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School of Graduate Studies

Faculty of Science

Engineering Geological Appraisal of Dam Foundation For Gumara Dam, Amhara Region, North Western Ethiopia

A Thesis

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AGERIE GENETU

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Abstract

Gumara irrigation project is being under taken by the Ministry of Water Resources for the development of the agriculture sector of the country. The project envisages the construction of a zoned earth dam for irrigation purpose. The Gumara Irrigation project is found in Northern Ethiopia, Amhara Regional state, south Gondar Administrative Zone.

The present research study was planned to investigate the engineering geological suitability of Gumara dam foundation. The study was aimed to determine the suitability of the dam foundation conditions against seepage, and deformation of the foundation material in response to the loads imposed by the dam. Thus, through the present study an attempt has been made to identify the adverse foundation conditions, which may pose problems during or after the construction stage. Based on the foundation conditions certain remedial measures have been worked out. This contributes towards the safety as well as the economic planning and execution of the Gumara irrigation projects.

From the review of the previous investigation reports and the investigations made during the present study through the surface, subsurface investigations and laboratory test results an overall appraisal of foundation condition of Gumara Dam has been made. In order to over come the various problems associated with the dam foundation suitable remedial measures have been worked out.

From the evaluation of the foundation condition it has been deduced that the rocks on the abutments and the bed rock in the river section are pervious and needs grouting to improve permeability condition. The left abutment slope is kinematically stable for the present geometric configuration however, it may become unstable during the stripping operations if it is cut at steeper angles. On abutments the rocks are weathered and contain wide open joints these undesirable rocks have to be stripped off before laying the embankment. The most serious problem in the foundation section is the presence of thick organic soil strata, which are about 47m thick. These soils are semi-pervious and highly compressible. In order to improve the permeability condition of these soil strata certain preventive and curative measures are suggested. Further, to overcome the compressibility of organic soils some possible treatments have been suggested.

Annexure

Introduction Chapter I

1.1 General

Dams are civil engineering structures build across the river valley to impound large volume of water to be used for single or multipurpose use of; power generation, irrigation purpose, flood control, ground water recharge and water diversion.

The most common reasons for building dams are to concentrate the natural fall of a river at a given site, thus making it possible to generate electricity; to direct water from rivers into canals and irrigation and water-supply systems; to increase river depths for navigational purposes; to control water flow during times of flood and drought; and to create artificial lakes for recreational use. Many dams fulfill one or several of these functions.

The construction of dam requires an engineering geological investigation. In the course of last few decades, geological study of the dam sites is being looked up on as a matter of considerable importance, since a number of projects happened to fail due to the adverse geologic conditions.

A dam must be impervious to water, leakage through or under a dam must be prevented to avoid excessive water loss and to prevent undermining of the structure itself. A dam must also be constructed in such a way as to withstand the forces exerted upon it. Some forces that engineers must consider when designing a dam are gravity (which tends to pull the dam down); hydrostatic pressure (from water behind the dam); uplift (vertical forces tending to reduce the weight of the dam) caused by hydrostatic pressure on the foundation; ice pressures; and earth stresses and tensions, including the effects of earthquakes.

When a site is being considered for construction of a dam, earthquake hazards must be taken into account as part of a thorough geologic analysis. In addition, geologists must determine whether the natural foundations are subject to seepage and whether they have the strength to support the weight of the dam and the water that will back up behind it.

The engineering geological investigation contributes towards the safety as well as the economic planning and execution of the engineering projects. Therefore, the engineering investigation in the foundation of a dam must be conducted with care and caution for the long life of the project.

It has long been mentioned that Ethiopia is placed among the poorest nation in the world. The majority of the population lives in rural areas and the economy of the country is dependent on rain fed agriculture. Due to the recurring cycle of long drought time to time and the erratic nature of the rainfall contribute a lot for the decline of the country's economy.

To reduce the sever poverty of the farmers living in rural areas and to alleviate the problems caused by the erratic rainfall on the agriculture sector, Ethiopian Government device a strategic policy that centers on the development of rural areas. As a result numerous agriculture projects are now under way aimed at the utilization of the rivers for irrigation purpose. Among such projects Gumara irrigation project is one, which is being under taken by the Ministry of Water Resources for the development. The project envisages the construction of a zoned earth dam and canal network mainly for irrigation purpose.

The proposed research study is planned to investigate the engineering geological suitability of Gumara dam foundation.

1.2 Study area Location

The Gumara Irrigation project is found in Northern Ethiopia, Amhara Regional state, south Gondar Admistrative Zone. The area is accessible with vehicle, which is about 50km from the capital town Bahir Dar. From Bahir Dar to Woreta town, the first 24 km is on Asphalt road. The rest however, is poor network of all weathered road which is accessible only in dry seasons.

The geographical location of the dam site is 37˚ 47′ 44″ E longitude and 11° 44′ 30″ N latitude. The dam site is located on the Ethiopian Mapping Agency toposheet No. 1137-B4 of MEBEREJ at a scale of 1:50,000.

1.3 Project Review – Appraisal Through Previous Studies

Gumara Irrigation Project has been studied through number of studies by number of organizations/ Agencies/ Individuals. These includes, United State Bureau of Reclamation (USBR), Water and Power Consultancy Services, India (WAPCOS), A French engineering consultant BECOM and Water Works Design and Supervision Enterprise' and ICT (Indian Consultancy Firm) (2006).

An attempt has been made to carefully review the investigations and findings of all previous studies. A systematic summary of previous studies is presented in the following paragraphs;

1.3.1 Study by (USBR) in 1964

The United State Bureau of Reclamation (USBR) was the first to undertake studies for investigation for land and water resource Development in Blue Nile Basin. In their study one dam site on the middle tributary of Gumara was identified.

In reconnaissance geological description of Gumara dam site it was stated that bed rock and overburden at this site include a thin layer of the scoraceous porous, younger volcanic rock on the valley floor. Bed rock doesn't crop out, but many rhyolitic boulders appear at the surface and the over burden is probably quite thin on average, as most of the upper slope are covered with reddish brown silty clay. At the valley bottom dark gray to black, heavy, plastic clay covers the bed rock. Along the stream channel, small gravel and sand deposit were observed. These are probably lens shaped and discontinuous.

The Younger volcanic flows mentioned earlier probably extends up the valley beyond the dam site and into the reservoir. The basaltic lava is very scoraceous where observed and will be permeable. The thickness of these flows may not be great, and since the reservoir is covered with impermeable overburden, seepage may not be excessive. At the dam site, the excavation of the overburden in the area for cutoff trench, the basalt would be exposed.

For subsurface investigation diamond drilling have to be employed to determine the permeability and thickness of the bed rock. If the thickness is not great the rock has to be removed down to the sound older volcanics cutoff trench. This is to assure sealing off the reservoir at the dam site.

A clayey soil blanket covers most of the reservoir area. It is assumed to be thick enough to prevent excessive leakage.

According to USBR findings, Gumara dam, in general, is believed to be geologically feasible to provide storage for the proposed project. However, USBR made the strong recommendations that the foundation material has to be thoroughly explored during pre construction studies.

1.3.2 Study by WAPCOS (1990)

Water and Power Consultancy Services, (WAPCOS), an Indian firm under took studies for the preliminary water resource development plan for Ethiopia during 1988-1990. Their studies were also of reconnaissance level extending over all the river basins of Ethiopia. They proposed two dam sites on two tributaries of Gumara; (i) Gumara I on Sendega and (ii) Gumara II on Main Gumara. Gumara I of WAPCOS was very near to the proposed dam site of USBR.

The WAPCOS study provided a description of the general geology around the Lake Tana area. Specific reference to the dam site geology consisted of stating that the abutments being composed of massive basalts under a cover of scree mixed with rock debris. Presence of younger volcanics that consists of heterogeneous permeable rocks has been indicated in the study.

The WAPCOS study has estimated that an excavation of a minimum of 8 m at the river bed and 4m at the abutments will be required before a moderately sound bed rock could be exposed at the dam site. The requirement for a deep cut off trench with adequate grouting has been indicated in the WAPCOS study. The potential use of basalts and rhyolites as rock fill material has also been mentioned in the study.

The reconnaissance level of WAPCOS study estimate the bed rock is shallow for Gumara dam sites but as seen from the boreholes log record of the Water Works And Design Enterprise Log report the bed rock exposed in Gumara B dam site at a deep depth which is below 47 meter.

1.3.3 Study by BECOM (1999)

The BCECOM, a French engineering consultant prepared Abbay River Basin integrated development master plan project (1999). Their studies were also of reconnaissance level but unlike WAPCOS (Indian Firm), and like USBR the Scope of their study was confined to Abbay Basin only. The BECOM report was based on the general survey of the whole Abbay basin considering economic and social aspect. The study reviewed the sites identified by both USBR (1964) as GM-6 (N11°45'30" and E 37°48'30") and WAPCOS (1990) as Gumara II (N 11° 44' 30" and E 37° 47' 40") renamed them as Gumara A and Gumara B. BECOM considered Gumara B as an alternative to Gumara A site (Fig.1.1). They have considers both sites but have not given their recommendation in favor of either.

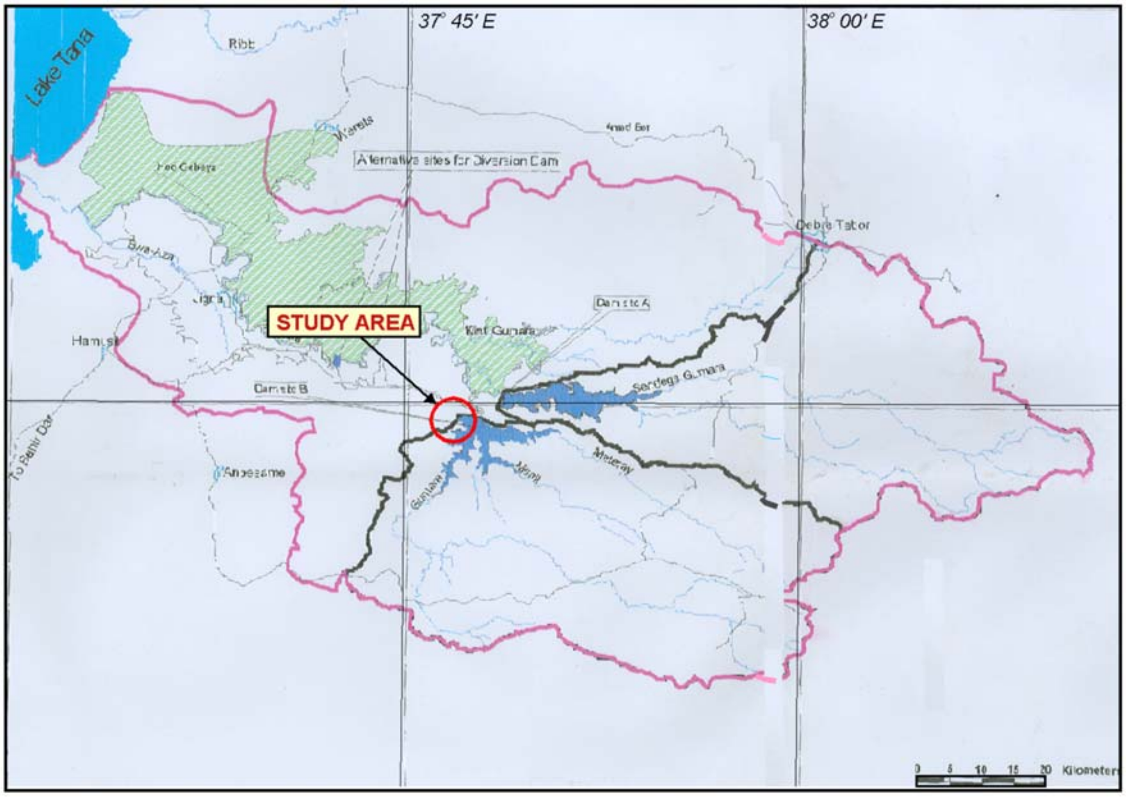

Fig. 1.1 Alternative Gumara dam sites as proposed by WAPCOS and BECOM.

