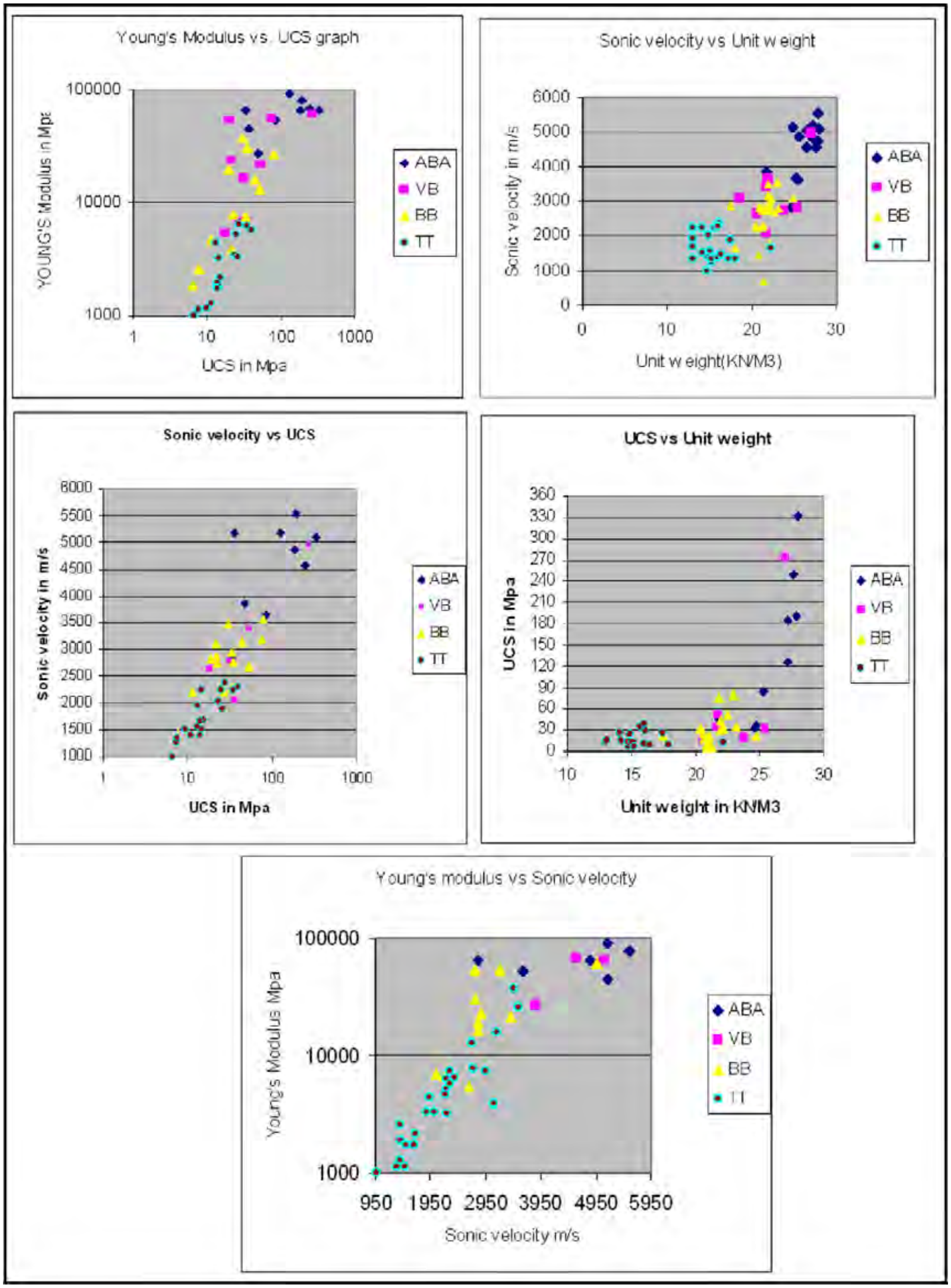


Fig. 6.2 Correlations between different parameters from laboratory results

6.2.2 Rock Mass Characteristics

Most of the engineering structures like dams, tunnels, underground powerhouse, roads etc. involve a large volume of rock mass. Rock mass contains discontinuity planes in the form of joints, fault planes, bedding plane and foliations which are responsible to control the strength and deformability of the rock mass. When the design engineer and the engineering geologist are confronted with rock, they must visualize the rock mass as an assemblage of intact rock blocks separated by different types of geological discontinuities. They must therefore consider the characteristics of the intact rock material and the discontinuities.

Geotechnical study provides information that can help to select appropriate excavation method, input data for powerhouse cavity design, identify potentional risk assessment with the excavation methods, support requirements, and enables to envisage the project finance. The procedure of interpreting site conditions and giving values for the various features of the rock mass can be called site characterization and should not be confused with rock mass classification.

Rock mass classification systems are design tools, which are used in conjunction with the engineering assessment and other design approaches. Rock mass characterization consists of quantifying the parameters governing rock mass behavior. These properties can be as intact rock characteristics, discontinuity (joint characteristics), and the density and pattern of discontinuities. The approach can be in phases of data collection, site characterization, modeling analysis and design. The main elements of site characterization are summarized in Fig 6.5.

It is important to note that the importance of the properties of the intact rock material will be generally over showed by the properties of the discontinuities in the rock masses. However, this does not mean that the properties of intact rock material should be disregarded when considering the behavior of jointed rock masses.

After all if discontinuities are widely spaced or if the intact rock is weak and altered the properties may strongly influence the gross behavior of the rock masses. Furthermore, a sample of rock material some times represents a small-scale model of the rock mass since they have gone through the same geological cycle. Nevertheless, in general, the properties of the discontinuities are of greater importance than the intact rock material.

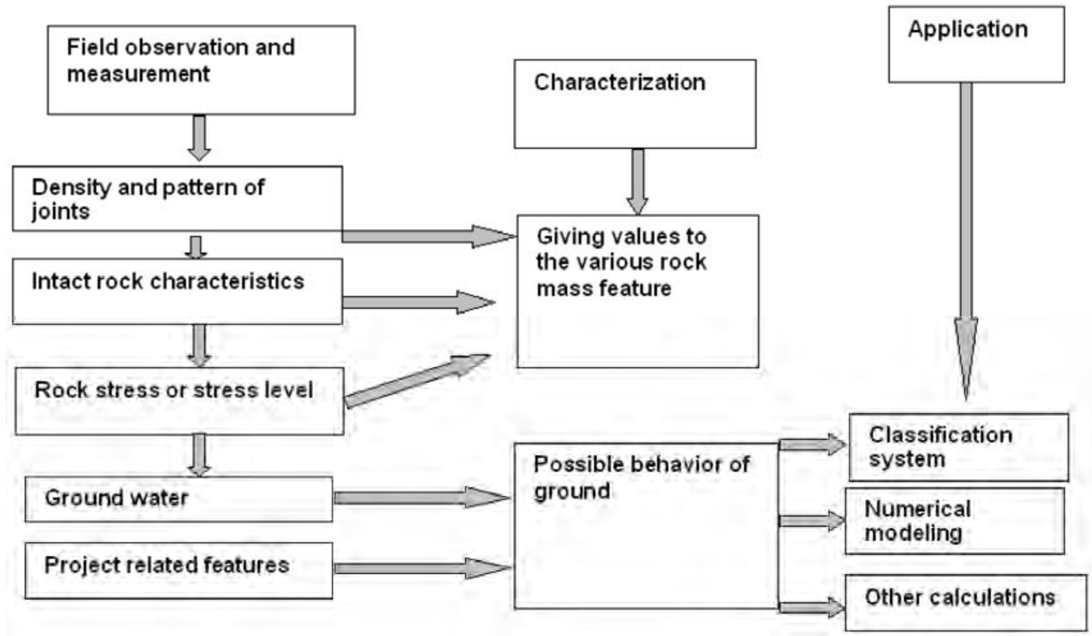


Fig 6.5 Site characterization (Palmstorm, 2000)

There appears to be no single parameter or index that can fully and quantitatively describe a jointed rock mass for engineering purposes. Various parameters have different significance, and only if taken together can they describe a rock mass satisfactorily. It is accepted that in the surface and near surface underground excavations the rock mass classification parameters are important to classify and characterize the rock mass as discussed in the previous chapter:

6.2.2.1 Rock Mass Characteristics of the Powerhouse Complex

Data for rock mass classification and characterization is collected on the following possible excavation faces (lay out in figure 6.6).

(i) Upstream face of the powerhouse cavern and SE face (breccia rock unit)
(ii) Downstream face of the powerhouse cavern on pillar and galleries wall and front faces following continuous exactions on aphanitic and breccia basalt units.
(iii) Access audits to the powerhouse cavern on aphanitic rock unit.
(iv) On access audits to the irrigation by pass and draft tubes on aphanitic rock unit.
(v) On upstream draft tube junctions of 1 & 2 on both rock units following excavation.
(vi) On draft tube faces and walls of 4 & 3 on both units following excavation.

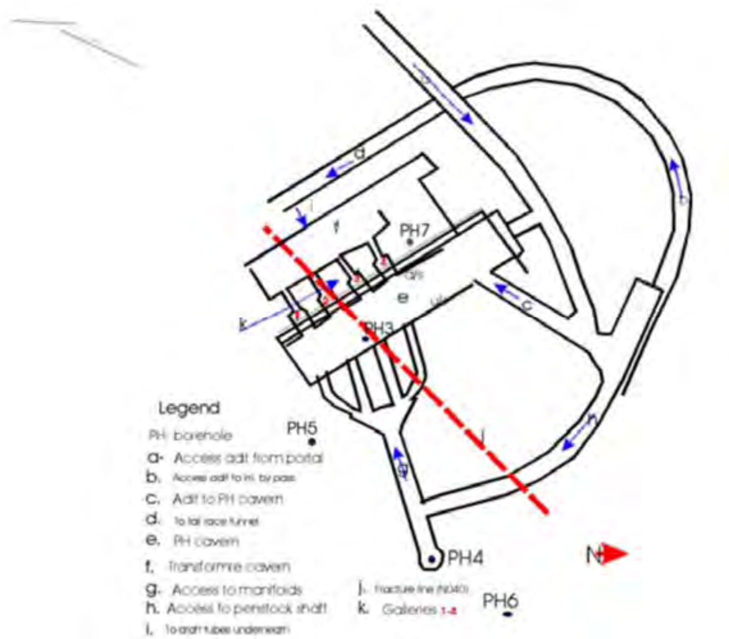


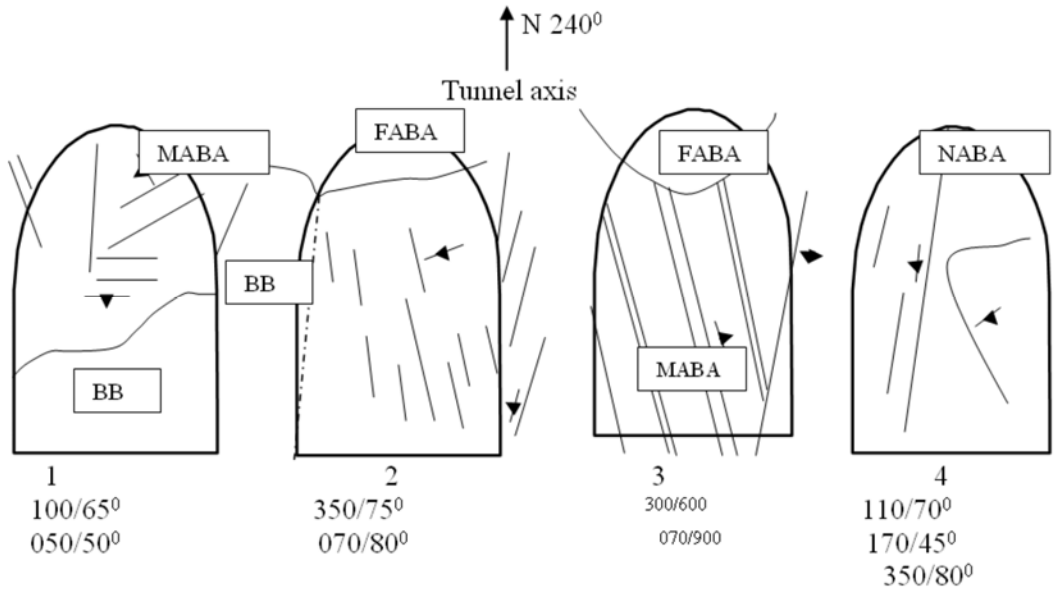
Fig. 6.6 Layout for the components of the powerhouse complex

The data collected and the observations made reveal that on most of the excavated faces the aphanitic basalt rock unit is fresh to slightly weathered but more affected by joints mostly 3 joint sets. Joints mostly strike through out the exposed faces and joint surfaces are smooth and planar with undulations (waviness) at meter scale. The major joints are the vertical columnar. This rock unit is more susceptible to fracturing during drilling and blasting. Some times the vertical and sub vertical joints are fully and partially filled by secondary minerals (calcites and zeolites) mostly non weathered and hard. The RQD and UCS values show variation from 75 – 95, (average 83.5) and 51 -167.7 (average (101) Mpa respectively. This may be due to the fracturing effect, jointing and slight weathering.

This rock unit occupied most part of the down stream powerhouse cavern, all portion of the access audit to the irrigation bypass and draft tubes and the bottom portion of access audit to the powerhouse cavern.

On the other hand, the breccia rock unit is slightly to moderately weathered but less affected by joints mostly 2 joint sets plus random. The joint surfaces are mostly rough. It is less susceptible to fracturing. On most of the exposures the RQD value is high (greater than 90%). The RQD and UCS values show very low variation ranges from 90-98 (average 95) and 40-48 (average 43) Mpa respectively. The high RQD value is due to less degree of jointing while the low UCS value is by the weathering effect and the very nature of the rock unit. Most part of the upstream and SE part of the powerhouse cavern is occupied by this rock unit.

The face maps in Fig.6.7 and plates in 1,2 &3 can be used as an additional demonstration tools for the facts discussed above.



BB-Breccia, FABA- Fractured aphanitic basalt, MABA- Massive aphanitic basalt

Fig. 6.7(a) Face maps on the 4 bus duct galleries mapped on the same day at 1465 m elevation

Following the identified N 040^0 trending weak structure, the rock mass is fractured and disintegrated. There was relatively unstable rock mass condition faced when the crane beam structure was on construction. It was through this structure water was gushing out during exaction and is still seeping out. It is also by this structural effect that the upstream wall of the powerhouse cavern is wet and damp even after the support measures are put.

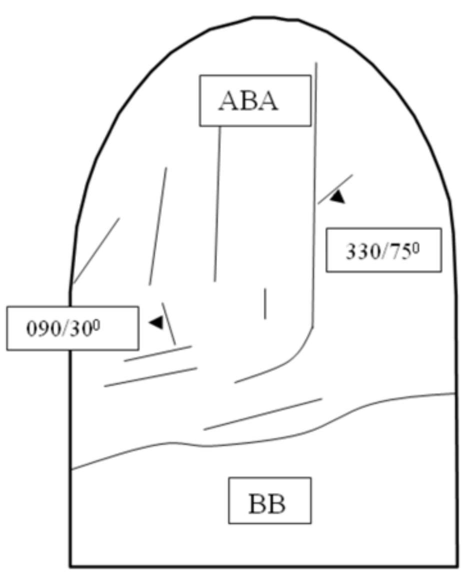
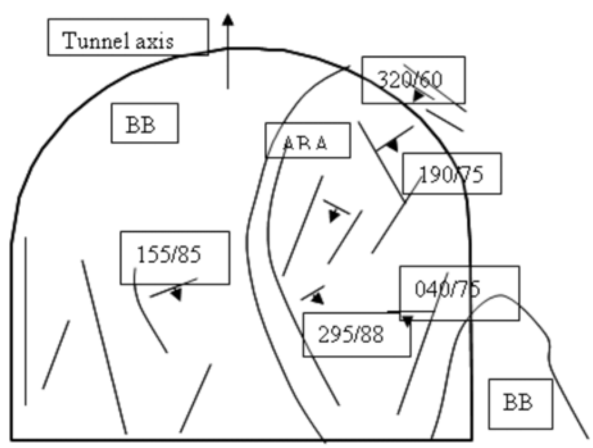


Fig.6.7 (b) Face on bus duct gallery unit 1

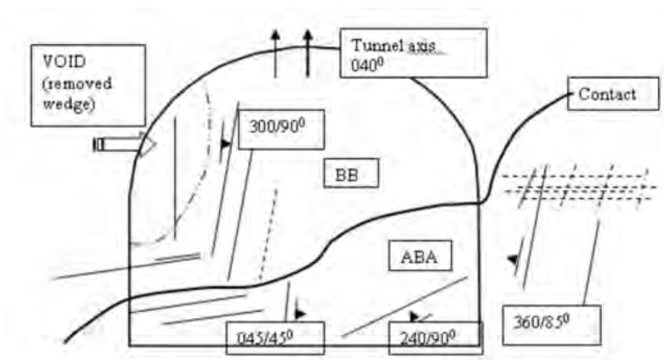


ABA- Slightly weathered -fresh, moderately fractured, sliced with closely spaced joints. BB-slightly-moderately weathered, dark greenish and calcified breccia

Fig.6.7(c) face map on adit to the upstream draft tube junction 1 & 2

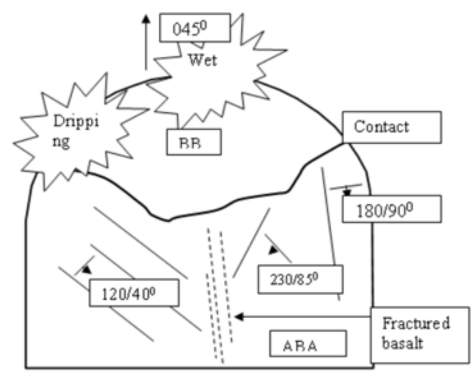
With the construction progress, the unsafe rock mass condition with respect to strength and water tightness aspects may continue to face on few specific locations as it is clearly observed on excavation faces of bus duct galleries unit 2 & 3. On these two units the rock mass is highly disintegrated (fractured) even to such a degree that on some specific parts of the face it was difficult to rate the rock mass using rock classification system

Such effects were also faced on excavation progress towards draft tube junction Unit 1& 2. Formation of few rock wedges was part of the effect following fresh excavations. On this structural alignment up to 30 cm wide cavities are formed on upstream powerhouse cavern. These cavities are partially and fully filled by weathered calcite infillings and water is continuously seeping out.



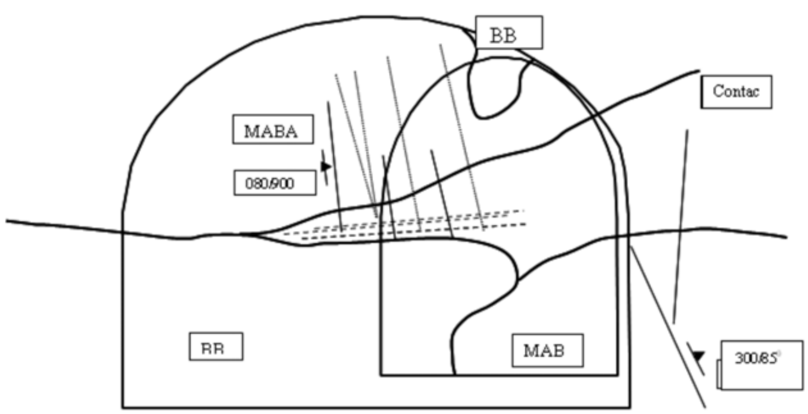
AA-Aphanitic basalt, fractured and relatively less jointed
ABA-Sliced and jointed Aphanitic basalt disturbed by fracturing

Fig.6.7(d)Upstream draft tube junction of unit 1 & 2



AA-Aphanitic basalt, fractured and relatively less jointed
BB-Clarified and moderately to slightly weathered ,greenish-grey breccia

Fig.6.7(e)Face on draft tube unit 4



MABA-Massive aphanitic basalt , FABA-Fractured aphanitic basalt, BB-Brecciated basalt

Fig.6.7 (f) Face map on draft tube 1 & 2

To strengthen the discussion on rock mass characterization, plates 1, 2 and 3 are attached for rock feature demonstration on excavation faces.


Two joint sets with smooth joint surface in Aphanitic basalt


Breccia unit- no defined discontinuities


Breccia basalt

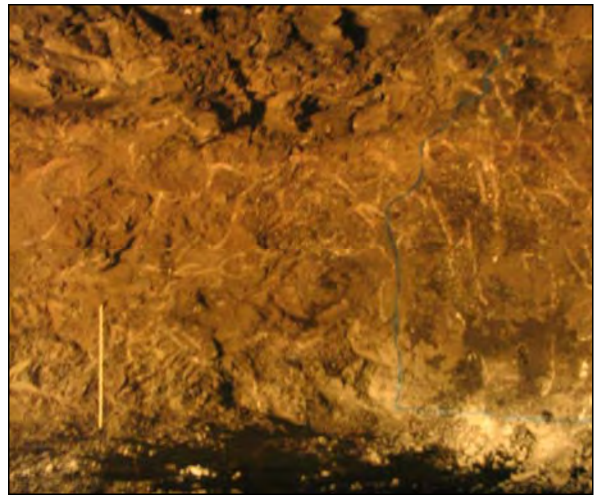

Fractured Aphanitic basalt in a bus duct gallery

Weathered and fractured breccia basalt on fracture line u/s PH cavern wall


Slightly weathered breccia with rough surface (no considerable joint sets)

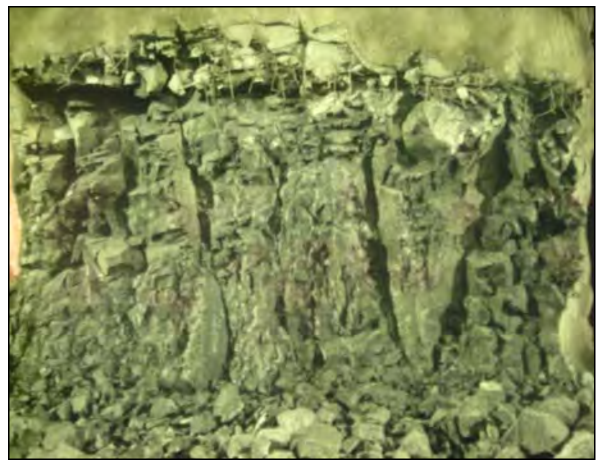
Plate 1 Rock features, as observed in powerhouse cavity

Aphanitic and breccia rock unit with well defined contact

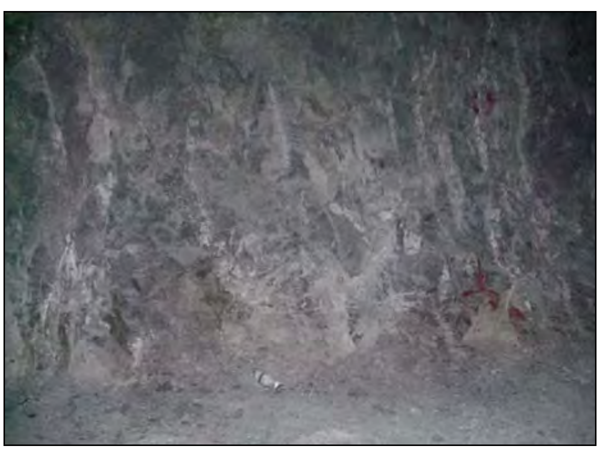


Breccia rock unit less fractured

Fracture line on weathered breccia rock unit



Non weathered, moderately fracture aphanitic rock unit with vertical joints on a pillar section



Massive breccia unit (no pronounced joints)

Weathered and fractured breccia unit on draft tubes

Plate 2 Rock features, as observed in powerhouse cavity

Massive breccia unit (no joint sets)

Aphanitic basalt having more than 3 joint sets and smooth joint surface

Calcite in filled cavity on moderately wreathed breccia (rough joint surface)

Plate 3 Rock features, as observed in powerhouse cavity

6.2.3 Application of rock mass classification in determining strength of jointed rock mass

Hoek and Brown (1980) developed an empirical approach to determine the strength of the jointed rock mass and formulated failure criteria. Based on the results of a number of projects, this criterion was modified by Hoek and Brown in 1988 and later by Hoek et al. (1992). For this empirical relation they used Beniawski's rock mass rating system (RMR) to work out material constants. Rock mass strength, has been defined through the geological strength index (GSI) proposed by Hoek (2003-2005). GSI value is calculated from Beniawski RMR.