The Abbay Master plan report summarizes the geology and engineering geology of the proposed dam sites as mentioned in the following paragraphs;

The site is entirely located in volcanic formations of the 'Shield Group', which historically corresponds to the second phase of the main volcanic eruption. More precisely, it refers to the second event of 'Termaber basalt eruption' of Miocene age. This basaltic unit overlies the basaltic unit of the first sequence of Termaber activity in this area, which being slightly older in age.

Some Quaternary volcanic rocks are known in Debre Tabor quadrangle: the main ones correspond to the filling of the paleo-Valley of the Gumara rivers overlapping the second Termaber series. Alluvium materials are also present in the riverbed and below the

Quaternary volcanic deposited. Recent alluvium deposited in terraces, at least 10 m above the actual river bed, have also been noticed around a down stream of the dam site area. These alluviums are mainly composed of sandy gravel and clay to silty clay material. The lower valley of the Gumara river has been flattened by flood deposits of the Lake Tana giving marsh and swampy area and layered lacustrine deposits.

1.3.4 Study by 'Water Works Design and Supervision Enterprise' and ICT (Indian Consulting Firm) (2006)

All the previous studies made prior to WWDSE & ICT were at reconnaissance level and no feasibility study was conducted. However, in the recent study conducted by WWDSE associated with ICT, a feasibility investigation was carried out on the foundation of the dam on Gumara B site. This site was selected by assessment based on visual inspection of the two sites suggested by Both WAPCOS (1990) and BCEOM (1999). It has been observed that the two sites are technically feasible with regards to topography. The geology of the two sites should not vary much as they are very near to each other. According to this study, the height of the proposed dam and the reservoir capacity are also comparable. However, there is a saddle dam involved in case of Gumara A whereas there is no such feature present in case of Gumara B site. Thus Gumara B is considered preferable as compared to Gumara A site.

1.4 Objective of the present study

An earth dam foundation must support its weight and the force of the water acting on it with adequate safety under the worst possible combination of high water, seepage forces and natural stresses. The major problem associated with embankment dams is the seepage problem through the dam foundation. Besides, the presence of undesirable soils in the foundation may result into differential settlement. Therefore, the first step in the design of earth dam is a systematic foundation investigation and thorough appraisal.

The Gumara dam project envisages irrigating 13,976 ha of farmland. The success of this project will entirely depends on its safe functioning and to meet out the irrigation needs of the command area. Therefore, it becomes essential that the dam foundation must be watertight and it must provide a stable foundation to withstand the loads imposed by the dam. From the initial stage of the feasibility study using boreholes by WWDAS with ICT the foundation is challenging because of the existence of thick organic clay soil in the foundation area.

Investigation of the suitability of the foundation of the dam is essential for the safe design and planning. Based on this fact the the present research work have the following objectives;

General Objective

Engineering Geological appraisal of the dam foundation

Specific Objectives

(i) To determine important index and engineering geological properties of the rocks and soils present in the foundation area of the dam and to workout their suitability for the dam foundation.

(ii) To determine the seepage potential of the foundation material and to workout suitable measures for seepage control.

1.5 Methodology

In order to achieve the above-mentioned objectives the following systematic methodology has been adopted;

i) Preparation of the base map of the study area from existing topographical maps.
ii) Literature review to have an overview of geological, geomorphologic, hydro-geological and engineering geological condition of the dam site and the surrounding areas.
iii) Geological Mapping of the dam site.
iv) Sub-surface exploration through Borehole logs.
v) Collection of soil and rock samples from dam foundation area for laboratory testing and analysis to determine various index properties.
vi) Assessment of the seimicity risk of the project area using the seismic risk map of Ethiopia.
vii) Assessment of permeability of the dam foundation material through existing water pressure test data and qualitative assessment through surface and sub-surface geological conditions.
viii) Assessment of stability conditions of the abutments. Collection of necessary data and samples for laboratory testing.
ix) Based on the above methodology suitable remedial measures to improve the foundation condition has been suggested.

1.6 Importance of the study

An earth dam foundation hydraulically must be safe against seepage erosion and must not allow enough water to percolate through them to deplete the reservoir and structurally must support its own weight and the force of the water acting on it with adequate safety under the worst possible combination of high water, seepage forces and natural stresses. Thus the engineering geological investigation in the foundation of a dam must be conducted with care and caution for the long life of the project.

Gumara Irrigation project is planned to meet the irrigation need of 13,976 ha of farmland which contribute for the utilization of the scarce water resource. The proposed structure is an embankment dam on the river Gumara.

The present research study was aimed to determine the suitability of the dam foundation conditions against seepage, and deformation of the foundation material in response to the loads imposed by the dam. Thus, through the present study an attempt has been made to identify the adverse foundation conditions, which may pose problems during or after the construction stage. Thus, based on the foundation conditions certain remedial measures have been worked out. This contributes towards the safety as well as the economic planning and execution of the Gumara irrigation projects.

1.7 Application of the result

The data / information generated through this study may highlight certain aspects of the dam foundation soil and rock characterization and its suitability. This contributes towards the safety as well as the economic planning and execution of the Gumara irrigation projects. Through the present study an attempt has been made to highlight the problems associated with the engineering suitability of the dam foundation. Thus, the present study may extend an insight for the project geologist and engineers to emphasize future investigations on the problems mentioned through the present study. Moreover, the later researchers intending to work on the same subject or in the same study area may also utilize data generated through this study.

1.8 Limitations

The present research was conducted under time, resources and the financial constraints. Therefore, results and findings of this study may be considered as indicative only. Further elaborate studies might be necessary on some or all aspects of the findings.

General overview of the study area Chapter II

2.0 Introduction

The river Gumara, on which the present Gumara irrigation project is proposed, forms a part of Lake Tana sub-basin under Abbay River Basin. The Abbay basin is the most important river basin of Ethiopia. It accounts for almost 20 percent of Ethiopia's land area; 50 percent of its total average rainfall; 25 percent of its population; 39 percent of national cattle herd; and over 40 percent of cultivated land and crop production. The basin is a key surplus food producing area of Ethiopia. It is therefore, critically important in terms of national agricultural economy and for national food security.

The Gumara river rises in the high mountainous area south and east of the town Debre Tabour at an approximate elevation of 3050 meter. After flowing through a length of 132.5 km, the river joins Lake Tana about 35 km north of the town Bahir Dar. It has a drainage area of about 1893 sq.km.

2.1 Location and accessibility of the study area

The Gumara Irrigation project is found in Northern Ethiopia, Amhara Regional state, south Gondar Admistrative Zone. The area is accessible with vehicle, which is about 50km from the capital town Bahir Dar. From Bahir Dar to Woreta town, the first 24 km is on Asphalt road. The rest however, is poor network of all weathered road, which is accessible only in dry seasons.

The geographical location of the dam site is 37˚ 47′ 44″ E longitude and 11° 44′ 30″ N latitude. The dam site is located on the Ethiopian Mapping Agency topo sheet No. 1137-B4 of MEBEREJ at a scale of 1:50,000. Fig. 2.1 shows the location and accessibility of the Gumara project area.

2.2 General layout of Gumara dam project

The proposed dam height on the Gumara river is a 65 meter having a gross storage capacity of about 186 MM^3 with 500 m crest length. The location of the site lies on in main stream of Gumara river before its confluence with Sendega Gumara. Two canal systems off taking on both banks from the dam to irrigate 13.976 ha of land.

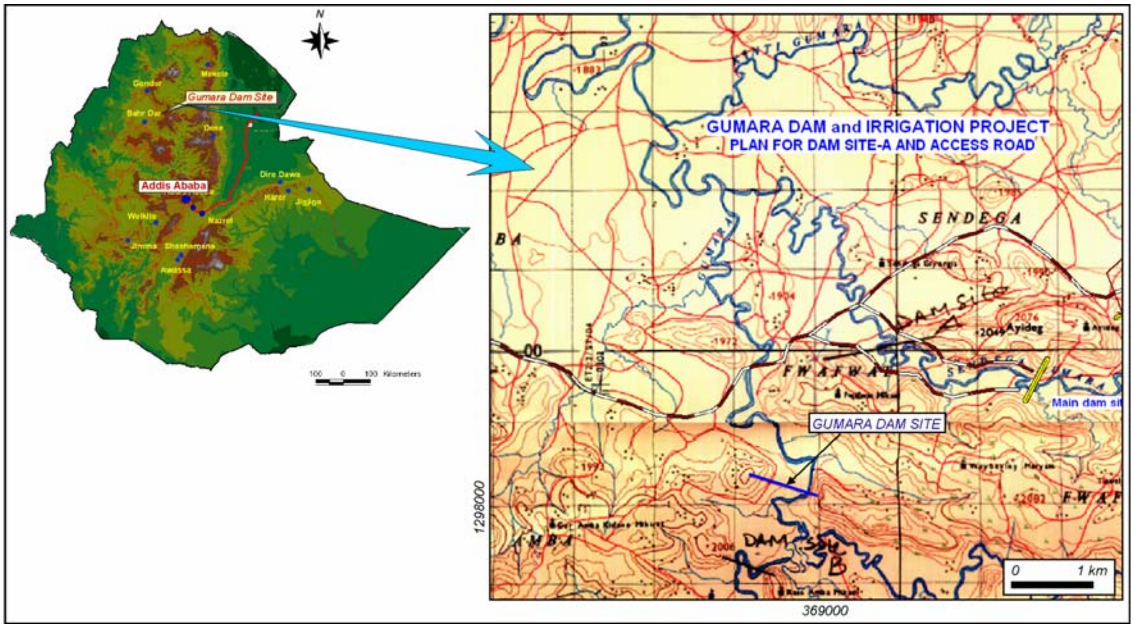

Fig. 2.1 Location of the Gumara Project site

2.3 Topography and Drainage of the Basin

The geology, the regional tectonic activities, erosional and depositional processes determine the general geomorphologic set up of a given area. The thick basaltic flow, hills together with their top plateau at the summit and the vast flat low lands at their foot is the normal topographic features in the dam site area and in the surroundings.

The Lake Tana sub-basin falls on north Western part of the Ethiopia. It is characterized by large, flat to very gently sloping plain bordering the lake on the north east and by an extensive area of gently rolling hilly uplands on the South. Recent lava flows, hilly rocky land, low marshy areas and mountain terrain comprise a sizable portion of the landscape.

The Gumara river upper and middle part of the catchment is characterized by mountainous, highly rugged and dissected topography with steep slopes. The lowest part of the catchment is characterized by valley floor with flat to gentle slopes where the river over flows its bank during rainy seasons. Fig. 2.2 shows the topography and drainage of the area around Gumara river basin.

Most of the study area is characterized by homogenous flat topography (plain), with slope ranging from 1 to 2%. The foot slopes are relatively steeper (approximately 5 to 10%). The extensive flat plain as numerous subdued facets (depressions), which are predominantly wetlands.