$\delta'1=\delta'3+\delta ci\ (mb*\delta'3/\ \delta ci+S)^{a}$ …….eq.6.1

Mb = mi exp ((GSI-100) / (28-14D)) …….eq.6.2

S= exp (GSI-100)/ (9-3D) …….eq.6.3

a= ½+1/6($e^{-GSI/15}$- $e^{-20/3}$) …….eq.6.4

‘s’ and ‘a’ are constants related to GSI

mb = value for material constant ‘mi’

D = parameter depending upon the excavation mode, which is considered equal to 0.

δci = the unconfined compressive strength.

‘δ’1’ & ‘δ’3’ are minimum and maximum effective stresses.

The geologic strength index (GSI) is determined by the following relation (Hoek and Brown)

For, RMR_{76} > 18, GSI = RMR_{76} …….eq.6.5

RMR_{89} > 23, GSI = RMR_{89} –5 …….eq.6.6

For GSI > 25 (undisturbed)

a = 0.5

For GSI < 25

S = 0, a = 0.65 – (GSI /200)

Table 6.8 Rock mass parameters determined by the Construction Company

Unit	GSI	UCS(Mpa)	mi	mb	S	a	c	ϕ	δ_{cm}	E(Gpa)
ABA	65	150	25	7.1	0.0205	0.502	2.76	45^0	55.4	30
BB+ VB	53	-	28	2.8	0.0054	0.505	0.92	45	7.9	10
ABA Aphanitic basalt					BB - Breccia basalt		VB – Vesicular basalt			

Table 6.9 Rock mass parameters determined using the above equations based on the average values of UCS and RMR collected in the field work

Unit	UCS (Mpa)	RMR	GSI	mi	mb	s	a	C (Mpa)	ϕ(^{0})	E (Gpa)
ABA	119	68.5	63.5	17	4.6	0.017	0.5	3.42	39.25	23.73
	119.69	65.2	60.2	17	4.10	0.012	0.5	3.26	37.6	19.67
	94.65	63.97	58.97	17	3.93	0.010	0.5	3.19	36.98	16.30
	109.14	67.25	62.25	17	4.42	0.015	0.5	3.36	38.62	21.14
BB	46	65	60	18	4.314	0.0117	0.5	3.25	37.5	12
	48.18	63.4	58.4	18	4.074	0.0098	0.5	3.17	36.7	11.26
	55.7	68.75	63.75	18	4.93	0.0178	0.5	3.43	39.37	16.46
ABA - Aphanitic basalt, BB - Breccia basalt UCS – Uniaxial compressive strength RMR – Rock mass rating GSI – Geological strength index							mi, mb,s and a – material constants C – Cohesion, ϕ - angle of friction E - deformation modulus			

The estimation of constants mb/mi, s, a, deformation modulus (E), the Poison’s ratio (v) and rock mass surface conditions can also be visualized from the chart represented by Hoek, Marinose Benissi (1998) for the generalized Hoek-Brown failure criteria based upon rock mass structure and discontinuity surface conditions. Based on the calculated values the rock mass fall in good and fair surface conditions as can be seen on the chart shown at Annexure ‘E’.

Note that in Table 6.9 the ‘mi’ constant is taken 17 and 18 for the two rock units from literature values, but the one taken into consideration by the construction company is 25 and 28 higher than literature values. Also there is some variation on s, c,ϕ, UCS, and E values. However, for the general analysis the company has employed conservative approach considering only the Breccia rock unit parameter; Cohesion ‘C’ = 1.5Mpa, Angle of friction ‘ϕ’ = $40,^0$ Geologic Strength Index ‘GSI’ = 55 and even angle of friction ‘ϕ’ = 30^0 for crane beam foundation.

The shear strength parameters can be determined from Bieniawiski’s rock mass rating system ‘RMR’ or by Mohr Coulomb envelope. The parabolic Hoek and Brown criteria has been converted in to the linear Mohr-Coulomb criteria with linear interpolation or considering the tangent to the curve at specific stress values to determine the shear strength parameters (c & ϕ). In Table 6.8 the shear strength parameters have been determined from direct shear tests while in Table 6.9 these are estimated from the following relations from RMR (Beniawski).

Cohesion ‘C’ = 0.05 RMR …….eq.6.7

Angle of internal friction ‘ϕ’ = 0.5 RMR +5 …….eq.6.8

There is no direct correlation between the linear Mohr-Coulomb Criterion and the nonlinear Hoek-Brown Criterion. Attempts have been made by Hoek and Brown to estimate ‘c’ and ‘φ’ from the Hoek-Brown equation but it is a major problem to obtain ‘c’ and ‘φ’ from the Hoek-Brown equation. If a series tests have been conducted on the rock mass, obviously test results should be used directly to obtain parameters ‘c’ and ‘φ’, using for example, plotting the Mohr circle and fitting with the best strength envelope, where ‘c’ and ‘φ’ can be readily calculated. Common problems were there is no or limited test results on rock mass. The suggested approach to obtain rock mass Mohr-Coulomb parameters ‘c’ and ‘φ’ is by generating a series of $\sigma 1$–$\sigma 3$ results by the Hoek-Brown equation. Then plotting the Mohr circle using the generated $\sigma 1$–$\sigma 3$ data and fitting with the best linear envelope, where ‘c’ and ‘φ’ can be readily calculated.

Shear strength parameters in Table 6.10 are determined from Mohr coulomb envelops (Fig.6.8) constructed by using the principal stresses determined from the rock mass parameters of field data by eq. 6.9.

$$\delta'1=\delta'3+\delta ci\ (mb*\delta'3/\ \delta ci\ +S)^{a} \qquad \text{…….eq.6.9}$$

The maximum value of δ'3 is considered to be one fourth of the UCS value and then it is decreasing to calculate the respective δ'1 values. The shear strength parameter determined from RMR value and from Mohr Coulomb envelops are approximately equal.

Table 6.10 Values for shear strength parameters for data taken on specific locations

Aphanitic basalt																
UCS (δci) (Mpa)	**119 (adit to irrigation by pas)**				**119.69 (Pillar and Galleries)**				**94.659 (Draft tubes)**				**109.14 (Adit to Powerhouse cavern)**			
δ'1	158	126	107	85	152	120	102	81	118	90	72	49	143	119	101	80
δ'3	29	20	15	10	30	20	15	10	23	15	10	5	27	20	15	10
C (MPa)	8				5				5.5				4.5			
$\phi(^0)$	39				39				38				39			
Breccia basalt																
UCS (δci)	46				48.18				55.7							
δ'1	59	37	23	16	62	42	28	16	76	51	33	19				
δ'3	11	5	2	1	12	6	3	1	14	7	3	1				
C (Mpa)	3.5				3.5				3.5							
$\boldsymbol{\phi(^0)}$	37				35				39							

6.2.4 Modulus of Deformation of Rock mass

Rock mass deformability Modulus can be determined by using rock mass rating (RMR) from various empirical relations.

Hoek (1983) relation for modulus of deformation;

$$Em\ (Gpa)= (1-D/2)*10^{((GSI-10)/40)} \qquad \text{…….eq.6.10}$$

Beniawski's relation (1978) for modulus of deformation;

$$Ed = 2\ RMR - 100 \text{ for } RMR > 55 \text{ and } UCS > 100 \text{ Mpa} \qquad \text{…….eq.6.11}$$

Agarwal et al., relation (1991) for modulus of deformation;

$$Ed = 10^{(RMR-30)/50} \qquad \text{…….eq.6.12}$$

Hoek and Brown (1977) relation for modulus of deformation;

$$Ed = (qc)^{0.5}/10 * 10^{(GSI-10)/40} \quad qc=UCS, \qquad \text{…….eq.6.13}$$

From the above relations, the modulus of deformation determined by Hoek and Hoek and Brownn(1977) are almost equal.. The modulus of deformation values presented in Table 6.9

are calculated by using Hoek and Brown relation (1977). Beniawski's relation will not be applicable here because the UCS values of rocks are mostly less than 100Mpa. The values determined by and Agarwal are also lower values. The modulus of deformation value determined in table 6.8 was stated to be determined by Hoek's relation. However, it does not agree with the formula that the calculated result should be 24 Gpa not 30 Gpa.

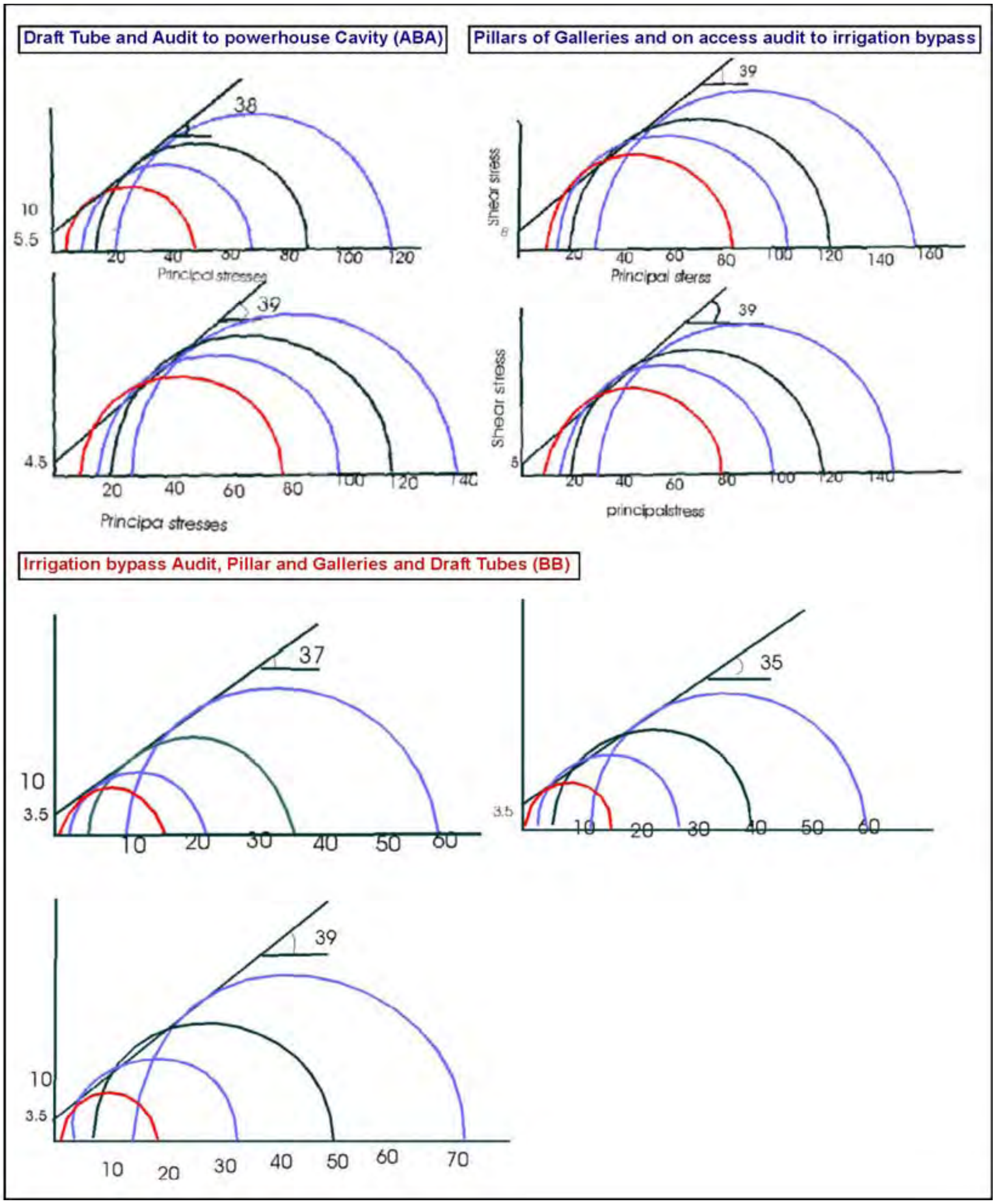


Fig.6.8 Determination of Shear Strength Parameters for rock mass at different localities

6.2.5 Joint analysis from exposures

To apply suitable classification on rock mechanics, fieldwork must provide basic data on characteristics of discontinuities in rock mass (Bieniawski, 1976, Bosscher and Connel, 1988). Preferred joint orientation, which can introduce anisotropy in permeability and strength, has been detected through the independent and random measurements of joint orientations. Detection of preferred orientation is of particular importance as it influences the rock mass property and can affect stability. Analysis of preferred joint orientation analysis is an important input for stability analysis and to determine the mode of failures. For this study it was very difficult to collect sufficient amount of joint data to carry out the analysis for the determination of proffered orientations. Shotcrete layer has covered most of the exposed rock face within the powerhouse cavity as a result rock is exposed at random places. Thus, the joint orientation data has been collected on random basis. With this limitation, Stereonet pole projection is carried out for the dip direction and dip amount of the collected data. In conjunction, the concentration contours and their representative great circles are plotted (Fig.6.7) to determine the preferred joint orientations (Table 6.11).