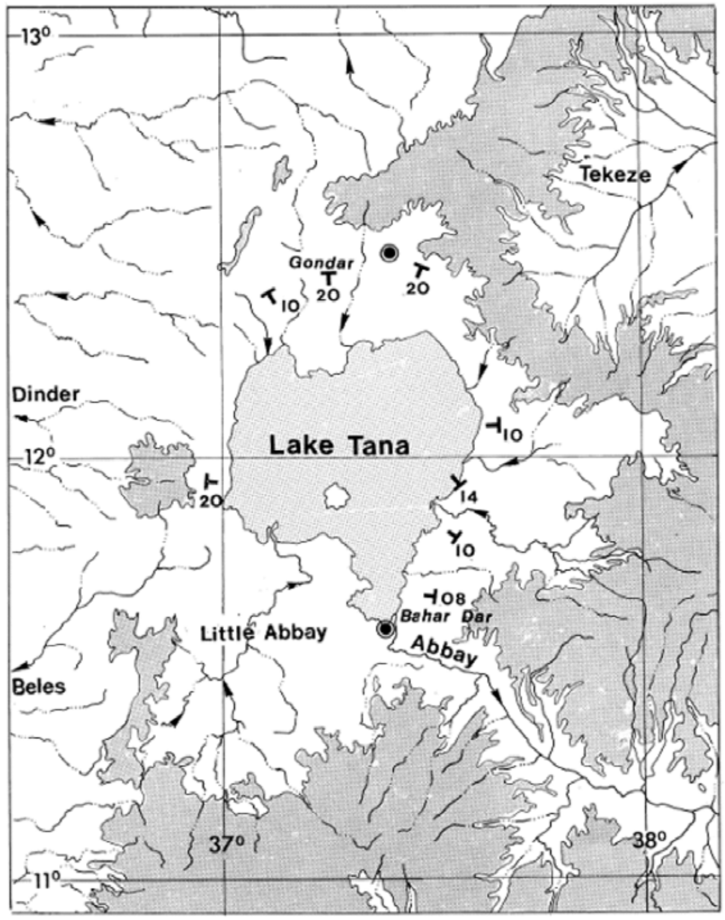

Fig. 2.2 Topography and drainage of the area around Gumara river basin (Source J.Chorowicz et al, 1998)

The proposed dam site is a wide U shaped valley with wide flat topography (<2% in slope grade). The elevation of the floor varies from 1920m at the river bed near the foot of the right abutment to 1930m near the foot of left abutment along the dam axis. This is characterized by wide valley floor of thick alluvial deposits. The valley is stretched almost north south direction. St. Michael church, found on relatively elevated plateau at the central part of the reservoir. The South and South east extreme of the area are connected to this church plateau that forms the ridge to Guramba. The northern extreme bounds the down stream area having a relatively high plateau in which church Teodros Michael is found. (Geotechnical report of Gumara dam, 2006).

The Lake Tana Sub basin lie on the North Western Ethiopia and in the north belonging to the Abbay basin, has an area of 198,000km^2. It has a drainage area of approximately 15,000 sq.km, situated at an elevation of 1786 meters a.s.l. The Lake Tana is country's largest fresh water Lake, covering about 3,000 sq. km on the central plateau of Ethiopia. The main tributary rivers are Gilgel Abbay entering the lake from the South West, Megech river from the north, and Rib and Gumara rivers from the east side. All of these rise in the high lands surrounding the lake. It has a catchment of 17,500 sq. km.(WAPCOS report, 1990)

The river Gumara raises in the high mountainous area South and east of the town Debre Tabour at an approximate elevation of 3050 meters. After flowing through a length of 132.5 km, the river joins Lake Tana about 35 km north of the town Bahir Dar. It has a drainage area of about 1893 sq.km.

2.4 Fluvial Geomorphology of the study area

The Gumara dam site is located in the lower part of Gumara River Basin, in North Western plateau. The main Gumara course flows North West to Lake Tana following the low-lying plain. Meandering features characterize the river around the dam site. Hence, the river represents a mature stage of river, which has cut down to an approximately graded profile. It can be said to have a profile of equilibrium. Side cutting is prominent in the channel which resulted in the development of meander, the present channel geometry is the result of deposition of sediments along the slip of side and erosion at the under cut side (Preliminary fluvial geomorphology report of Gumara Dam, 2006).

Meanders usually change their position. They move both down stream and to the side. The sideway movement occurs because at the bends the swiftest currents shift towards the outside bank causing erosion at the out side of the curve. In this way the Gumara river migrated side ways and slightly down stream by eroding its outer bank. In general Gumara river channel around the study area is classified as mature river channel and possesses the following characteristics:

- Position: Found in plain lying adjacent to the mountain range
- Erosion: Down curing is slight and side cutting is dominated
- Valley: Broad and trough shaped
- Longitudinal profile: Water falls and rapids are absent. Valley bottom is graded so that the longitudinal profile exhibits a relatively smooth curve. The gradient is moderate.

• Stream pattern: The river moves in meanders

2.5 Climate

Abbay (or Blue Nile) Basin, including its tributaries, being one of the largest basins in Ethiopia. The general elevation of the plateaus with in Abbay is above 2000 meters above sea level; with a number of isolated mountains that rise more than 1000 meters above the general surface.

The basin can generally be described as temperate at the higher elevations and tropical at the lower elevations. The Qolla zone lies below 1800 meters and has an average annual temperatures ranging from 16°C to 20°C. The Woina Dega zone lie between 1800 and 2400 meters, and has average annual temperatures ranging from 20^0C to 28^0C. The Dega Zone, above 2400 meters has average annual temperatures ranging from 10°C to 16°C.

The climate of the Gumara project area catchment results from its location and elevation (1785 to 3467 meters). The climatic elements such as temperature, relative humidity, wind speed and sun-shine hours are primarily required in estimating the potential evaporation. Five meteorological observation stations are located in and around the Gumara project area out of these two stations, namely, Bhair Dar and Debre Tabor are class I stations where observations of Rainfall, Temperature, Relative humidity, sun shine duration and wind speed are available. Bahir Dar station has Evaporation Measurements using pitche Evaporimeter. While Debre Tabor station is under operation since 1951, Bahir Dar station is operating since 1961. The remaining three stations namely, Addis Zemen, Gumara and Wereta are class III stations where only Rain fall and Temperature measurements are available (Table 2.1). The Gumara observation station is the only station that lies with in Gumara catchment. However, this station has become non-operational from April 1991.

Table 2.1 Details of Metrological observation stations located around the project area

No.	Station name	Latitude north	Longitude East	Altitude (m)	Period	Class	Years
1	Addis Zemen	12:07	37:52	1850	1963-2004	3	42
2	Bahir Dar	11:36	37:25	1770	1961-2004	1	44
3	Debre Tabor	11:53	38:02	2690	1951-2004	1	54
4	Gumara	11:50	37:38	1880	1978-1991	3	14
5	Woreta	11:55	37:41	1810	1969-2004	3	36

The rain fall in the Gumara catchment as well as in it's surroundings is Uni–modal type. Most of the rainfall is concentrated during the months of June to September covering with virtual drought from November through April. The four wettest months cover 85 percent of the total annual rainfall. The dry season being from October to May has a total rainfall of about 15% of the mean annual rainfall. The mean monthly rainfall for stations in the project area is given in Table 2.2. The rainfall data collected from Bahir Dar is relatively more reliable compared to the data set collected from other station because of it's long record length shorter time interval of observation.

Table 2.2 Mean Monthly Rainfall in and around the Gumara Project area.

Station	Jan	Feb	March	April	May	June	July	Aug	Sep	Oct	Nov	Dec	Total
Bahir Dar	2.8	2.2	7.3	23.9	78.6	189.7	430.7	391.3	193.7	93.8	19.8	3.5	1437.0
Addis Zemen	2.2	4.5	3.2	17.7	61.5	175.3	448.7	387.5	176.5	59.8	11.0	6.4	1353.9
Gumara	0.3	1.2	8.7	13.9	73.5	136.7	402.9	339.7	138.3	85.1	3.4	1.1	1204.8
Wereta	0.3	1.0	3.8	16.7	68.6	169.5	374.0	377.1	166.2	50.0	8.2	1.4	1236.7
Debre Tabor	6.8	8.9	30.5	37.0	83.9	180.4	446.5	461.0	188.2	82.2	34.2	96	1569.1

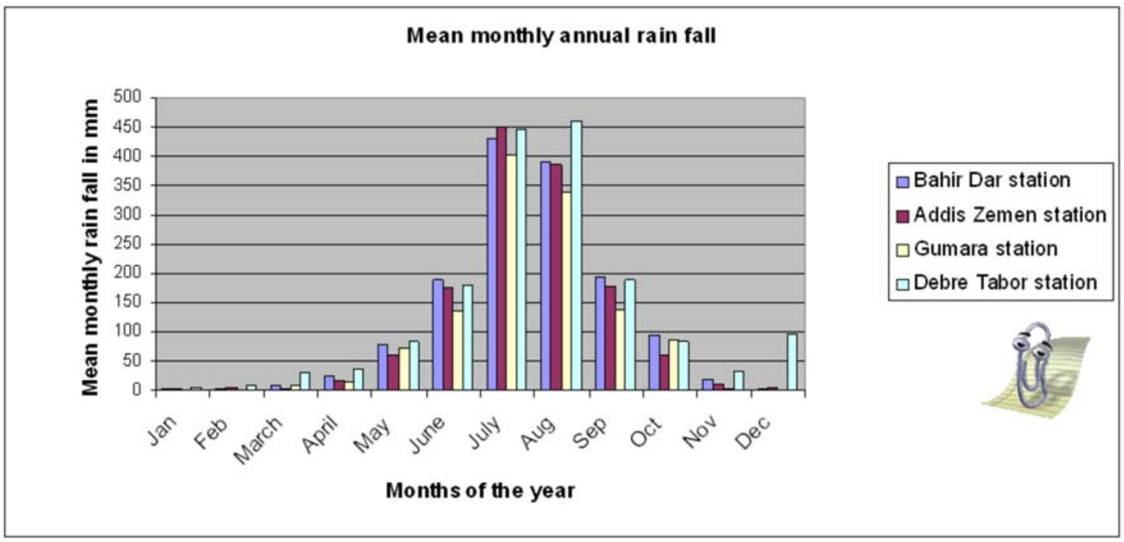

Fig. 2.3 Mean monthly annual rainfall as recorded at different meteorological stations around study area

In the farm area (Bahir Dar station), the mean monthly temperature variations through out the year are minor, being 16^0C in December to 21.3^0C in May.

Humidity values vary between 46 percent in April to 80 percent in August. Wind speed values vary 1.1 meter per second in October and November to 2.5 meter per second in June.

Mean sun shine duration is 9.7 hours in January and is reduced to 4.8 hours during July and Pan Evaporation rates are between 3.3 mm per day in December and 5.1 mm per day in

April. The mean Annual temperature, Humidity, Wind speed and pan evaporation is given in Table 2.3.

Table 2.3 Summary of climatic data and potential Evaporation as recorded at Bahir Dar station

Description	Jan	Feb	March	April	May	June	July	Aug	Sep	Oct	Nov	Dec	Annual
T(^{0}C)	16.6	18.0	20.4	20.9	21.3	20.0	18.6	18.4	18.7	18.9	17.9	16.3	18.8
H (%)	57	50	47	46	57	65	78	80	74	68	63	59	62
W(m/s)	1.7	2.0	2.1	2.0	2.2	2.5	2.1	1.7	1.5	1.1	1.1	1.3	1.8
SD (hrs)	9.7	9.5	9.0	9.2	8.2	7.1	5.3	4.8	6.6	8.5	9.5	9.5	8.1
Eo (mm/day)	3.6	4.2	4.8	5.1	4.7	4.2	3.3	3.1	3.5	3.7	3.5	3.3	3.9
T – Temperature, H – Humidity, W – Wind speed, SD – Sun shine duration, Eo - Pan Evaporation													

2.6 Hydro chemistry of Gumara River

Water is one of the most important input for realizing and sustaining high production. However, its management transportation to the far land method and frequency of irrigation and quality of the water are intimately related to the development of water logging and soil salinity. In addition to this knowing the chemical constituent of the river water, ions and cations, is used to know their effect on concrete structures.

2.6.1 Irrigation water quality

The main criteria for assessing the quality of water for irrigation are salinity, sodicity hazard and a specific ion effect. The most important criteria regarding salinity are the total salt concentration. Since there exists straight line correlation between the electrical conductivity (EC) and total salt concentration of water. So, to evaluate the salinity hazard is to measure its EC. From the EC values, the total salt concentration can be calculated using the formula:

$$\text{Salt concentration, me/l} = \frac{EC\ (\eta\ S/m)}{100} \qquad \text{……..eq.2.1}$$

Many times, salt concentration is also expressed as total dissolved solids (TDS), which are determined gravimetrically. TDS can also be calculated from EC of the water using the following equation.

$$\text{TDS, mg/l} = EC\ (d\ S/m) * 640 \qquad \text{…….eq. 2.2}$$

On the basis of salt concentration, US salinity laboratory staff (Richards, 1954) divide the irrigation water in to five classes (Table 2.4).