Table 6.11 Representative sets of the joints for the two units based on their orientation

Unit	Joint Set	Orientation	Percentage (%)	Location	Remarks
ABA	Set1	152/84^0	27.7	Access adit to irrigation by	-
	Set2	182/86^0	16		
	Set3	270/88^0	13.6		
	Set4	292/88^0	11.36		
	Set5	245/86^0	9		
	Se6	046/85^0	9		
	Set7	062/60^0	7		
ABA	Set1	190/85^0	21.4	Pillar and gallery faces	-
	Set2	350/85^0	17.8		
	Set3	308/88^0	14.28		
	Set4	115/49^0	12.5		
	Set5	065/62^0	12.5		
	Se6	239/88^0	10.7		
	Set7	279/78^0	7		
BB	Set1	026/82^0	40	Draft tubes (1&2 junction,3and 4)	-
	Set2	302/86^0	23.33		
	Set3	145/85^0	20		
	Set4	115/88^0	10		
	Set5	182/60^0	6.7		
BB	Set1	040/76^0	25	Pillar and gallery	Joints have less frequency and equal distribution
	Set2	330/60^0	16.7		
	Set3	103/86^0	16.7		
	Set4	060/62^0	16.7		
	Set5	153/75^0	16.7		
	Set6	151/42^0	16.7		
	Set7	302/58^0	16.7		

In addition to joint orientation, measurement is taken for joint spacing, continuity, separation and roughness conditions as presented in Table 6.12.

Joint spacing and condition of joints such as opening or separation of joints (distance between joint surfaces), continuity and roughness of joints as well as joint fillings was critically assessed. The continuity, spacing and joint filling influence the behavior of the rock mass.

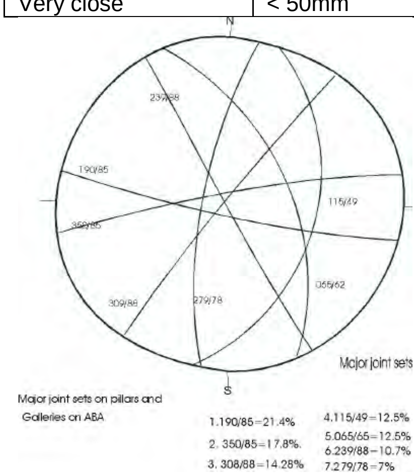
In the inspected portion of the cavity, the spacing of most joints range from less than half a meter to 1- 2m, which are almost closely spaced (Table 6.11). At most places, the joints are impregnated with secondary material particularly in brecciated basalt. Some of the joints are totally or partially filled by secondary minerals like calcite infillings. In both units joints are tight and the surfaces are almost rough for Breccia unit and smooth for Aphanitic basalt. Waviness and roughness characteristics are clearly manifested on the aphanitic and brecciated units, respectively. Most of the major joints are steeply dipping (75 to 88^0) (Table 6.11). According to Deere's joint spacing classification (table 6.13), most joints are moderately close resulting a rock mass grading of blocky/ seamy.

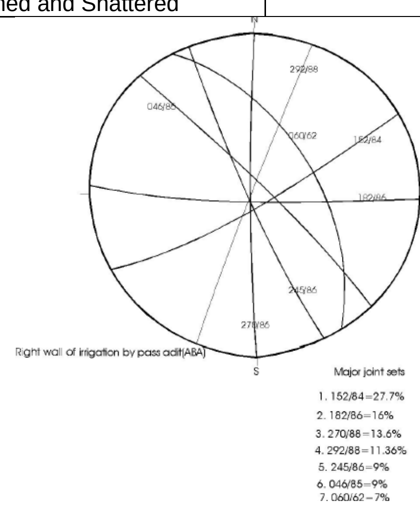
Table 6.12 Average values of some joint parameters according to the field data

Location	Unit	Spacing(m)	Length (m)	Separation	Roughness	Remark
Data on 16 faces	ABA	0.23 -0.52 1 - 2 > 3	1 - 3 > 3	< 1mm 1 -5 mm 1 – 3 cm – (few)	Smooth	Most joints are closely spaced
Data on 6 faces	BB	0.26 - 1 1 - 2 >3	1 - 3 > 3 (few)	< 1mm (few) 1 – 5mm	Rough	

Table 6.13 Deere's classification for joint spacing

Description	**Spacing of joints**	**Rock mass grading**	**Remark (Table6.12)**
Very wide	> 3m	Solid	Few joints (5%)
Wide	1 – 3m	Massive	20 %
Moderately close	0.3 – 1m	Blocky/ Seamy	Most joints (75%)
Close	50 – 300mm	Fractured	Specific location
Very close	< 50mm	Crushed and Shattered	





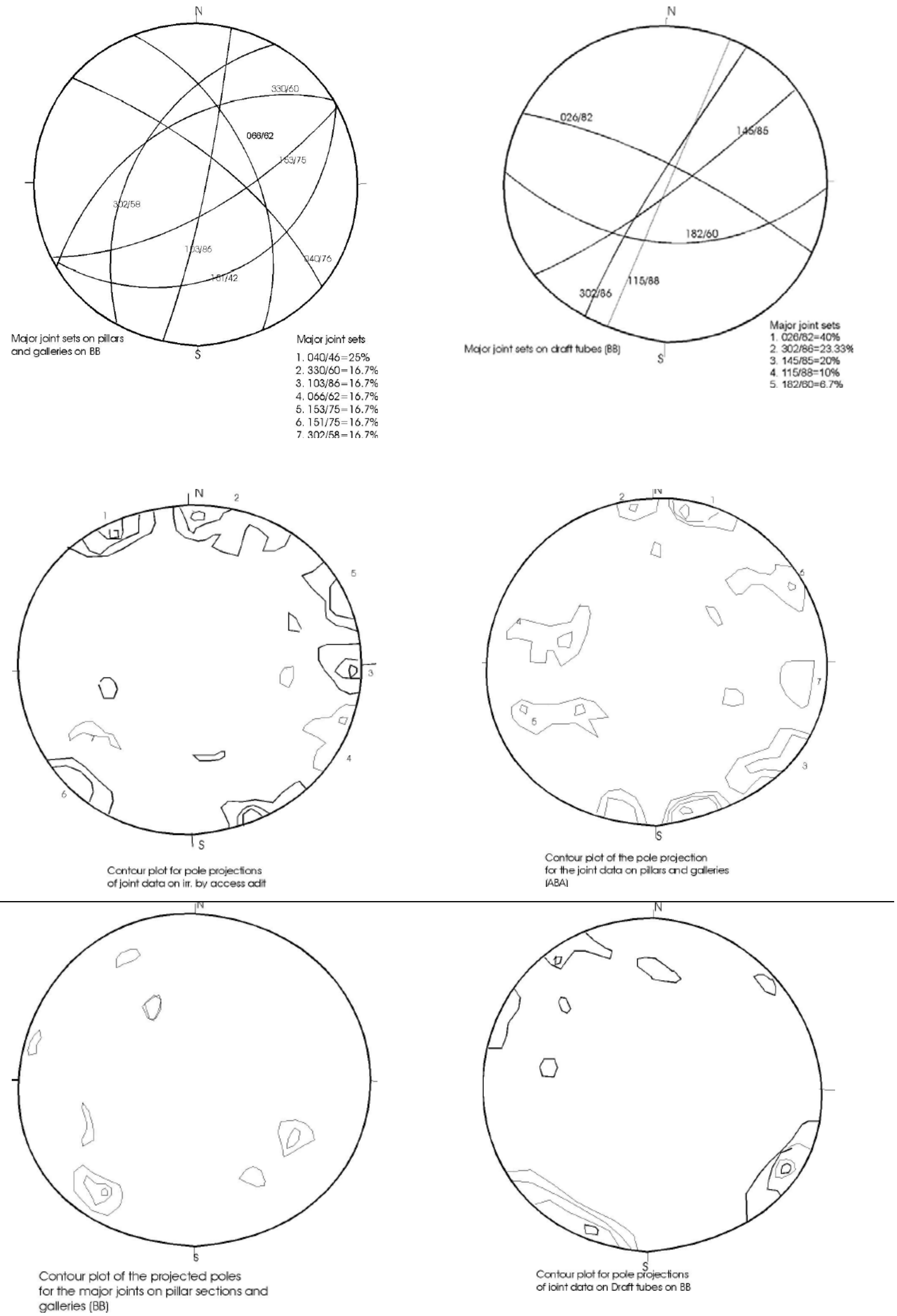


Fig. 6.9 **Stereonet contour plots and preferred orientations of the major joint sets on three representative excavation faces**

CHAPTER VII REVIEW OF STABILITY AND SUPPORT

7.1 Introduction

The increase in size of underground mining operations during the last few decades has led to the introduction of a concept of permanent underground excavations. In addition to being large in size, the excavations can house expensive equipment; consequently they must be secured against rock falls and other forms of instabilities. Civil engineers are mostly concerned for permanent underground powerhouse excavations and caverns for the storage of oil or gas are required to remain stable for periods in excess of 20 years.

Because any form of instability can not be tolerated, the resources allocated to the design and installation of support systems is normally adequate and, sometimes even generous. The design of under ground excavation is, to a large extent the design of underground support systems. These can range from no support in the case of temporary excavations in good rock to the use of fully grouted and tensioned bolts or cables with mesh and sprayed concrete for the support of large permanent civil engineering excavations (upper bound of under ground support design).

For a constant set of geological conditions, the cost of support design (including geological investigations, support design and installation) will be related to the type and size of excavation categories. Given the task of designing underground excavation, steps should be followed from the determination of the type of excavation methods and control of the excavation operation to characterize the rock mass. The basic aim of under ground excavation design should be to utilize the rock it self as a principal structural material, creating as little disturbances as possible during the excavation process and adding as little as possible on the way of concrete or steel support. In their intact state and subjected to compressive stresses, most hard rocks are far stronger than concrete and many are of the same order of strength as steel. Consequently it does not make economic sense to replace a material, which may be perfectly adequate with one which may be no better. Hence, an accurate interpretation of the geology is an essential prerequisite to a rational design. Based on the interpretation analysis stability problems are identified (Hoek, 1999).

The principal sources of instability are;

(i) Instability due to adverse structural geology tends to occur in hard rocks, which are faulted, jointed and where several sets of discontinuities are steeply inclined.

(ii) Instability due to excessive high rock stress is also generally associated with hard rock and can occur when excavation at great depth or where very large excavations are created at reasonable shallow depth.

(iii) Instability due to weathering and /or swelling is generally associated with relatively poor rock but it may also occur in isolated seams with an otherwise sound hard rock.

(iv) Instability due excessive ground water pressure or flow can occur in almost any rock mass but it would normally only reach series portions if associated with one of the other forms of instabilities.

Generally, it is good to have engineering judgment about instabilities because a rock mass is a complex assemblage of different materials and its behavior does not approach to the simple models which engineers and geologists have to construct (Hoek and Brown, 1980).

7.2 In situ Stresses

Rock at depth is subjected to stresses resulting from the weight of the overlying strata and from locked in stresses of tectonic origin. When an opening is excavated in this rock, the stress field is locally disrupted and a new set of stresses is induced in the rock surrounding the opening. Knowledge of the magnitudes and directions of these in-situ and induced stresses is an essential component of underground excavation design since, in many cases, the strength of the rock is exceeded and the resulting instability can have serious consequences on the behavior of the excavations.

At a point below the rock surface of the undisturbed rock mass, there are stress due to the weight of the overlying materials, and also due to the confinement and past history. These stresses are known as in-situ stresses. The in-situ stresses vary considerably from one point to the other, they may be almost zero at some points, where as at some other points, they may be very high, even approaching the failure stress. When the in-situ stress state is disturbed by the construction of excavations, earthworks and engineering structures produces a new distribution of stresses these stresses are known as induced stresses. The in situ stresses include components: (i) Gravitational stress due to weight to overlying rock and (ii) The effect of latent stresses.

For intact rocks the walls will not fail until the circumferential stress equals the unconfined compressive strength (UCS), of the rock. And in rocks that are not affected by tectonic forces, the circumferential stress does not exceed approximately the twice the overburden pressure; hence failure of crushing can occur only at considerable depth. In very strong rocks, the critical depth is 35,000 ft. (John R. Schultz et al. 1955).

The in-situ stresses may be approximated and the order of their magnitude determined by various methods but the accuracy of estimating is always doubtful. Accurate field measurements are always required for ascertain the margin of error. Field measurements of in situ stress are quite common in the mining practice, however, in the civil engineering practice, the field measurements are rarely done because of high cost of measurements (Hoek, 1999).

For the present study in- situ stresses are determined using the empirical relations since there has been no instrumental data. The overlying rock mass stress is the function of the depth at average unit weight of the rock. This can be estimated from the relation given by eq.7.1.

$\delta v = \gamma z$ ………eq.7.1

Where 'δv' is the vertical stress; 'γ' is the unit weight of the overlying rock and 'z' is the depth below surface.

The mean unit weight of the breccia unit is 22 KN/ m^3. Therefore, vertical stress is estimated to be about 6380 KN/m^2 (12.47Mpa) corresponding to 290m over burden at the powerhouse elevation.

To calculate the horizontal stress, the ratio of the horizontal stress to the vertical stress should be determined. The empirical relation with Poison's ratio (ν), $k= \nu/(1-\nu)$ (Hoek et al., 1995) has been used to calculate the magnitude of the coefficient of geostatic pressure (k). The value of Poison's ratio is determined to be 0.3 from the laboratory test. Thus the value of k will be: (0.3/91-0.3) = 0.43.

Therefore, the horizontal stress will be: $\delta h = k\delta v = k * \gamma * z = 0.43*22$ KN/m^3*290m = 2743.4 KN/m^2 = 5.36Mpa.

However, measurements of vertical stress at various mining and civil engineering sites around the world confirm that this relationship (given by equation 7.1) is valid though there is a significant amount of scatter in the measurements (Hoek, 1999).

The horizontal stresses acting on an element of rock at a depth '*z*' below the surface is much more difficult to estimate than the vertical stresses. Normally, the ratio of the average horizontal stress to the vertical stress is denoted by the letter '*k*' such that:

$\sigma h = k\sigma v = k \gamma z$ …….eq. 7.2

Terzaghi and Richart (1952) suggested that, for a gravitationally loaded rock mass in which no lateral strain was permitted during formation of the overlying strata, the value of *k* is independent of depth and is given by $k = v\ (1 - v)$, where v is the Poisson's ratio of the rock mass. This relationship was widely used in the early days of rock mechanics however, as discussed below; it proved to be inaccurate and is seldom used today.

Measurements of horizontal stresses at civil and mining sites around the world show that the ratio '*k*' tends to be high (K >1) at shallow depth (1000m) and that it decreases at depth (Brown and Hoek, 1978, Herget, 1988). In order to understand the reason for these horizontal stress variations it is necessary to consider the problem on a much larger scale than that of a single site.

Sheorey (1994) developed an elasto-static thermal stress model of the earth. This model considers curvature of the crust and variation of elastic constants, density and thermal expansion coefficients through the crust and mantle. Sheorey's equation which can be used for estimating the horizontal to vertical stress ratio *k*.

This equation is;

$$K = 0.25+7E_h(0.001+(1/Z)) \qquad \text{.......eq. 7.3}$$

where *z* (m) is the depth below surface and *Eh* (GPa) is the average deformation modulus of the upper part of the earth's crust measured in a horizontal direction. This direction of measurement is important particularly in layered sedimentary rocks, in which the deformation modulus may be significantly different in different directions.

For the present study measurements on average horizontal deformation modulus (Eh) are not available. However, through this study an attempt is made to utilize the total modulus of deformation (Ed), which has been determined empirically by using Hoek and Brown (1977) relation. The modulus of deformation (Ed) comprises of two components; modulus of vertical deformation and modulus of horizontal deformation. Thus, if the total modulus of deformation (Ed) will be used in eq. 7.3, it is presumed that the value of constant 'K' will be conservative.

By taking the depth 'z' equal to 290m (average depth of powerhouse cavity from ground surface) and taking average modulus of total deformation (Ed) equal to 13 Gpa (As determined by empirical relation given by Hoek and Brown (1977) the constant 'K' calculated by eq. 7.3 will be 0.65.

Thus, the horizontal stresses by using ‘K’ equal to 0.65 will be 8.11Mpa which is relatively better approximation though not accurate according to the practical conditions as suggested by Hoek and Brown, 1982). In general, according to the concept given by John R. Schultz et al (1955), the Tana Beles project will not be affected by serious horizontal stress disturbances.

7.3 Support proposal

The general behavior of the rock mass dictates a stable rock mass but the instability of some side walls requires a certain confining pressure to be induced during anchors.

After detail analysis is made considering different parameters and conservative safety factors for the whole dimensions of the cavities, the designers of the construction company had proposed the following permanent supports for application.

Crown of cavern

Shotcrete and rock bolts as required for protection;

(i) 15cm thickness steel fiber reinforced shotcrete or two layer of shotcrete (2*7.5cm), reinforced each layer with wire mesh ϕ 5mm, 150mm * 150mm

(ii) Length (L) = 12m , 36*12 anchors, pattern 2.8 * 2.8

(iii) Two layer of 10cm (2*10cm) shotcrete reinforced each layer with wire mesh Φ5mm, 150mm*150mm.

Side walls of the caverns

Shotcrete rock bolts as required for protection;

(i) 10cm thickness steel fibers reinforced shotcrete or two layers of shotcrete (2*5cm) reinforced each layer with wire mesh Φ5mm, 150mm*150mm.

(ii) Bolts Φ 25mm, L= 8-10m, pattern 2*2m (10m in the upper section of the wall and 8m in the lower section).

(iii) Two layers of shotcrete (5cm) reinforced with wire mesh Φ 5mm, 150mm*150mm.

(iv) It was also proposed that for temporary support bolts and fiber reinforced shotcrete will be used.

7.4 Support design for application

The application of the designed supports had agreed to be mainly based on the rock mass rating (RMR) value. The tunnel quality index (Q) system was not taken as a parameter to propose support systems and hence Q data had not been collected by the geologists. Table 7.1 gives the agreed designs to refer for application. Supports are applied referring the designs based on the geological condition and excavation diameter with close supervision of the EEPCO geologists and engineers to control the dimensions, grids, thickness and viscosities of shotcrete.

Table 7.1 Design considerations to refer for support application

(i)	Spot bolting (25mm diameter bolt) for tunnel diameter = 7.5 m, RMR>71 and for tunnel diameter = 6m , RMR > 55	=	3 - 4 m length
(ii)	Pattern bolting (25mm diameter bolt), for RMR < 71, for tunnel diameters 7.5 and 6 m and for enlarged chambers	=	3-4 m length
(iii)	Cross bars (40mm diameter), depending to be applied on pillar section when unstable	=	8-12 m length
(iv)	Cross bars (40mm diameter), depending to be applied on pillar section when unstable	=	8-12 m length
(v)	Welded mesh (150*150mm grid),	=	5mm diameter
(vi)	Shotcrete -- single layer shotcrete 7.5 cm and 15cm double layer		

Crane beam – concrete structure

The supports on the crane beam concrete structure are not based on the rock mass Characteristics, rather consideration had for the loads of the crane to be permanently installed and other parameters. Here 40 mm bolt 12m length grouted and pre-tensioned solid bars applied on 1*1 m grid on 3 types of rows depending. Type ‘A’ and ‘B’ on 2 rows, type ‘C’ on 3 row (disturbed part). These are all over excavation treatments.

Cavern roof and sides

Here hollow bars are applied. On a wall - roof - wall arrangement, there are 38 rows having 12 bolts on each row. The shotcrete thickness on cavern roof is 40cm. While on the wall the shotcrete thickness is 20cm and the rock bolts are 3,6,10 and12 m lengths based on the actual condition.

Pillar and gallery sections

Different lengths of cross bolts with shotcrete and wire mesh are applied. The welded mesh and shotcrete are applied together on most faces; 150*150mm grid 5mm diameter with shotcrete. Two layers: single layer shotcrete 7.5 cm and 15cm double layer. The single layer is applied for primary support on access adits and working chambers where required. The shotcrete can be with or without steel fibers. The double layer shotcrete is applied with wire mesh in between i. e. two layers 7.5 cm shotcrete before and after wire mesh. The application of both steel fiber shotcrete with welded mesh is termed as double reinforcement.

Access adits

Rock bolts 2 - 4 m length are applied on unsafe access adits. The adit from portal to powerhouse (1.8km) is by single layer shotcrete where the rock units are weathered and fractured. However, at majority portions this adit and the access adit to the irrigation by pass and draft tubes is not treated since it is stable. The rest of the portion: the adits to the penstock shaft and powerhouse cavern and the enlarged section which provides access to these adits are treated with spot bolts and single layer shotcrete as required.

7.5 Recommended Support Measures Based On Rock Mass Classification And Joint Analysis

Rock mass classification systems can be utilized to work out the support design for the underground openings. For this purpose Bieniawski's Geomechanics Classification (RMR System) or Barton's `Q' system can be utilized. However, the Barton's `Q' system is considered to be the best for the support design in underground openings.

The support types and their application densities vary for different rock classes and excavation dimensions (spans). Though these rock mass classification systems can not be used as support treatment guidelines for the powerhouse cavern, they can be applicable for treatment of adits, galleries and draft tubes which have spans of less than 10 m (Table 7.2).

Table 7.2 Guidelines for excavation and support of 10 m span rock tunnels in accordance with the RMR system (After Bieniawski 1989).

Rock Mass Class	Excavation	Rock bolts (20 mm dia, fully grouted)	Shotcrete	Steel sets
I – Very Good Rock RMR: 81-100	Full face, 3 m advance	Generally no support required except spot bolting	Generally no support is required	Generally no support is required
II – Good Rock RMR: 61-80	Full face, 1-1.5 m advance complete support 20 m from face	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire	50 mm in crown where required	None

		mesh		
III – Fair Rock RMR: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast complete support 10 m from face	Systematic bolts 4 m long spaced 1.5 – 2m in crown & walls with wire mesh in crown	50-100 mm in crown and 30 mm in sides	None

As discussed earlier in Chapter 5 from the rock mass rating (RMR) classification, more than 81% of the data falls in good rock class. Hence, for less than 10m diameter tunnels rock bolts of 20mm diameter and 3m long can be applied with 2.5m spacing and occasional wire mesh. However, those specific locations where the rock mass fall in fair class need 4m bolt length spaced at 1.5 -2 m with wire mesh in crown and walls and 50–100mm shotcrete thickness (Table 7.2).

The other parameter which is more applicable for underground support is the tunnel quality index (Q) classification system. In order to relate the `Q' value with stability and support Barton introduces one more parameter known as Equivalent Dimension, D_e.

$$D_e = \frac{Excavation\,Span, Diameter\,or\,Height\,(m)}{Excavation\,Support\,Ratio\,ESR} \qquad \text{………eq.7.4}$$

Barton et al (1980) had also provided a relation by which length of the rock bolts can be estimated.

$$L = \frac{2 + 0.15B}{ESR} \qquad \text{……..eq.7.5}$$

Where;`B' is the excavation width, 'L' is the length of the rock bolt and `ESR' is the excavation support ratio.

In addition, the maximum unsupported span and permanent roof support pressure can be estimated from ' Q' by;

$$MS = 2\,ESRQ^{0.4} \qquad \text{……eq.7.6}$$

Permanent roof support pressure `Proof'

$$Proof = \frac{2\sqrt{J_n}\,Q^{-1/3}}{3J_r} \qquad \text{…….eq.7.7}$$

Where; J_n is the Joint set number and J_r; Joint roughness number.

Table 7.3 summarizes the possible supports that can be applied determined from tunnel quality index (Q) value of field data using the empirical relations 7.5, 7.6, 7.6 and corresponding support categories in annexure C..