Table 2.4 Salinity hazard of irrigation water

EC of irrigation water, η S/m	Salinity class	Salinity hazard
100-250	C1	Very low hazard
250-750	C2	Low
750-2250	C3	Medium
2250-5000	C4	High
> 5000	C5	Very high

Most of the irrigation water falls into the range of 150 to 1500 η S/m. Though water having EC value above 1500 η S/m can cause serious damage.

In addition to the total salinity, the tendency of irrigation water to generate excessive levels of exchangeable sodium, which adversely affect the soil physicochemical properties (when the concentration of sodium far more greater than the concentration of the other cations affect the structure of the soil by reducing it's permeability). The useful parameter for expressing the sodium hazard of irrigation water is SAR. Sodium Adsorption ratio of water is calculated from equation as described by Richard (1954)

$$SAR = \frac{Na^{+}}{(\sqrt{(Ca^{2+} + Mg^{2+}}/2)} \quad \text{……..eq.2.3}$$

Where Na^{+}, Ca^{2+} and Mg^{2+} are in me/l. From the SAR of the irrigation water, ESR and ESP of the soil can be calculated using Equation

$$\frac{ESP}{100} = \frac{ESR}{1+ESP} = \frac{(-0.0126 + 0.01475\ SAR)}{1+(-0.0126 +.01475)} \quad \text{……..eq.2.4}$$

On the basis of SAR, the irrigation water have been divided in to 4 categories.

Table 2.5 Hazards of irrigation water

SAR of irrigation water me/l	Sodicity Class	Sodicity hazard
< 10	S_1	Low
10-18	S_2	Medium
18-26	S_3	High
>26	S_4	Very high

(Source: Richards, 1954)

During the present study, one water sample were collected form the Dam site for laboratory test. Based on the laboratory result of the current water sample the Analysis was carried out in Water Works Design and Supervision Enterprise Laboratory and the results are presented in Table 2.6.

Table 2.6 Result of the chemical quality of the water from Gumara dam Site

Parameters	Result
Total Dissolved Solid 105^oC (mg/l)	130.0
Electrical Conductivity (µS/cm	197
P^H	7.77
Sodium (mg/l Na)	7.8
Potassium (mg/l K)	2.8
Total Hardness ((mg/l $CaCO_3$	92.4
Calcium (mg/l Ca)	30.8
Magnesium (mg/l Mg)	3.78
Total Iron (mg/l Fe)	-
Manganese (mg/l Mn)	-
Fluoride (mg/l F)	-
Chloride ((mg/l Cl)	5.96
Nitrite (mg/l NO_2)	-
Nitrite (mg/l NO_3)	0.87
Alkalinity (mg/l $CaCO_3$)	95.4
Carbonate (mg/l CO_3)	Trace
Bicarbonate (mg/l HCO_4)	111.39
Sulphate (mg/l SO_3)	0.86
Phosphate (mg/l PO_4)	0.15

Table 2.7 Water quality of Gumara River around the Dam site

Sample location	SAR	EC(η S/m)	HCO_3, mg/l	Na%
Gumara	0.353	197.0	111.39	25.33

Thus from the water quality results of the Gumara river (Table 2.7) it may be concluded that the electrical conductivity (EC) value is 197.0. Thus the salinity hazard of Gumara water for irrigation according to US salinity laboratory staff (Richards, 1954) falls into C1 salinity class with Very low hazard salinity hazard. Also, the Sodium Adsorption ratio (SAR) value for Gumara water is 0.353. This implies that Gumara water for irrigation falls into S1 sodiciti class and have low sodicity hazard. Thus it may be finally concluded that the Gumara water is suitable for irrigation purpose.

2.6.2 Effect of Gumara river water on concrete structures

The coexistence of sulphate and chloride ions in water causes deterioration of reinforcement (concrete or other steel structures). The corrosivity of the water can be determined from the corrosivity ratio coefficient, CR (Mahadevaswamy, 2002). In corrosive subsurface water conditions, while doing excavations, a proper precaution has to be taken to reduce the effect of corrosion, especially in permanent excavations. According to Mahadevaswamy (2002), the value of the corrosivity coefficient can be determined from the following expression;

$$CR = \frac{0.028Cl + 0.021SO_4}{0.02(HCO_3 + CO_3)} \qquad \text{……..eq. 2.5}$$

Thus if the CR value is >1 the water is corrosive. Therefore, it is desirable to determine the corrosivity ratio coefficient, CR for the safety of the concrete and other reinforcements, particularly when the foundations are laid down in the zone of subsurface water.

The results indicates that the CR value for the Gumara river water is 0.083 which is lower than 1.0. This implies that the Gumara water is non corrosive in nature.

2.7 Ground Water Condition

Knowing the ground water condition is essential for various engineering structures. For the dam project the engineering significance of the ground water are:

(i) The retention of water in the reservoir basin is determined by the pezometric level and ground water divide position with respect to the reservoir level in addition to the geology of the area.
(ii) The presence of water in the rock mass cause water pressure with in the discontinuities, which result in reduction of cohesion and may cause problems of instability of abutments.
(iii) Ground water may pose problems in construction.
(iv) Ground water fluctuation may cause uplift problem in the foundation. In the project area two hydrostrategraphic units are identified.
(v) The fractured volcanic rocks out cropping in the abutment have steep morphology. Due to this morphology the ground water tables in this unit is not shallow and are poor aquifer.
(vi) The quaternary sediments alluvial and colluvial materials cover low lying areas. This unit occupy at morphologically suitable positions for recharge, infiltration, storage of significant amount of ground water.

During the site investigation done by the project Authorities in most of the boreholes with in the low lying area have a ground water measured at shallow depth. In order to know the exact position and fluctuation, the groundwater has been monitored by installing piezometers in two boreholes (BH-GD4 and BH-GD5) (Table 2.7). A simple stand pipe consisting of a PVC tube with slotted end and surrounding by a granular filter in the expected aquifer zone was placed in BH-GD4 at depth of 21 meter. The other pizometer was installed in BH-GD5 at a depth of 18 meter. As the observation from the measurement by the project Authority the result show the level of ground water was the same as before installation.

Table 2.8 The depth of ground water as observed in boreholes

Location	Borehole name	Depth of Ground water below the surface
Low lying area (Valley floor)	BH-GD4	4.5 meter
Low lying area	BH-GD5	9.5 meter

From the table the ground water table is shallow at the center of the valley along the dam axis (dam foundation area) therefore during trench excavation for the preparation of the dam foundation pumps should be available.

2.8 Seismicity of the area

The seismic risk maps are often published in national building codes, which recommend the precaution to be taken in each rank of hazard shown in the risk map. Judgment and modification to the expected intensity can be depending on the ground condition because thick soil deposit and bed rock do not have similar response for the same earthquake magnitude. According to the seismic risk map of Ethiopia 100 years return period, 0.99 probabilities by Laike Mariam Asfaw, (1986) the country is divided into zones of approximately equal seismic risks based on the known distribution of the past earthquakes.

Fig. 2.4 indicates that the Gumara dam site is located within intensity Zone 5 with ground acceleration of 0.02g. Hence, the project area lies in low seismic risk hazard zone thus the probability of occurrence of earth quake in this area is least. According to Johnson (1988), these seismic intensity zones are related to the ground Acceleration as follows;

Intensity (MM)	<5	5	6	7	8
Ground Acceleration	0.01g	0.02g	0.05g	0.1g	0.2g

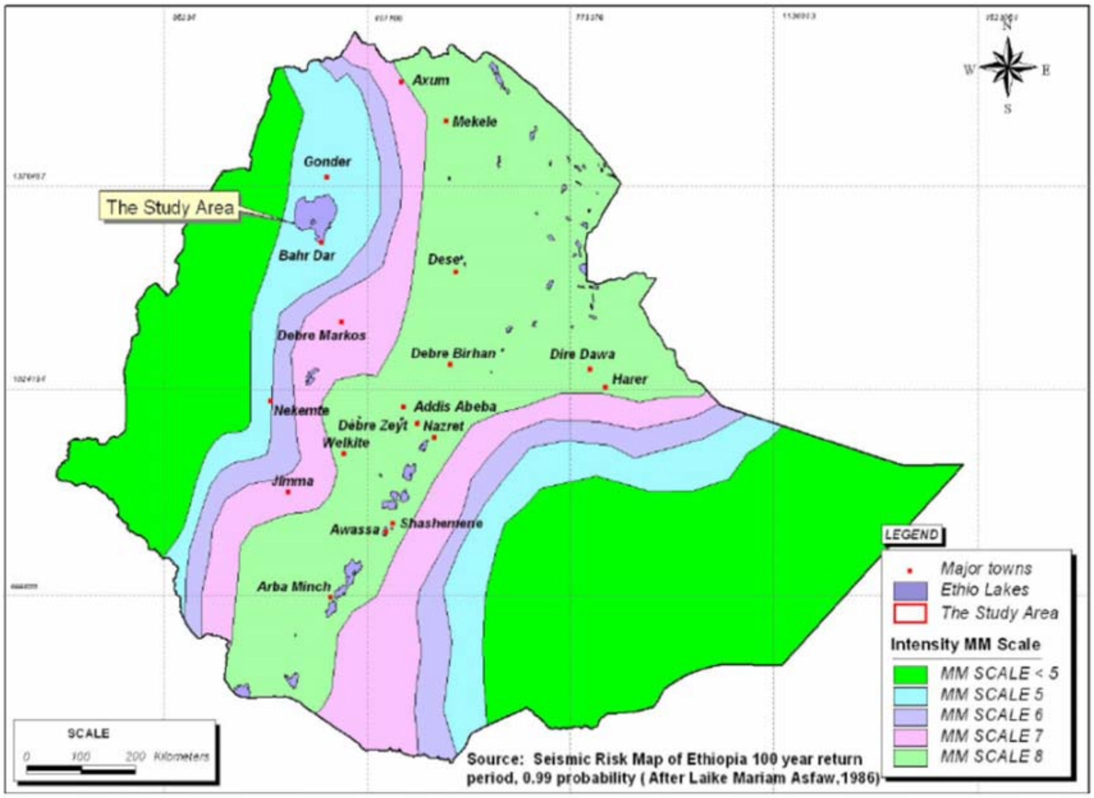

Fig. 2.4 Seismic risk map of Ethiopia 100 years return period, 0.99 probabilities by Laike Mariam Asfaw, (1986)

2.9 The storage capacity of the reservoir

To calculate the reservoir capacity of the proposed Gumara dam 1:50000 scale top sheets were used. The countours in an interval of 20m were digitized upto maximum reservoir level. For digitization purpose ArcGIS software was employed. Later the area of the successive contours were determined using the software capability.

Table 2.9 Computed average capacity of the reservoir of the proposed Gumara dam

Parameter	Dimension		
Dam height in meter	20	40	60
Average area (m^2)	534,765.1	4,074,010.2	7,942,144.99
Storage capacity in Mm^3	5.35	51.43	171.6

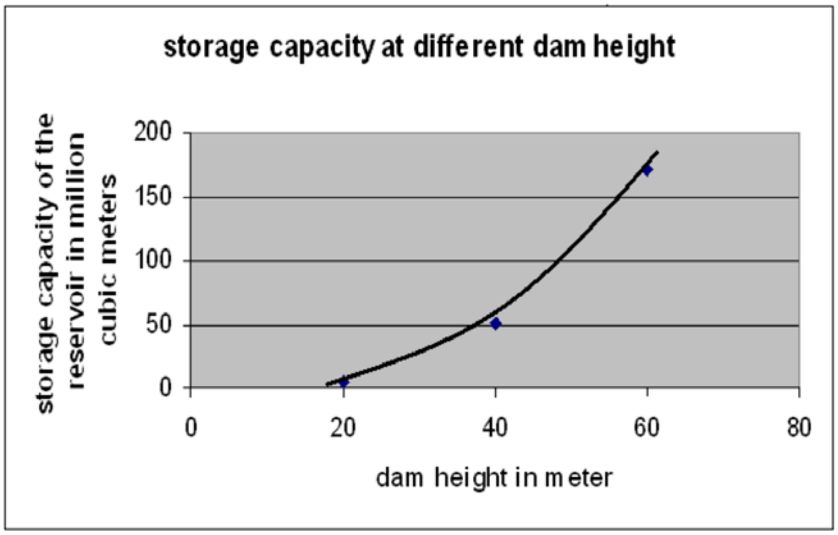

Fig.2.5 Computed storage capacity at different dam heights

Thus by using planimeter area for successive contours total reservoir volume at different levels were calculated. On the proposed dam axis of Gumara three contours at equal interval of 20 meters above the datum has been worked out. Table 2.9 and Fig.2.5 show the computed storage capacity at different dam heights. Thus the maximum storage capacity at maximum reservoir level comes out to be 171.6 M m^3.