Table 7.3 Possible supports on cavities based on Q value

Items	Tunnels 6-8m span	Cavern wall (u/s & d/s)	Cavern side wall	Cavern roof
Bolt length(m)	2.9 – 3.2	5.3	12 - 14	8 - 9
Bolt spacing(m)	1.6 – 2.4	2 - 4	2.5	2 – 2.5
Shotcrete thickness(mm)	40 - 50	60 - 80	80 - 110	90 - 120
Permanent roof support reassure( )	0.26 – 1.82	0.48 – 1.72	0.46	0.48 – 1.72
Max. unsupported span(m)	3.6 – 6.5	2.84 - 5.4	5.2 – 5.6	2.84 – 5.4

Support measures may also be recommended from RQD data as proposed by Deere et al. 1970) for 6 to 12m diameter tunnels (Table 7.4). However, RQD is a singe parameter and does not consider the joint geometry and tunneling direction, it may not have significant application.

Table 7.4 Support recommendations for tunnel in rock (6 to 12 m) based on RQD when conventional excavation is applied (after Deere et al. 1970)

RQD	Bolting	Shotcrete thickness	Possible locations
>90	None to occasional	None to occasional, local application, 5-8.5cm	Specific locations where BB unit lies
75-90	Pattern bolting, 15-18 cm center	None to occasional local application, 5-8.5 cm	Most gallery & draft tube walls and faces on ABA
50-75	Pattern, 9-15 cm center	10 cm or more in crowns and sides	Draft tubes and galleries where the weak structure crosses

7.6 Stability from Preferred Joint Orientation Aspect

In underground excavations in hard rock, failure is frequently controlled by the presence of discontinuities such as faults, shear zones, bedding planes and joints. The intersection of these structural features can release blocks or wedges which can fall or slide from the surface of the excavation. Failure of the intact rock is seldom a problem in these cases where deformation and failure are caused by sliding along individual discontinuity surfaces or along lines of intersection of surfaces. Separation of planes and rotation of blocks and wedges can also play a role in the deformation and failure process. An analysis of the stability of these excavations depends primarily upon a correct interpretation of the structural geological conditions in the rock mass followed by a study of the blocks and wedges, which can be released by the creation of the excavation. Identification and visualization of these blocks and wedges is by far the most important part of an investigation (Hoek, 1999).

In this study effort is made to work out the preferred orientation of major joints to check the possible failures (wedge or planar). Discontinuity plane, slope face and angle of internal friction along sliding planes were plotted on circular projections for kenematic analysis. The spatial relation of these planes can be interpreted for the possible mode of failure. On hard rock mass slope the main modes of failures are wedge, plane or topping failures. Criteria are developed by Markland (1977) to decide the dominant mode of failure along the intersection of two or more than two planar discontinuities. According to this test wedge failure can prevail if the contact of two discontinuity planes dipping towards each other daylight on the excavation face at an angle greater than the internal friction (ϕ) of the intersection plane. In other words, the plunge of the line of intersection must be less than the face angle, measured in the direction of the line of intersection (Markland et al. 1977). Similarly, a plane failure may take place if the potential failure plane dips towards the excavation and is less than the slope angle.

In general Markland's conditions are:

- Plane failure……………….. $\alpha_f > \alpha_p > \phi$ …….eq.7.8
- Wedge failure……………… $\alpha_f > \alpha_i > \phi$ …….eq.7.9

Where α_f is the excavation angle; α_p is the dip of the potential plane failure; α_i is the plunge of line of intersection of two wedge forming planes and ϕ is the angle of internal friction.

The preferred orientation of the joints is presented in Fig. 7.1 with the excavation faces and respective friction angles (phi circles) worked out from RMR values and Mohr envelope.

As shown in Fig. 7.1, most joints are sub-vertical to vertical as the excavation faces and hence do not full fill the kinematics checks in an extent for possible failures to occur. However, some sets of joint and wall faces in Table 7.5 may have the probability to yield failures provided that all other conditions are fulfilled.

Table 7.5 Sets of wall face and joint orientations that fulfill kenematic checks for <u>Failure</u>

Data	Wall face/joint	Failure type	Location
N 060/90	Wall face	plane	Access adit to irrigation bypass
N 060/62	joint		
N 182/86	joint	wedge	Access adit to irrigation bypass
N 060/62	joint		
N 155/90	Wall face	plane	Pillars and galleries
N 153/75	joint		
N 151/40	joint		
N 045/90	Wall face	plane	Pillars and galleries

N 040/76	joint		
N 115/49	joint	wedge	Pillars and galleries
N 065/62	joint		
N 230/90	Wall face		Draft tubes
N 182/60	joint		
N 320/90	Wall face		Draft tubes
N 182/60	joint		

7.7 Overall Stability condition and support requirements

The main objective of the present study is to characterize the rock mass in order to propose the possible support measures in the underground powerhouse cavity. The overall result of the study shows that the rock mass in the powerhouse cavity is of good quality. During the present investigation no serious instabilities in the rock mass were observed. However, based on the observations and review of the previous investigation results support measures have been worked out.

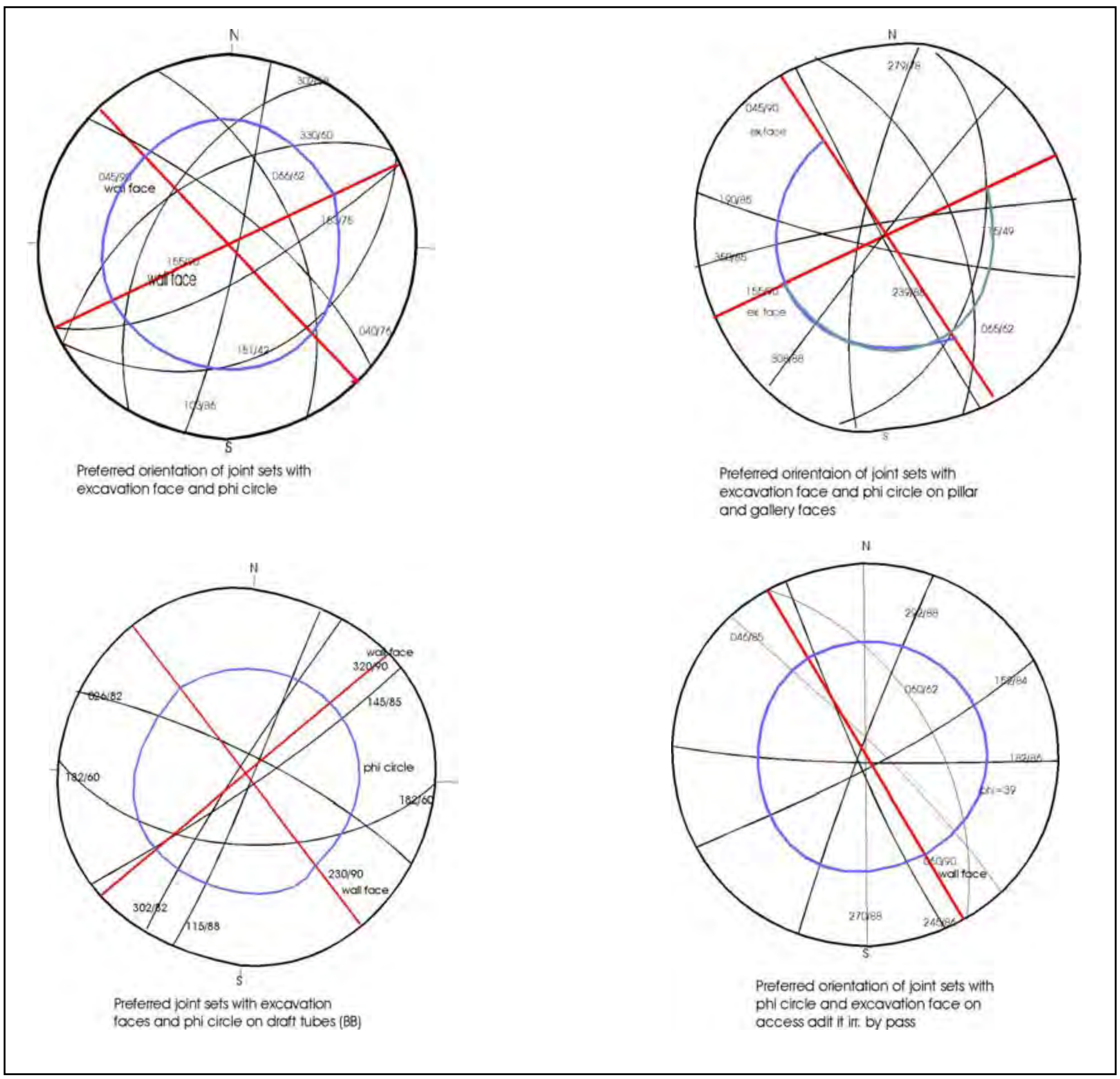


Fig 7.1 Stereonet projection of joints with slope face and phi circle.

Most of the proposed supports based on this study do have close agreement with the actual supports applied. However, it appears that the support applications practiced by the company are on conservative side this may be due to the scale of the project, its economic importance, and for the longer effective life of the project.

The overall condition of the powerhouse cavity may be summarized as:

(i) Most of the access adits excavated for different purposes are not provided with primary supports; however they are still stable resisting the frequent shaking and disturbances during blasting and hammering. This indicates the stability condition of the rock mass, which in general is stable.

(ii) The protected layers are stable; there are no cracks developed. The large dimension powerhouse cavern (22*105m*41m) and the different access adits are stable as no failure has been manifested since its excavation. This indicates in-situ stress are low and rock mass is stable.

(iii) The access adit from the portal has crossed paleosoils and decomposed volcanic tuffs on specific locations. On such units there have been manifestations of instabilities. Detached rock block fall from the roof and walls due to the inability to carry the loads. In addition, there are also weathered and fractured breccia rock unit on the same route. On such specific locations it will be advisable to take remedial measures rather than waiting for failure to occur.

(iv) On the most portions of the powerhouse cavern (wall, roof) and most access adits, there are no weak layers or decomposed units. These are laid on aphanitic and breccia rock units classed as good rock masses. Hence, it can be concluded that the rock mass is suitable for underground cavern and accessory adits. The rock mass classification and characterization systems also confirm the same.

Thus from the present study it may be concluded that the rock mass in the underground powerhouse cavity is relatively of good quality and may not pose much serious stability problems. However, it would be safe to provide proper designed supports at specific locations where rock mass show moderate to high degree of weathering, sheared rocks and weak rock units.

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CHAPTER VIII CONCLUSION AND RECOMMENDATIONS

8.1 Conclusion

The stability of underground structure is totally dependent on the quality of the rock mass through which the excavation passes. Instability of excavation faces, inflow of ground water and development of stresses are some of the problems associated with jointed rock masses. The rock mass suitability for underground excavation is controlled by the strength of the intact rock, the discontinuity condition and the spacing in association with the influence of ground water. In addition, the type and quality of excavation method has also an influence over the stability and the performance of the underground structure.

Large-scale projects like Tana Beles project requires detail investigation and construction follow up with safe and economical construction methods. Because of uncertainties in the rock mass characteristics, ground water conditions and uncontrolled excavation may result in loss of human life and properties. The Tana Beles project has been under construction for over two years and presently more than 70% of the powerhouse complex has been excavated.

The main objective of the present study is to characterize the rock mass for engineering suitability of the powerhouse cavity and to evaluate its stability. For this systematic review of previous investigations was carried out. Besides primary data on rock mass characterization has also been collected from the excavated rock faces. Based on the rock mass characterization stability condition of underground structure has been evaluated.

The powerhouse complex is excavated through aphanitic and breccia rock units by drill and blasting technique. This specific site was investigated to be free of faults. However, one structural discontinuity is identified during this study, which is influencing the cavity components. Most of the powerhouse cavity has been noted to be stable, however instability was noticed along the fault line which trends N40^0 and dips towards 310^0 with 88^0 dip angle. All the other systems of faults out side the powerhouse area are believed to be in active and far from tectonic activities. So, they will not be dangerous to the structure.

The influence of ground water is found to be insignificant throughout the powerhouse cavity. Concentrated flow was observed once when the access adit to penstock shaft was opened. Some minor seepage and damp surfaces were also manifested owing to the weak structure, which traverse through the rock mass in the powerhouse cavity.

The Tana Beles project site is located where the seismic effects are insignificant as this site falls into zone 5 of MM intensity Seismic risk map of Ethiopia 100 years return period, 0.99 probabilities by Laike Mariam Asfaw, (1986).

In order to understand the general geological condition of the site and to make comparison with the actual situation a careful review of the site feasibility investigation report has been carried out. For this data on geophysical investigation, borehole-drilling core logs and water pressure test results were reviewed. The geophysical and water pressure test results confirm the stability and the water tightness condition of the powerhouse cavity.

The average RQD values from borehole log results indicates that the rock mass is of good quality. On logs, the uni-axial compressive strength (UCS) values for aphanitic rock unit and breccia rock unit ranges in extreme contract. Aphanitic rock demonstrates high strength whereas breccia in contrast possesses low strength. This is most probably due to the inherent nature of the breccia unit.

For the present study primary data collection was made randomly as most of the excavated slope faces are covered with the shotcrete supports. Special attention was given to collect data following new excavation on galleries and draft tubes. Based on the collected data the rock mass rating (RMR) and tunnel quality index (Q) indicates that the rock mass is of good and fair quality, respectively. The UCS values determined during RMR classification show great variation for aphanitic and breccia rock units hence agree with the UCS values as mentioned in core logs. The RQD value used for both classification systems (RMR and Q) for aphanitic and breccia units are of good and excellent quality, respectively. However, the differences of UCS and RQD values are compensated by other parameters, as for both units the RMR value is the same. Moreover, variation is seen between UCS values for log samples and the values determined during the present study.

The construction company has set its own classification ranges to classify rock mass according to RMR. The ranges do not correspond to the internationally accepted Bieniawski's guide line. Hence, the RMR values which have to be grouped as class II are being grouped as class I (A).

Further, based on the RMR values different failure criteria parameters and material constants are determined. These parameters are used to characterize the rock mass. Shear strength parameters(C andϕ) determined from RMR and Mohr circle envelope are almost equal with slight variation in Cohesion values for aphanitic basalt. However, the 'C' values used in the final geological report of the company are relatively lower than the estimated values. In addition, the material constants (mi) adopted on the final geological report, 2007, by the company to determine the other parameter (mb) is greater and far from the standard values.

The physical condition of the rock mass is described by fresh to slightly weathered, moderately jointed aphanitic basalt with major vertical and continuous joints having smooth joint surfaces. On the other hand the breccia unit is slight to moderately weathered and less jointed having rough joint surface. In both cases the dominant joints are vertical and the spacing is moderately close resulting in a blocky/ seamy rock mass grading.

The excavation for powerhouse complex is done with controlled drill and blast method followed by scaling and mucking. During these processes, considerable over breaks and instabilities were not manifested.

The support types and system of application determined from empirical relation of the ‘Q’ system are closer to the one which is being applied in the powerhouse cavity. However, the supports being applied are stronger and systematic than what is estimated.

The preferred joint orientation analysis for wall faces indicated the existence some possibilities of plane and wedge failure.

The overall stability condition of the powerhouse cavity can be summarized as;

Most of the access adits excavated for different purposes are not provided with primary supports; however they are still stable resisting the frequent shaking and disturbances during blasting and hammering.

The protected layers are stable; there are no cracks developed. The large dimension powerhouse cavern (22*105m*41m) and the different access adits are stable, as no failure has been manifested since its excavation. This indicates in-situ stress are low and rock mass is not subjected to stress disturbances.

The access adit from the portal has crossed paleosoils and decomposed volcanic tuffs on specific locations. On such units there have been manifestations of instabilities. Detached rock block fall from the roof and walls due to the inability to carry the loads. In addition, there are also weathered and fractured breccia rock unit on the same route. On such specific locations it will be advisable to take remedial measures rather than waiting for failure to occur.

In the powerhouse cavern (wall, roof) and most of the access adits, there are no weak layers or decomposed units. These are laid on aphanitic and breccia rock units classed as good rock masses. Hence, it can be concluded that the rock mass is suitable for underground cavern and accessory adits. The rock mass classification and characterization systems also confirm the same.

Thus from the present study it may finally be concluded that the rock mass in the underground powerhouse cavity is relatively of good quality and may not pose much serious stability problems.

8.2 Recommendations

Recommendations are very important and significant if actions are to be taken in correspondence to them. For projects, which are on the way of completion, recommendations may not have relevance. However, recommendations may be learning tools for other projects to be taken into action. Accordingly, few recommendations are forwarded based on the overall activities of the site having in mind that the project is successful by its nature and the construction procedure.

(i) Though uncertainties are always there, detail investigation of such large-scale projects is very important before construction. The geophysical investigations have traced the presence of weak structures though their depth of investigation is limited to shallow depths. However, the powerhouse site is reported to be a block free of faults. Such overlooks should be avoided because that weak structure is most probably the one manifested during construction. As a matter of chance the effect of this structure was low and controllable.

(ii) Subsurface investigation by borehole drilling is very important design tool if managed properly. The details and safeties made on core samples are good for this project. But sometimes there are cases where the RQD values do not correspond to fracture indexes. Such conditions must be properly justified if reasons are there otherwise may give wrong interpretations. Improper sampling or wrong fracture index may be one reason.

(iii) For underground structures the tunnel quality index (Q) rock mass classification system is proved to be more appropriate. However, for this case it is only the Rock Mass Rating (RMR) classification system is being practiced to qualify the rock mass for price quotation and then to propose support systems. Even the RMR classification ranges set by the construction company do not agree to the standard one. Good rocks are lumped to be excellent and fair to good quality. In this regard no scientific reason may be given perhaps this may be for high price quotation.

(iv) Whenever underground uncertainties such as weak structures are faced during construction, problem appreciation will not be sufficient. In addition, it will be advisable to make efforts to identify the probable cause. The weak structure trending N040^0 and dipping towards 310^0 has created constructional problems during the first phases of construction. Attention was given to treat the rock mass following its RMR value. In this regard efforts should be made to justify the route cause of such a problem from the beginning. Such approach would have been useful in evolving proper remedial measures. The weak structure may not bring serious problem on the structure. However, it will be advisable if possible efforts are made to overcome problems related to leakage since it results in weakening of the supports, ground water pressure on roofs and walls which may bring inconveniency on the future performance of the project.

(v) It will be advantageous to treat the instabilities observed on specific locations of the access portal due to weak rocks rather than ignoring their effects and waiting for possible failure to occur.

(vi) Drainage pipes are not provided on the roof and walls of the cavern. This is so because there was no ground water problem. However, it is good to provide drainage pipes for any suspected water in future.

(vii) Generally the hydro geological condition does not seam to bring major construction and failure problem. But there needs the provision of drainage pipes and if possible grouting of the identified fracture line. Because the water that seeps through any weak access can accumulate to this zone and other possible locations producing large pressure on surface supports which may result to failure.

In addition, water can act as the main agents of chemical weathering and decomposition to take place through out the life of the project (50 years) contributing for lose of the structural strength through time.

(viii) The rock material constant (mi) considered for failure criteria analysis are too large as compared to the standard value this may lead to overestimated support design.

(ix) The present study has been conducted under the constraints of time, resources and financial support, therefore the results and the recommendations made through this study must be considered as indicative only. More elaborate systematic studies would be required before coming to any final decisions on.

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Annexure

Annex A: (a) Water Pressure Test results for Borehole PH 7

Depth (m)	Pressure guage (Kpa) in 5 steps					Absorption l/min/m					L-value(Lugeon unit)					Flow regime	Lu
	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5		
20-25	100	300	500	300	100	1.4	1.28	2.14	0	0	6.2	3	3.5	0	0	Void filling	0
30-35	100	300	500	300	100	0.42	1.66	1.84	0	0	1.8	3.9	3	0	0	Void filling	0
50-55	100	300	500	300	100	0	0.12	0.2	0	0	0	0.3	0.3	0	0	Dilation	0
90-95	100	300	500	300	100	2.64	7.76	6.92	6.74	4.3	9.8	33.1	14.2	22.5	19.8	Wash out	33.1
100-105	100	300	500	300	100	2	0.92	1.08	0.28	0	6.7	1.8	1.5	0.5	0	Void filling	0
110-115	100	300	500	300	100	0	0.58	0.14	0	0	0	1	0.2	0	0	Void filling	0
130-135	100	300	500	300	100	0.24	0.36	0.08	0	0	0.5	0.5	0.1	0	0	Void filling	0
150-155	100	300	500	300	100	0.24	0.02	0.04	0	0	0.04	0	0	0	0	Void	0
170-175	100	300	500	300	100	0	0.04	0	0	0	0	0.1	0	0	0	-	
180-185	100	300	500	300	100	0	0	0	0	0	0	0	0	0	0	-	0
190-195	100	300	500	300	100	0.04	0.02	0.62	0	0	0.1	0	0.6	0	0	Dilation	0
210-215	100	300	500	300	100	0.02	0	0.06	0	0	0	0	0.1	0	0	Dilation	0
220-225	100	300	500	300	100	0	0	0	0	0	0	0	0	0	0	-	0
230-235	100	300	500	300	100	0	0	0.204	0	0	0	0	0	0	0	Dilation	0
250-255	100	300	500	300	100	0	0.02	0.16	0	0	0	0	0.2	0	0	Dilation	0
270-275	200	400	800	400	200	0	0	0.04	0	0	0	0	0	0	0	Dilation	0
280-285	100	300	500	300	100	0	0	4.44	0	0	0	0	4.9	0	0	Dilation	0
295-300	200	400	800	400	200	0	0	0	0	0	0	0	0	0	0	-	0
301-306	200	400	800	400	200	0	0	0	0	0	0	0	0	0	0	-	0

(Source: Feasibility Report of Tana Beles Project, Part 'C', 2005)

Annex A: (b) Water Pressure Test results for Borehole PH 3

Depth (m)	Pressure test in bars in 5 steps					Absorption l/min/m					L- value (Lugeon unit)					Flow regime	Lugeon value
	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5		
258-263	2.75	4.75	8.75	4.75	2.75	0	0	0	0	0	0	0	0	0	0	-	0
263-268	2.4	4.4	8.4	4.4	2.4	0	0	0.1	0	0	0	0	0.1	0	0	Dilation	0
268-273	2.4	4.4	8.4	4.4	2.4	0.04	0	0.04	0	0	0.2	0	0	0	0		0.2
273-278	2.37	4.37	8.37	4.37	2.37	0	0	0.02	0	0	0	0	0	0	0	Dilation	0
278-283	2.37	4.37	8.37	4.37	2.37	0	0	0.04	0	0	0	0	0	0	0	dilation	0
283-288	2.64	4.64	8.64	4.64	2.64	0	0	0.1	0	0	0	0	0.1	0	0	dilation	0
288-293	2.57	4.57	8.57	4.57	2.57	0	0	0.04	0	0	0	0	0	0	0	dilation	0
293-298	2.66	4.66	8.66	4.66	2.66	0	0	0.06	0	0	0	0	0	0	0	dilation	0
298-303	2.76	4.76	8.76	4.76	2.76	0	0	0.06	0	0	0	0	0.1	0	0	dilation	0
303-308	2.76	4.76	8.76	4.76	2.76	0	0	0.02	0	0	0	0	0	0	0	dilation	0
308-313	2.24	4.24	8.24	4.24	2.24	0	0	0.06	0	0	0	0	0.1	0	0	dilation	0
313-318	2.7	4.7	8.7	4.7	2.7	0	0	0.08	0	0	0	0	0.1	0	0	dilation	0
318-323	2.25	4.25	8.25	4.25	2.25	0	0	0.04	0	0	0	0	0	0	0	dilation	0
323-328	2.45	4.45	8.45	4.45	2.45	0	0	0.04	0	0	0	0	0	0	0	dilation	0
Note: - Pressure in Kpa for all the above steps discussed is (200.400, 800,400,200).																	

(*Source: Feasibility Report of Tana Beles Project, Part "B", 2005)*

Annex B: rock mass classification Q system

1.	Rock Quality Designation	RQD
A	Very Poor	0 – 25
B	Poor	25 – 50
C	Fair	50 – 75
D	Good	75 – 90
E	Excellent	90 – 100
Note:	(i) Where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q. (ii) RQD interval of 5, i.e., 100, 95, 90, etc., are sufficiently accurate.	

2.	Joint Set Number	J_n
A	Massive, no or few joints	0.5 – 1
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint set	4
E	Two joint set plus random joints	6
F	Three joint set	9
G	Three joint set plus random joints	12
H	Four or more joint sets, heavily jointed	15
J	Crushed rock, earthlike	20
Note:	(i) For intersections, use ($3.0 \times J_n$). (ii) For portals, use ($2.0 \times J_n$).	