Geological setting Chapter III

3.1 Regional geological setting and structures

The Ethiopian plateau is not a simple, undeformed structural block. In the North West, it contains the Tana basin, a faulted depression located between the Erosional escarpment overlooking the Sudan plains to the west and, to the east, the tectonic escarpment of the plateau margin overlooking the Afar depression.(J. Chorowicz et al, 1998)

As J. Chorowicz et al, 1998 study, the Tana basin lies at the convergence of three grabens:the Debre Tabor graben from the east, the Gondar graben from the north-northwest, and the Dengel Ber graben from the south-southwest. The grabens are impressed in the mid-Tertiary flood basalt pile; lack of exposure of the sub-volcanic terrain hides evidence for any earlier structural history. Zanettin et al. (1980) propose that Eocene pre-Afar graben faulting extended northwest across what is now the N Ethiopian plateau to the Sudan border. This 'Ashanghi graben' is considered to have formed a westward projection of an Eocene Gulf of Aden rift.

The northern sector of the Tana basin is an asymmetric graben (the Gondar graben), the western boundary fault zone being framed by clearly defined NNW–SSE- to N–S-trending faults. Farther north, the Gondar graben is cut by the West Tana escarpment. In its southern sector the Gondar graben floor preserves olivine basalt flows and overlying lignitiferous lacustrine sediments. Together these form a thin cover upon faulted mid-Tertiary basalts (Usoni, 1945; Tezera and Heeman, 1983). The younger basalts have yielded an 8–10 Ma radiometric apparent-age range (Yemane et al., 1985). They mantle older faults that now lack morphologic expression, whereas the youngest faults cut across these mid-Tertiary flows and retain identifiable scarps. The southern sector of the Tana basin comprises the Dengel Ber graben. Its western border is framed by NE–SW- to NNE– SSW-striking faults. The graben is largely masked by the profuse volcanism that has built up the Danghila plateau. Grabham and Black (1925) report outcrop of probable Mesozoic sandstone on the eastern flanks of Mt. Belaya, consistent with this discovery. The indication is therefore one of shoulder uplift along the western side of the Dengel Ber graben, in concert with observed block faulting. This shoulder uplifts favoured location of the West Tana erosional escarpment along the western border of the graben, resulting in inversion of the graben topography. The southward continuation of the Dengel Ber graben remains to be elucidated.

Over the eastern side of the Tana basin, alluvial cover partly masks the Debre Tabor graben. North of Debre Tabor, E–W-striking faults with a morphologic expression indicating southerly downthrows, can be traced farther east, here they turn to ESE–WNW before termination against the Guna shield volcano. The faulting then reappears on the southeastern flanks of the shield. On the opposing, other side of the Debre Tabor graben, a complementary set of E–W faults veers to the east-southeast farther east, and is downthrown to the north.

According to the description made to Mohr, P.A report 'The geology of Ethiopia, 1961' immediately following and contemporaneous with the major uplift of the continental mass, immense quantity of lava were extruded from fissures and volcanic centers. Although considerable denudation of Mesozoic rock had also taken place before the eruption of the volcanic rocks, the area was not deeply eroded.

The plateau volcanics or the Trap series essentially predates the rift faulting and therefore forms the high Ethiopian and Somali plateaus. The Aden (Younger) volcanic series post dates the rift faulting and largely confined to the floor of the rift valley, the Lake Tana basin and to younger, manifestly fresh lava volcanic cones, and explosion crater which in some places are still active.

The Lake Tana sub basin is formed by structural deformation, erosion and extrusions of volcanic rocks. It is surrounded by volcanic mountains, consisting primarily of basaltic lava flows with associated with tuff, trachytic and rhyolitic rock types. Some of the tectonic features associated with the formation of lake Tana basin are northeast south west faults and dykes of the lake; down warped, faulted, and steeply dipping volcanic rocks along the south western side of the lake and rift-fault structures NW of the Lake Tana which cut through the structural basins in which are deposit of lake beds (WAPCOS report, 1990).

3.1.1 Mesozoic sedimentary rocks

The Abay River Basin regionally shows a thick succession of Mesozoic sedimentary rocks. These generally horizontal beds of sedimentary rocks are exposed in the numerous valleys within the Abay Basin. The succession is made up of the Adigrat Sandstones, Gohatsion

Formation, Lagajima Limestones, Mugher Mudstones and Debre Libanos Sandstones (Getaneh, 1981,1991; Russo *et al*, 1994).

Although they are not exposed in the area of the lake Tana basin, at least a part of the Mesozoic sequence is expected to be encountered at great depth below the volcanic strata. The northern extent of the individual Mesozoic formations is not clearly defined/observed, implying that maybe not all the units have reached the area of Tana basin. Grabham and Black (1925) report outcrops of probable Mesozoic sandstone on the eastern flanks of Mt. Belaya.

This sequence, in turn rests on the basement complex that includes Precambrian granites, gneisses, schists, marble and phyllites.

3.1.2 The Volcanic Rocks

The rocks in the vicinity for the Lake Tana are mainly extrusive volcanic rocks representing three or more phases of volcanic activity. To a limited extent fluvio-lacustrine sediments are also represented.

The three major divisions or phases of volcanism that are recognized are:

3.1.2.1 The 'plateau Volcanic' rocks

The 'Plateau volcanic' consisting of extensively, horizontally bedded, massive generally basaltic lava flows that cover the plateau. Which are the lower and the oldest of the Tertiary age volcanic rocks that comprise the bulk of the flat-lying beds. The series is a unit of very thick lava flows, mostly of basaltic composition; with more silicic differentiates near the top. A division is made between the lower, thick, massive Ashangi group of plateau basalt which are extraordinary uniform in composition, and the upper, more silicic lavas of Magdela group. Disconformity between the two groups has not been definitely proven. A slight unconformity is indicated between the Trap Series and underlying Mesozoic sedimentary rocks as well as irregular contacts due to erosion of Mesozoic sediments. The age of the Ethiopia Trap Series, based on paleontology, is between the end of cretaceous and Pliocene.

The greater portion of the Ethiopian plateau is covered by the extensive volcanic rocks. In general these rocks crop out or underlie the surface in all parts of the country where altitude exceed 2400 meters. “Plateau Basalt” usually described them chronologically as “Old Volcanic formation”.

3.1.2.2 The ‘volcanic Mounts’ rocks

Late phase volcanic mounts and intrusive dikes, plugs etc. Mostly more acidic varieties of tracyitic or rhyolitic, phyroclastics of tuff and breccias etc. These rocks generally associated with high mountains or intrusion penetrating the lower ‘Plateau’ volcanic rocks. Some of these intrusive rocks are in, or they are resting on, Precambrian rocks.

They are perhaps remnants of extrusive erosion and are believed to be related to the some later phase of volcanic activity that formed the high mountains and intrusion.

The ‘Volcanic Mounts’ generally they have steeper dips rising to the vent areas (The higher mountains rising above represent the dying, explosive and more acidic viscous phases of volcanism. Associated with this phase numerous dikes and plugs generally of trachytic composition.)

These rocks occur on the east, north and west side of Lake Tana. West of the lake, the generally flat beds have been tilted so, as to dip about 30° south easterly the basin.

3.1.2.3 Quaternary volcanic rocks

‘Younger’ very late phase recent eruption of volcanic rocks have occurred innumerous places in the Blue Nile river basin. In the Lake Tana sub-basin basaltic lava from numerous small, widely spaced craters and cinder cones covers an area of several hundred kilometers, particularly on the south and southwest side of Lake Tana and extend down stream along the Blue Nile valley. Other recent flows are located on the east and north sides of the Lake.

This younger (Aden) volcanic series clearly more recent than the Tertiary Trappean series, its characterized by its extensive distribution.

This volcanic series is rather diversely constituted, mostly of basaltic and andesitic rocks, more or less alkaline, sometimes plagioclase-olivenitic, often doleritic, with titaniferous augite etc. The occurrence is commonly marked by acid lavas of tracyitic or rhyolitic type.

The 'Younger Volcanics' lava flows, which have been occurred after along period of erosion, such as the lava at the southern end of Lake Tana. This is the youngest phase of volcanic activity in the Lake Tana sub basin.

The Present Lake was formed primarily by this younger volcanism which dammed off the previously eroded valley and drainage system, impounding a broad, relatively shallow body of water.

3.1.3 Pliocene and Pleistocene sediments

The Chilga continental rift basin, located in northwest Ethiopia, consisting of sediments composed of claystones, siltstones and silty sandstones, volcanic ashes, lignite beds, and vertebrate and plant fossil-rich sandstones provides ample information for such a sequence stratigraphic interpretation.

Radioisotopic and paleomagnetic dating techniques were used to constrain the age of the basin filling sediments and the underlying basaltic rocks. Results indicate that the Chilga sediments were deposited between 27 – 28 Ma (Mulugeta Feseha et al., 2001; Mulugeta Feseha, 2002; Kappelman et al., 2003).

Results indicate that the Chilga sediments are composed of framework grains of mainly unaltered plagioclase, potassium feldspars, and volcanic rock fragments; the matrix is commonly enriched in kaolinite and illite-montmorillonite; and the cement is mainly composed of authigenic siderite and some iron oxide coatings. Interpretations of these characteristics suggest that the Chilga sediments were deposited in alluvial-lacustrine environment with a nearby fine-grained sediment source indicating a reducing diagenetic environment (Mulugeta Feseha, 2002).

The chelgga Lake Beds, several hundred feet thick, consist mostly of clays, water lain ash beds, siliceous shale, sandstone and lignite beds. They have been cut off sharply by rift type faults. Fig. 3.1 shows the regional geological setup of the study area.

3.2 Local Geology

The geology of the area around the project site comprises the 'Plateau volcanic' consisting of extensively, horizontally bedded, massive generally basaltic lava flows that cover the plateau in the regional geology indicated by the aphanitic basalt. Which are the lower and the oldest of the Tertiary age volcanic rocks that comprise the bulk of the flat-lying beds. The series is a unit of very thick lava flows, mostly of basaltic composition; with more silicic differentiates near the top. The age of the Ethiopia Trap Series, based on paleontology, is between the end of Cretaceous and Pliocene.

The rhyolitic plugs and trachytic basalt indicated in the local geology is correlated to the younger (Aden) volcanic series which is more recent than the Tertiary Trappean series; it's characterized by its extensive distribution. This volcanic series is rather diversely constituted, mostly of basaltic and andesitic rocks, more or less alkaline, sometimes plagioclase-olivenitic, often doleritic, with titaniferous augite etc. The occurrence is commonly marked by acid lavas of trachytic or rhyolitic type.