3.	Joint Roughness Number	J_r
(a)	***Rock-wall contact, and (b) Rock wall contact before 10 cm shear***	
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough or irregular, planar	1.5
F	Smooth, planar	1.0
G	Slickensided, planar	0.5
Note:	(i) Descriptions refer to small and intermediate scale features, in that order.	
(c)	***No rock-wall contact when sheared***	
H	Zone containing clay minerals thick enough to prevent rock-wall contact	1.0
J	Sandy, gravelly or crushed zone thick enough to prevent rock-wall contact	1.0
Note:	(ii) Add 1.0 if the mean spacing of the relevant joint set ≥ 3 m. (iii) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented for minimum strength.	

4.	Joint Alteration Number	ϕ_r approx.	J_a
(a)	***Rock-wall contact (no mineral fillings, only coatings)***		
A	Tight healed, hard, non-softening, impermeable filling, i.e., quartz or epidote	–	0.75
B	Unaltered joint walls, surface staining only	25 – 35°	1.0
C	Slightly altered joint walls. Non-softening mineral coating, sandy particles, clay-free disintegrated rock, etc.	25 – 30°	2.0
D	Silty- or sandy-clay coatings, small clay fraction (non-softening)	20 – 25°	3.0
E	Softening or low friction mineral coatings, i.e., kaolinite or mica. Also chlorite, talc, gypsum, graphite, etc., and small	8 – 16°	4.0

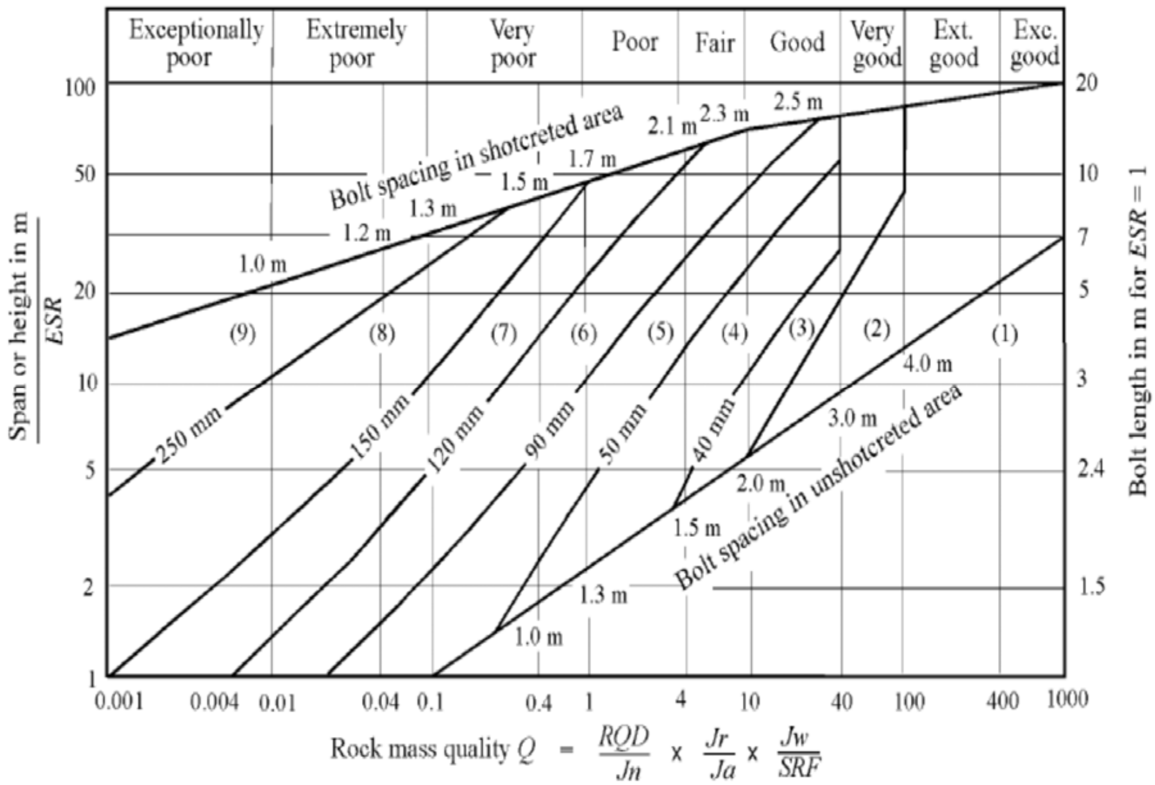
	quantities of swelling clays		

(b)	***Rock wall contact before 10 cm shear (thin mineral fillings)***		
F	Sandy particles, clay-free disintegrated rock, etc.	25 – 30°	4.0
G	Strongly over-consolidated non-softening clay mineral fillings (continuous, but < 5 mm thickness)	16 – 24°	6.0
H	Medium or low over-consolidated softening clay mineral fillings (continuous, but < 5 mm thickness)	12 – 16°	8.0
J	Swelling-clay fillings, i.e., montmorillonite (continuous, but < 5 mm thickness). Value of J_a depends on percent of swelling clay size particles, and access to water, etc.	6 – 12°	8 – 12
(c)	***No rock-wall contact when sheared (thick mineral fillings)***		
K, L, M	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6 – 24°	6, 8, or 8 – 12
N	Zones or bands of silty- or sandy-clay, small clay fraction (non-softening)	-	5
O, P, R	Thick, continuous zones or bands of clay (see G, H, J for clay condition description)	6 – 24°	10, 13, or 13 – 20

5.	**Joint Water Reduction Factor**	**Water pressure**	**J_w**
A	Dry excavation or minor inflow, i.e., < 5 l/min locally	< 1 (kg/cm^2)	1.0
B	Medium inflow or pressure, occasional outwash of joint fillings	1 – 2.5	0.66
C	Large inflow or high pressure in competent rock with unfilled joints	2.5 – 10	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings	2.5 – 10	0.33
E	Exceptionally high inflow or water pressure at blasting, decaying with time	> 10	0.2 – 0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay	> 10 (kg/cm^2)	0.1 – 0.05
Note:	(i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed. (ii) Special problems caused by ice formation are not considered.		

6.	**Stress Reduction Factor**	**SRF**
(a)	***Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated***	
A	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10
B	Single weakness zone containing clay or chemically disintegrated rock (depth of excavation ≤ 50 m)	5
C	Single weakness zone containing clay or chemically disintegrated rock (depth of excavation > 50 m)	2.5
D	Multiple shear zones in competent rock (clay-free) (depth of excavation ≤ 50 m)	7.5
E	Single shear zone in competent rock (clay-free) (depth of excavation ≤ 50 m)	5
F	Single shear zone in competent rock (clay-free) (depth of excavation > 50 m)	2.5
G	Loose, open joint, heavily jointed (any depth)	5
Note:	(i) Reduce SRF value by 25-50% if the relevant shear zones only influence but not intersect the excavation.	

Annex C: Estimated support categories based on the tunneling quality index *Q* (After Grimstad and Barton 1993)



REINFORCEMENT CATEGORIES

1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting with 40-100 mm unreinforced shotcrete
5) Fibre reinforced shotcrete, 50 - 90 mm, and bolting
6) Fibre reinforced shotcrete, 90 - 120 mm, and bolting
7) Fibre reinforced shotcrete, 120 - 150 mm, and bolting
8) Fibre reinforced shotcrete, > 150 mm, with reinforced ribs of shotcrete and bolting
9) Cast concrete lining

Annex Annexure' D': complete laboratory test results of the four rock units

		Unconfined Compression		Triaxial Compression			
UNIT	γ	UCS	YM(E)	δ1	δ3	TYM(E)	VP
ABA	24.76	36.15	44701				5168
	27.23	126.61	91968				5172
	24.62	33.38	65380				2820
	27.83	191.47	79363				5550
	25.36	85.33	52932				3647
	25.65			205.53	10	79170	4880
	27.2	184.98	65692				4853
	26.46			315.31	15	66740	5040
	27.59			144.21	10	90889	4740
	28.01	331.22	65340				5109
	27.88			414.78	5	79719	4736
	26.44			286.65	15	44191	4578
	21.68	48.54	26671				3849
	25.24			99.14	3	32996	3686
	27.68	249.92	67711				4583
VB	26.98	272.6	60516				4974
	21.65	35.66	6956				2046
	25.37	31.5	16384				2807
	23.76	19.93	53794				2751
	21.98			154.54	6	33707	3615
	20.6	17.84	5397				2645
	18.58			98.03	10	15429	3086
	21.69	52.93	21765				3401
BB	21.86	75.37	54552				3201
	20.62	26.82	6750				2228
	21.42			62.31	5	8263	2291
	17.53	21.92	23241				2883
	22.29			124.77	10	19213	3097
	23.22	34.85	30218				2772
	21.2			105.76	10	17805	2821
	20.75	19.05	19072				2818
	21.827			98.1	15	15411	2749
	21.82			109	15	12644	2882
	18.06			28.92	3	2962	1679
	21.08	22.26	7773	52.44		8865	2737
	22.55	52.35	12935				2705
	22.96	79.21	25974				3553
	21.41			96.93	20	5411	2709
	20.38	32.88	7274				2297
	21.91	29.96	37444				3477
	21.94	43.66	15894				3142
	22.15	33.12	7439				2948
	21.21	11.42	4690				2220
	20.87	7.58	2586				1412
	24.77	21.51	3834				3109
	22.05			67.82	10	5962	1793
	21.28	6.37	1803				650
TT	14.16	25.4	5253				2255

	13.08	15.53	2145				1686
	14.75	6.75	1007				989
	13.06	13.39	4348				1942
	15.66	34.38	6241				2248
	14.82	14	1933				1395
	13.13	14.87	3191				2251
	16.16	27.77	6426				2383
	14.23	14.49	1760				1512
	14.91	23.6	3359				2035
	15.09	12.97	13.13				1566
	16.02	39.63	5703				2298
	17.45	26.21	3293				1893
	13.05			42.05	10	687	1340
	15.23	7.34	895				1257
	15.31			32.09	5	1333	1365
	16.08	11.18	1261				1392
	16.46	9.67	1137				1493
	17.35			75.89	15	2017	1333
	17.98	7.63	1116				1350
	22.2	14	1737				1657

Annex E: chart represented by Hoek, Marinose Benissi (1998)

Generalized Hoek-Brown criteria δ1'=δ3'+δc(mb*(δ3'/δc)+s)[a] δ1'=Major principal effective stress at failure δ3'=Minor principal effective stress at failure δc=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	**Surface condition**	Very good=very rough un weathered surface	Good= rough, slightly weathered or altered surface	Fair= Smooth, moderately weathered or altered surfaces	Poor = slikensided, highly weathered surfaces with compact coating or fillings containing angular rock fragments	Very poor = slikensided, highly weathered surfaces with soft clay coatings or fillings
Structure						
BLOCKY –very well interlocked undisturbed rock mass consisting of cubical blocks formed by three orthogonal discontinuity sets	mb/mi	0.6	0.4	0.26	0.15	0.08
	s	0.19	0.062	0.015	0.003	0.0004
	a	0.5	0.5	0.5	0.5	0.5
	Em	75000	40000	20000	9000	3000
	v	0.2	0.2	0.25	0.25	0.25
	GSI	85	75	62	48	34
VERY BLOCKY- interlocked partially disturbed rock mass with multifaceted angular blocks formed by four or more discontinuity sets	mb/mi	0.4	0.29	0.16	0.11	0.07
	s	0.062	0.021	0.003	0.001	0
	a	0.5	0.5	0.5	0.5	0.53
	Em	40000	24000	9000	5000	2500
	v	0.2	0.25	0.25	0.3	0.3
	GSI	75	65	48	38	25
BLOCKY/ SEAMY- folded and faulted with many intersecting discontinuities forming angular blocks	mb/mi	0.24	0.17	0.12	0.08	0.06
	s	0.012	0.004	0.001	0	0
	a	0.5	0.5	0.5	0.5	0.55
	Em	18000	10000	6000	3000	2000
	v	0.25	0.25	0.25	0.3	0.3
	GSI	60	50	40	30	20
CRUSHED-poorly interlocked, heavily bracken rock mass with a mixture of angular and rounded blocks	mb/mi	0.17	0.12	0.08	0.06	0.04
	s	0.004	0.001	0	0	0
	a	0.5	0.5	0.5	0.55	0.6
	Em	10000	6000	3000	2000	1000
	v	0.25	0.25	0.3	0.3	0.3
	GSI	50	40	30	20	10