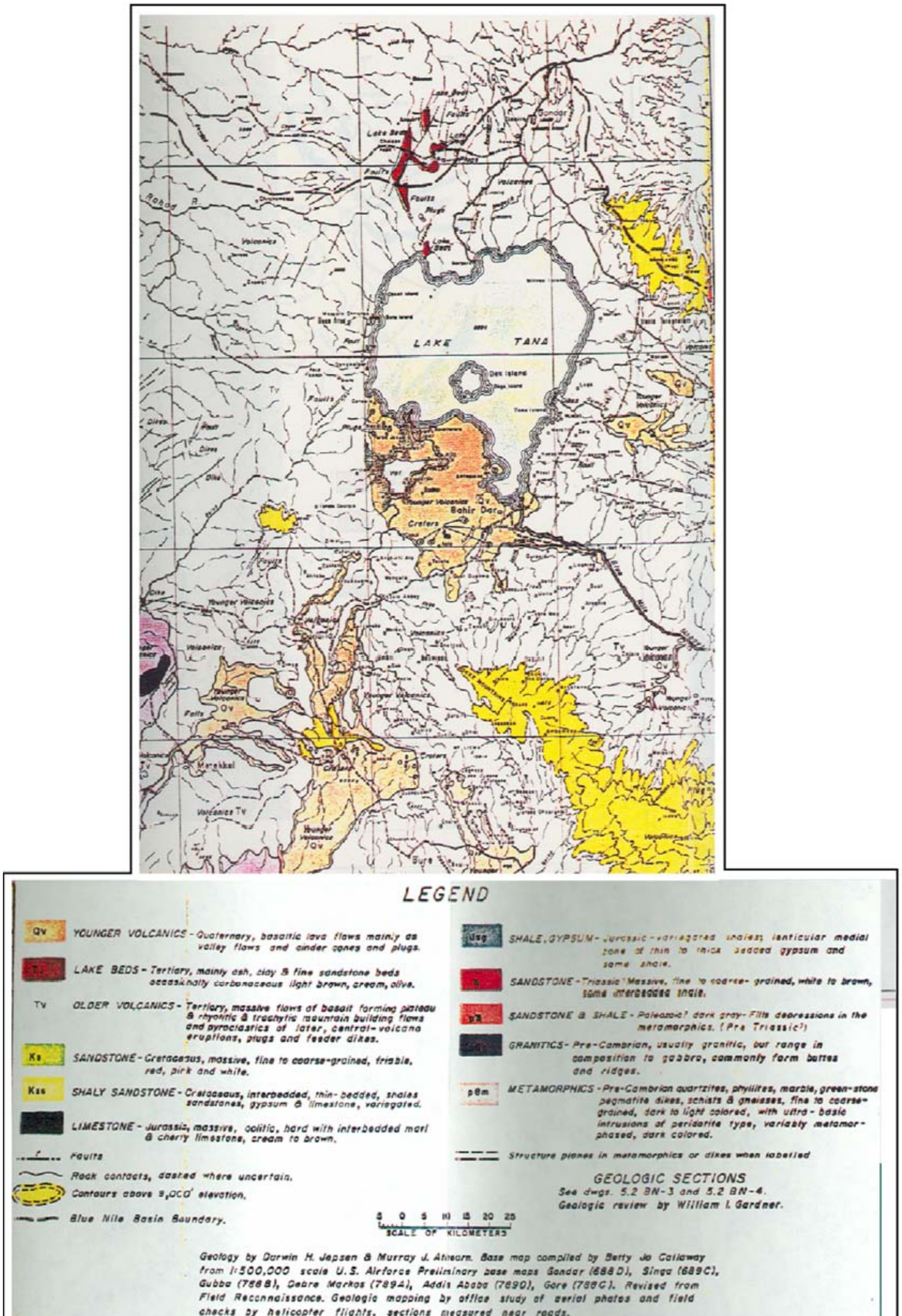

Fig. 3.1 Regional Geological Map of the study area

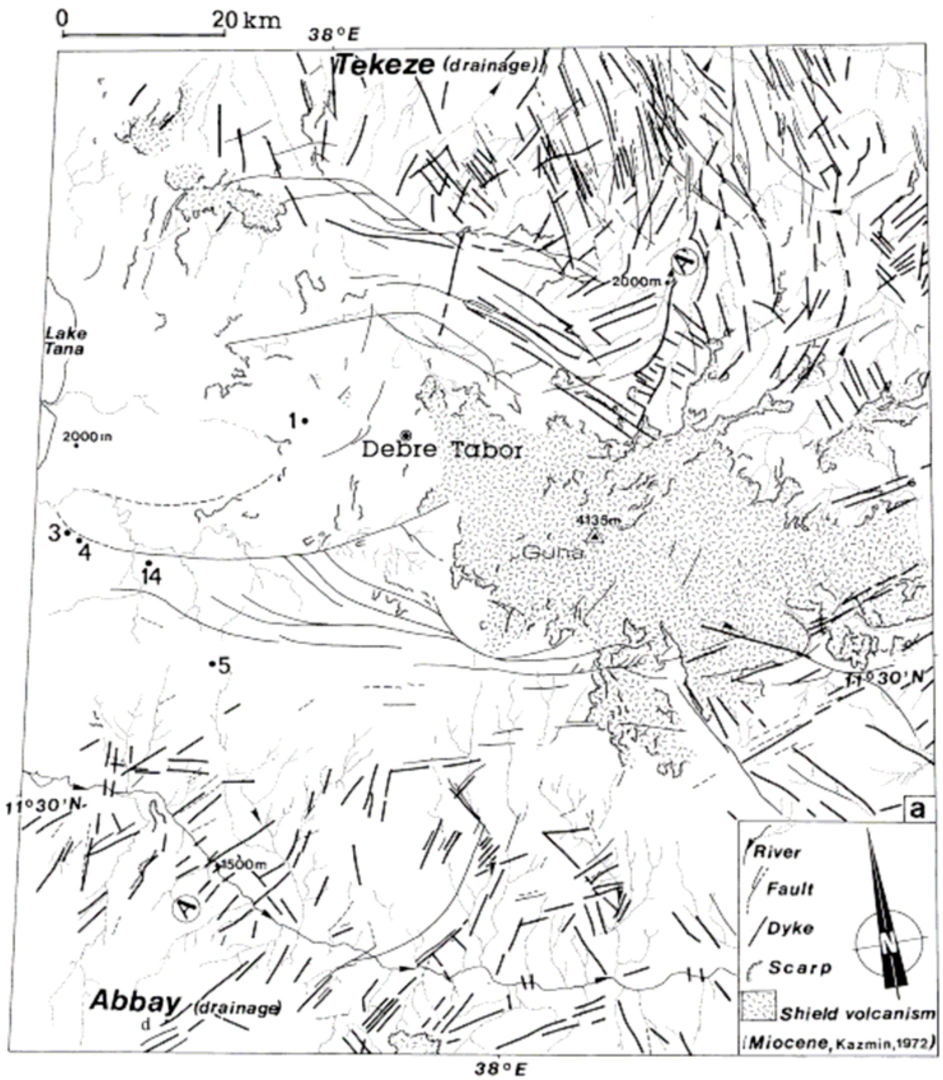

Fig. 3.2 Structural map of eastern side of Tana basin.
(Source: J. Chorowicz et al, 1998)

The quaternary volcanic formation occurs as typical cones, with craters, flows and volcanic bombs accompanied by solfateras and thermal-mineral spring. There are two thermal springs around the study area one of this is on the upper side of the reservoir and the other is in the near by area called Wanzaye. These lavas still quite unaltered by weathering.

The alluvial or the terrace deposit covers the valley of the extensive relatively plain area around the project area and all over the flood plain of Gumara namely the Fogera plain. The geology of the area around the dam site is shown through Fig. 3.3

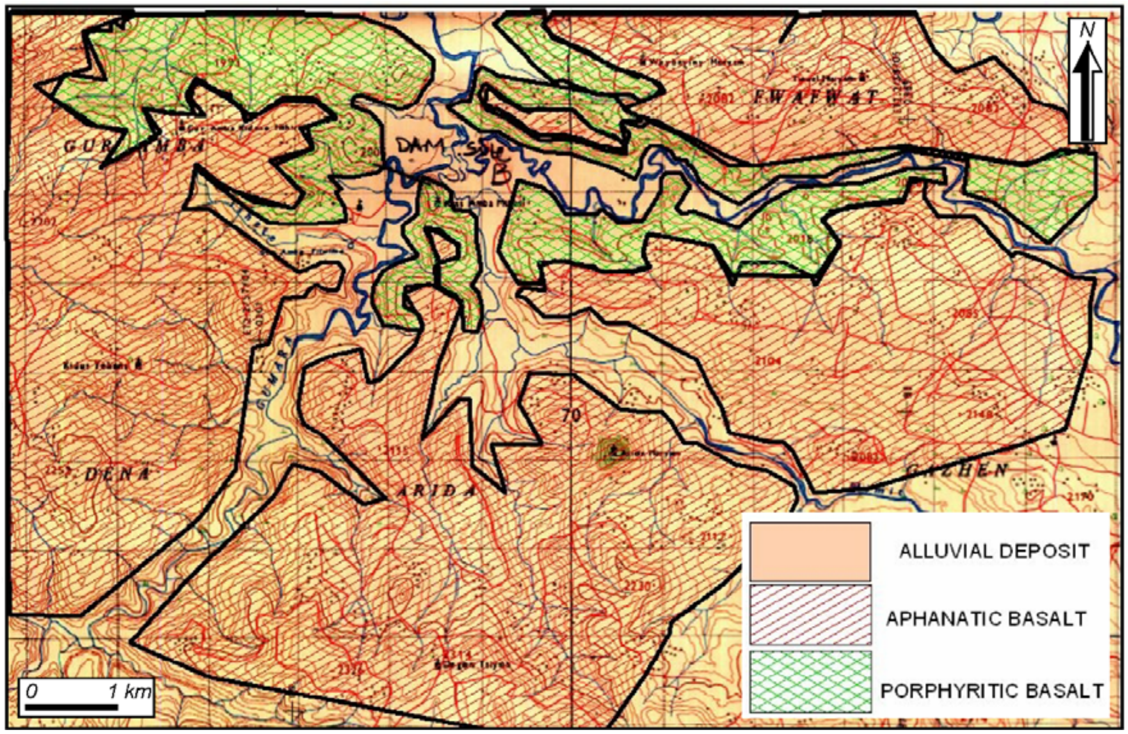

Fig. 3.3 Geology of the area around the dam site (Source; Gumara Geotechnical Report, 2006)

3.3 Geomorphology and Geology of the dam site

The thick basaltic flow, hills together with their top plateau at the summit and the vast flat low lands at their foot is the normal topographic features in the dam site area and in the surroundings.

To appraise the geology of the dam site, seven boreholes with varying depth, traverses along the reservoir and abutments and excavation of test pits along the dam axis have been carried out. As seen from the above observation feldspar rich basalt is the main geological unit in the dam site area (Fig. 3.4).

The proposed dam site lie on in the wide U. valley with wide flat topography .The Gumara river flows close to the foot of the right bank. It flows around the dam axis in 15°NE direction. The extensive flat plain has numerous subdued facets (depressions), which are predominantly wetlands. It is covered by recent alluvial deposit of silty clay soil and the river bed is covered by a rounded alluvial sandy gravel deposit.

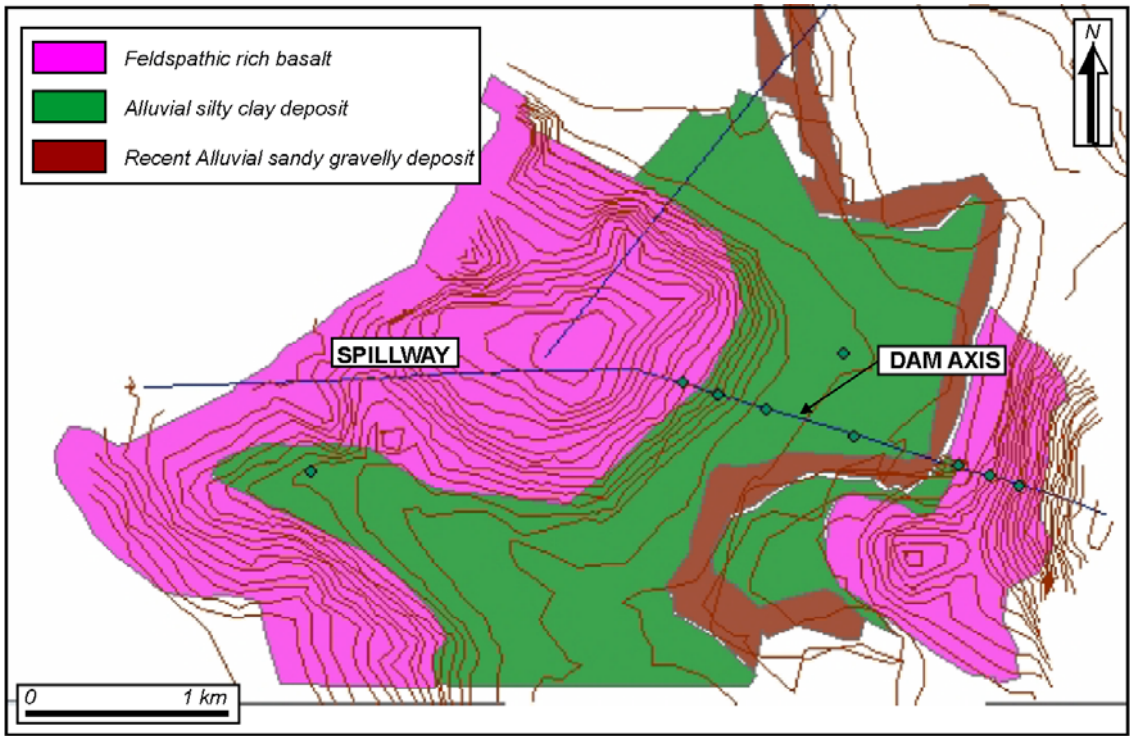

Fig. 3.4 Geology of the dam site

The right abutment is very steep ridge composed of a feldspar rich basaltic rock. The nature of this rock is highly vesicular and out crops exposed in the upper part of the ridge. In the lower part this bank is covered by weathered rock debris and sandy soil. In the middle part of this bank an aphanitic basaltic dyke exposed by crossing the feldspar rich basalt. Which have columnar in nature. The left bank also composed of feldspar rich basalt but it is relatively fresh un like the right bank.

The dam site and the surrounding areas have the following geological unit of succession.

3.2.1 Feldspar rich Basalt

This is the dominant rock type in the surrounding, both in the abutments and in the lower part of the as revealed from boreholes drilled along the dam axis. The feldphatic basalt is vesicular in nature, especially in the top part along the exposed surface and filled with secondary mineral calcite.

The exposed part of this layer is affected by network of fractures followed by physical weathering. Disintegration of the rock is frequently seen rather than chemical weathering. Well developed soil profile is not seen along this top part of the rock unit except some colluvial rock in the abutments. This unit is crossed by aphanitic dykes in both banks. From the observations from the permeability test and bore hole logs the rock becomes fresh, strong and tight below a certain depth.

3.2.2 Alluvial deposits / Terrace deposits/

Alluvial deposits are transported by running water and settled when the speed of the flowing water is no longer sufficient to carry them. This deposit is restricted to low-lying area close to river course and foot of ridges and hills. This unit consists of silt and clay, locally with pocket of medium grade sand and rounded river gravels along the river channel and banks.

This top part of alluvial deposit including the present river channel deposit has minimum thickness of 5 meter at the bank of the river to maximum thickness of about near the left abutment along the dam axis. It has a thickness of about 7m at the center of the dam axis.

3.2.3 Organic soil

The lower part of the alluvial deposit start from 7m on average depth from the natural ground level in the valley floor along the dam axis, to the maximum depth of 47 meters consist of alternate layers of thick organic black silty clay and thin layers of sandy gravel of river deposit. The occurrence of the rounded basaltic gravels in the foot of the left abutment and in BH-GD4 indicate the ancient river channel of Gumara. The thick organic deposit have bad odor due to the high organic content during the drilling time to collect the core samples but after air dried the volatiles disappear and there is no odor and the color of the core sample changed from black to gray color .

The existence of this thick organic soil is associated with the development of local marshy ponds or the rising and lowering of the lake beds in the past.

3.2.4 Dark aphanitic basalt

With a gradual contact to the feldpatic basalt, very fine aphanitic basalt occur at the about the depth 50 meters at the center of the dam axis. The nature of the contact to the feldspar rich basalt again occurs alternating the two textures.

Engineering Geological Appraisal of the Dam Foundation Chapter IV

4.0 Introduction

Both rocks and soils are involved as a foundation material or as a construction material in engineering structures. All engineering structures simple or sophisticated they are founded on geological materials. The rocks in particular, contain discontinuities in the form of bedding plane, joints, faults and folding. These discontinuities are responsible to control the strength and deformation characteristic of the rock mass. Similarly, the soil mass is the disintegrated loose material which is composed of solid particles with voids in between. These voids may be filled with air or water or both. It derives mainly from rock material broken down by physical or chemical weathering. Water, wind and ice are responsible for their transportation and deposition. The performance of an engineering work will depend on the correct assessment of the engineering properties of rocks and soils (Johnson, 1991)).

The Gumara dam site foundation is composed of both soils and rocks. The valley part is covered by thick alluvial soil deposit and the banks are composed of basaltic rocks. Joints and dykes affect the rocks on abutments. In addition rocks at abutments are also affected by varied degree of weathering.

4.1 Engineering Characterization of Rocks

The rocks are involved as a foundation material and as an abutment slope forming martial at Gumara dam project. The performance for the rock mass as a foundation material or to form a stable slope entirely depends upon the properties of the rocks. In order to evaluate the suitability of the rock mass it is essential to characterize rocks based on certain engineering properties like strength, deformability and permeability. This may also help in adopting proper remedial measures to over come the possible adverse effects related to strength and deformability characteristics of the rock mass (Johnson, 1991).

4.1.1 Geological description

Topographically Gumara dam site form a part in a wide ‘U’ shaped valley. This is characterized by wide valley floor of thick alluvial deposits. This unit consists of silt and clay locally with pockets of medium grade and rounded river gravels along the river channel and banks. The top part of alluvial deposit including the present river channel deposit has a minimum thickness of 5 mm the left bank of the river channel (BH-GD3). At the foot of the left bank this alluvial deposit have a maximum thickness of about 16 m (BH – GD5) and 7 meter at the center of the dam axis (BH – GD4). The lower part of the alluvial deposit starts from about 7 meter average depth from the natural ground level to a maximum depth of 47 m (BH-GD4) consisting of alternate layers of thick organic black silty clay and thin layers of sandy gravel of river deposit (Gumara dam Geotechnical report,2006).

Both the left and right abutments are covered by feldspar rich basaltic rocks. The exposed part of this layer is affected by intense widely spaced joints and varied degree of physical weathering.

4.1.2 Discontinuities

Discontinuities or weakness planes are those structural features which separate intact rock blocks within a rock mass. The discontinuity planes are mainly bedding planes, tension joints, shear joints, fault plane, shear zone and dyke.

The important joint property or character /factors are orientation, spacing, continuity, surface characteristics, the separation of discontinuity surface and the thickness and nature of the filling material. The strength of rock mass greatly depends upon the shear strength of the discontinuity surface and also on characteristic of the discontinuities. Assessment of joint characteristics is extremely important in evaluation of dam site projects because of the following reasons;

(i) The system of joints controls the permeability of the rock mass.
(ii) The system of joints promotes weathering significantly.
(iii) Orientation of joint influence rock slope stability.
(iv) The surface characteristics of joint influence the shear strength of the rock mass.
(v) The separation and filling of joint influence the strength and permeability of the rock mass.

In the present study area the old feldspar rich basalt on both abutments are intruded by young aphanitic basalt dikes. In addition the feldspar rich basaltic rock out crop in both banks affected by joints. The discontinuities data has been collected from the dam abutments. The discontinuity data has been stereographically analysed and the preferred orientation, thus obtained, are presented in Table 4.1 and Fig. 4.1.

Table 4.1 Preferred orientation of joints as observed on the dam abutment

Location	Preferred Orientation of Joint planes (degree)							
	Joint J1		Joint J2		Joint J3		Joint J4	
	Strike	Dip Dir/Amount	Strike	Dip Dir/Amount	Strike	Dip Dir/Amount	Strike	Dip Dir/Amount
Abutment	250	160 / 78	199	109 / 68	82	352/ 53	348	258 / 70

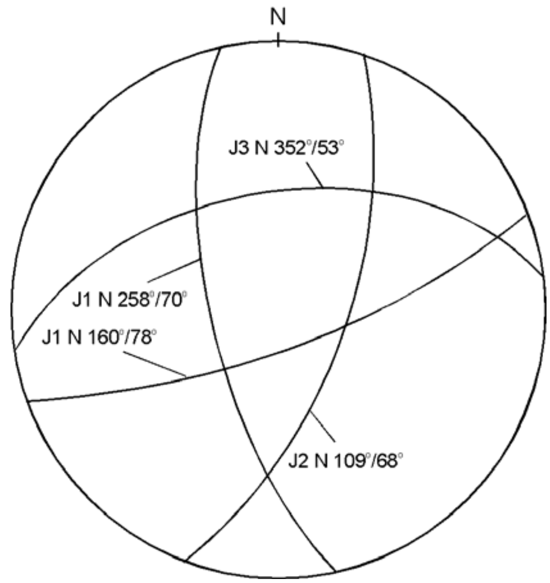

Fig. 4.1 Preferred orientation of joints as observed on the dam abutment

4.1.3 Weathering

Weathering of rocks is brought about by physical disintegration, chemical decomposition and biological activity. The type of weathering, which predominates in an area largely depend upon climate, mineral composition of rock mass, texture porosity and the incidence of discontinuities within it. To assess the weathering ability of rocks such as shales and chalk a slake durability test has been introduced. Franklin and Chandra found a general qualitative correlation between the slake durability index and the rate of weathering. Weathering classification remains descriptive or qualitative because the wide range of rock types. The most successful attempts have been based on a simple description of the geological character of the rock in the field. Some agreement on five weathering grades ranging from fresh rock to completely weathered grades are used to relate its performance (Bell F.G, 1978)

The foundation rocks in the present study area are affected by various degree of weathering. The depth and degree of weathering are important parameters to decide the stripping limit for laying the foundation. As observed on rock out crops the right abutment is highly weathered. In the boreholes, in the left abutment the degree of weathering has been encountered in each borehole log shown in Table 4.2.

Table 4.2 Weathering Variation as observed in bore holes

Depth	Degree of Weathering	Rock Type
Bore Hole BH-GD6		
0.70-2.55	Fresh	Feldsphatic
2.55-6.90	Moderately Weathered	Basalt Vesicular_basalt
6.90-8.20	Fresh	Massive feldspatic basalt
8.20-20.00	Fresh	Vesicular feldsphatic basalt
Bore hole GD_7		
2.10-3.00	Fresh	Massive feldspatic basalt
3.00-13.30	Highly Weathered	Vascular basalt
13.30-19.20	Fresh	Feldspatic basalt
1920-1960	Highly Weathered	Vesicular basalt
1960-20	Fresh	Feldspatic basalt

4.2 Rock mass classification

An engineering classification of rocks is to assess the suitability of a rock mass for a given project. The development of rock mass classification scheme was initiated about 125 years ago. Different classification system place different emphases on various parameters. There are various classification schemes developed so far. Out of these some prominent classification schemes are Terzaghi's (1964), Deere et.al (1967), Wickham et.al (1972), Bieniawsk (1976), Barton et.al (1974) classification schemes (Singh, 1998)).

For natural rock slopes, classification system proposed by Bieniawski (1976) also known as Geomechanics classification or Rock Mass Rating (RMR) is widely used. The RMR system incorporates six parameters viz; Rock quality designation (RQD), the unconfined compressive strength (UCS), the Mean fracture spacing, discontinuity conditions ground water conditions and orientation of discontinuities.

For the present study, data pertaining to RMR has been collected from 8 locations in the limited out crops. For the condition of discontinuities, spacing, orientation of the discontinuities and ground water condition visual observations were made in the field. Schmidit hammer rebound value is used to determine the strength of the rock (UCS) where as RQD was determined using Palmstorm's volumetric count method. Based on the measurement /observation at each locality each of the parameters was assigned rating as per the standard table of the RMR system. The sum total of the rating for each parameter provides the RMR value, which is presented in Table 4.3.

4.3 Strength of Rock mass

The in-situ rock, or rock mass, is comprised of intact blocks of rock separated by discontinuities such as joints, bedding planes, folds, sheared zones and faults. These rock blocks may vary from fresh and unaltered rock to badly decomposed and disintegrated rock. Under applied stress, the rock mass behavior is generally governed by the interaction of the intact rock blocks with the discontinuities.

Strength of the rock mass is the quantitative fundamental value of the rock mass. For the present study empirical approach has been applied to workout the strength of the rock mass present at the dam site. For this purpose, Hoek and Brown failure criteria has been employed. In addition to this attempt has also been made to directly estimate the strength of the rock mass by utilizing the Rock mass rating system.

4.3.1 Strength of the Rock Mass by Hoek and Brown Failure Criteria

Hoek and Brown (1980) developed an empirical approach to determine the strength of the jointed rock mass and formulated a failure criterion for jointed rock mass. Based on results of number of projects this criterion was modified by Hoek & Brown in 1988 and later by Hoek et al. (1992). The Hoek-Brown criterion for jointed rock mass is given by equation 4.1.

$$\sigma_1' = \sigma_3' + \sigma_c \left(mb \frac{\sigma_3}{\sigma_c} + S \right)^a \qquad \text{...........eq.4.1}$$

Where, \`$m_b$'is the value of the constant 'm' for the rock mass, \`s' and \`a' are constant which depend upon the characteristic of the rock mass, σ_c is the uniaxial compressive strength of the intact rock pieces and σ_1' & σ_3' are the axial and confining effective principal stresses, respectively.

The original criterion works good for most of the rocks having good to reasonable quality in which the rock mass strength is controlled by tightly interlocking angular rock pieces. To determine the material constants m_b, 'S' and 'a' used in equation 4.1, Hoek and Brown initially used RMR (Rock Mass Rating) of Bieniawski (1976). It could work for rock masses having RMR > 25. However, it failed for rock masses having RMR < 25. In order to overcome this limitation Hoek & Brown developed a new 'Geologic Strength Index' (GSI).

Table 4.3 Rock mass classification based on Geomechanics classification

Location		UCS (Mpa)	RQD	spacing (Cm)	Ground condition	Disconti-nuity condition	Rating adjustment factor	RMR	Class	Rock quality
1298326N 368351E	Value	57.54	55.6	43.2	Dry	Slightly weathered, slightly rough	Favorable			
	Rating	7	13	10	15	10	-5	50	II	Fair Rock
1298330N 368349E	Value	59.76	75.4	34.8	Dry	Slightly weathered & Rough surface	Favorable			
	Rating	7	17	10	15	20	-5	64	II	Good Rock
1298355N 368349E	Value	71.8	85.3	25.25	Dry	slightly weathered and slightly rough	Favorable			
	Rating	7	17	10	15	14	-5	58	III	Fair Rock
1298361N 368114 E	Value	66.06	91.9	16.6	Dry	Slightly weathered & Rough surface	Favorable			
	Rating	13	20	8	15	14	-5	65	II	Good Rock
1298420N 368251 E	Value	44.73	100	31.6	Dry	Slightly weathered & Rough surface	Favorable			
	Rating	4	20	10	15	16	-5	60	III	Fair Rock
1298459 N 368300 E	Value	67.46	85.3	25	Dry	Slightly weathered Rough surface	Favorable			
	Rating	7	17	10	15	14	-5	58	III	Fair Rock
1298450 N 368425 E	Value	78.19	85.3	20.25	Dry	Moderately weathered & Slightly rough	Favorable			
	Rating	7	17	10	15	10	-5	54	III	Fair Rock
1298266 N 368315 E	Value	77.37	85.3	37	Dry	Slightly weathered, slightly Rough surface	Favorable			
	Rating	7	17	10	15	12	-5	56	III	Fair Rock

For $RMR_{89} > 23$, $GSI = RMR_{89} - 5$

Where RMR_{89} is Beinwaski's 1998, Guide line for RMR classification system of rocks.

By using GSI material constants can be estimated by equations 4.2, 4.3, 4.4 and 4.5;

For GSI > 25 (undisturbed rock mass)

$$\frac{m_b}{m_i} = \exp\left(\frac{GSI - 100}{28}\right) \qquad \text{.......eq. 4.2}$$

$$S = \exp\left(\frac{GSI - 100}{9}\right) \qquad \text{.......eq. 4.3}$$

$$a = 0.5 \qquad \text{.......eq. 4.4}$$

For GSI < 25; S = 0 ; $a = 0.65 - \frac{GSI}{200}$ ....eq. 4.5

For the present study RMR data was collected from 8 localities as shown in Table 4.3. The uniaxial compressive strength has been determined by Schmidt hammer at all representative sites from where RMR data has been collected. The value of material constant 'mi' has been directly adopted from the standard table (Hoek and Brown, 1980). The RMR values are used to determine 'Geologic Strength Index' (GSI). This value is used for determining 'm_b' and 'S' constants. Tabel 4.4 and Fig. 4.2 shows the major principal stresses and shear strength parameters, cohesion and angle of friction determined by Hoek and Brown failure criteria.

Perusal of Table 4.4 indicates that the cohesion of the rock mass at the dam site as determined from Hoek and Brown failure criteria varies from 2.5 to 3.5 MPa with an average value of 3.03 MPa. Similary, angle of friction of the rock mass at the dam site varies from 33^o to 42^o with an average value of 38.75^o.

4.3.2 Direct determination of the shear strength from RMR

Bieniawski (1976) proposed a direct empirical relation to work out shear strength parameters i.e. cohesion and angel of friction by using RMR with the relations given by equations 4.6 and 4.7.

$$C = 0.05*RMR \qquad \text{.......eq.4.6}$$
$$\phi = 0.5*RMR+5 \qquad \text{.......eq. 4.7}$$

Table 4.5 presents the comparison of shear strength parameters, cohesion 'C' and angle of internal friction 'φ' as determined by Bieniawiski's RMR system and Hoek and Brown

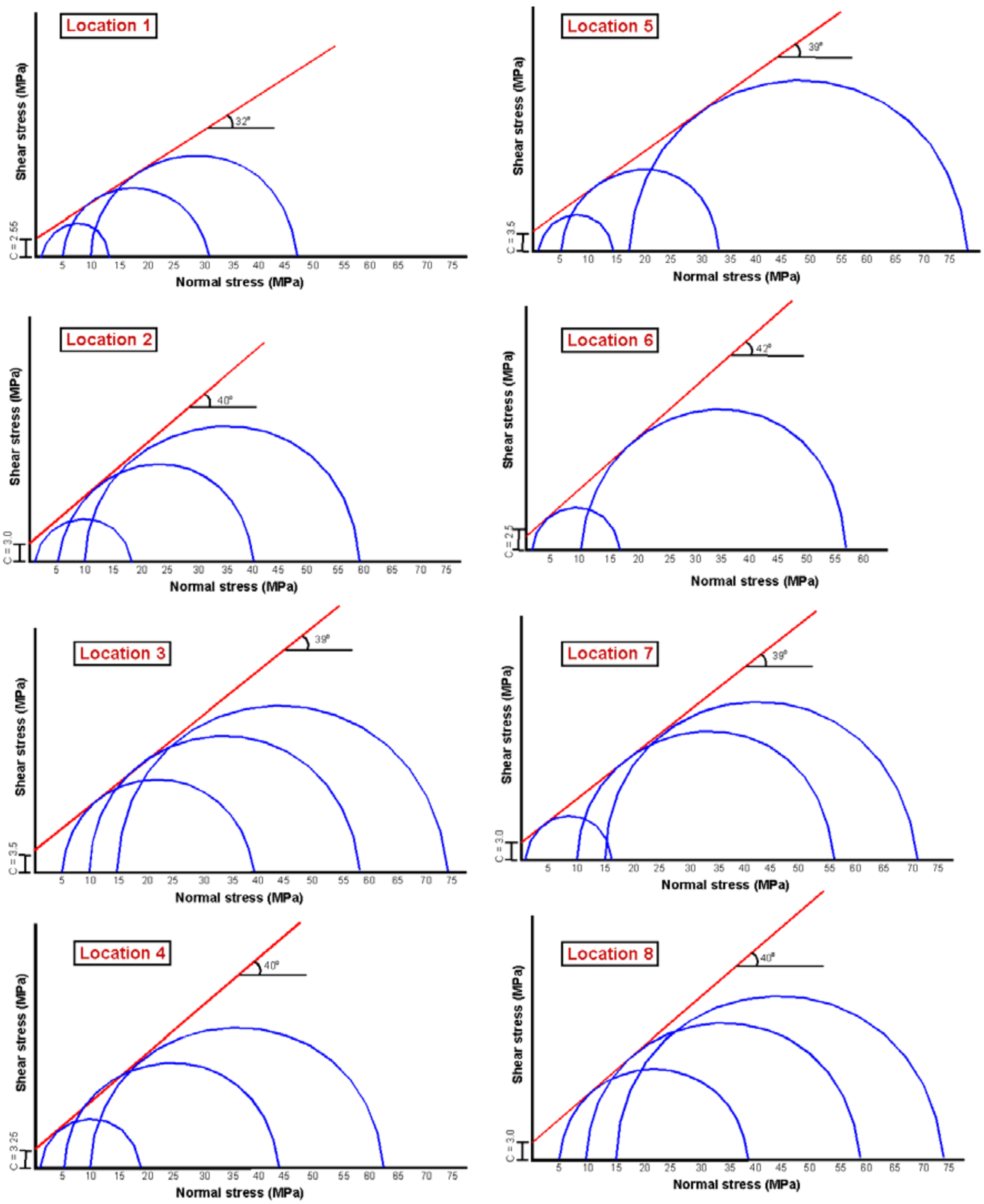

Fig. 4.2 Determination of Shear strength parameters from Hoek and Brown failure criteria for the Rock mass exposed at the Dam site

failure criteria. A perusal of Table 4.4 indicates that the cohesion 'C' of the rock mass varies in a range of 2.5 to 3.5 Mpa as determined from Hoek and Brown Failure criteria whereas, this range varies from 2.5 to 3.25 MPa as determined from Bieniawiski's relation using RMR values. Also, angle of friction 'Φ'of the rock mass varies in a range of 33 to 42^0 as determined from Hoek & Brown Failure criteria whereas, this range varies from 30 to

Table 4.4 Major principal stress and shear strength parameters as determined by Hoek and Brown Empirical approach method

Location (UTM)	RMR	UCS Mpa	mb	S	Major Principal Effective Stresses (MPa)		Cohesion (Mpa)	Angle of friction (Degree)
					Confining stress (δ3)	Axial stress (δ1)		
L1 (1298326N 368351E)	50	57.54	2.38	0.0022	1	13.01	2.55	33°
					5	31.3		
					10	47.1		
					15	60.4		
L2 (1298330N 368349E)	64	59.76	3.932	0.011	1	17.56	3.0	40°
					5	39.84		
					10	58.87		
					15	74.69		
L3 (1298355N 368349E)	58	71.8	3.175	0.0054	5	39.17	3.5	39°
					10	58		
					15	73.7		
					18	82.27		
L4 (1298361N 368114 E)	65	66.06	4.07	0.0117	1	18.88	3.25	40°
					5	42.35		
					10	62.34		
					16	81.97		
L5 (1298420N 368251 E)	60	44.73	3.41	0.0067	1	13.88	3.5	37°
					5	32.85		
					11	52.124		
L6 (1298459 N 368300 E)	58	67.46	3.1747	0.0054	1	16.45	2.5	42°
					10	56.54		
					17	77.54		
L7 (1298450 N 368425 E)	54	78.19	2.75	0.0035	1	16.37	3.0	39°
					10	56.59		
					15	71.97		
					20	85.74		
L8 (1298266 N 368315 E)	56	77.37	2.954	0.0043	5	39.18	3.0	40°
					10	58.07		
					15	73.77		
					20	87.79		

37.5^0 as determined from Bieniawiski's relation using RMR values. The average value of Cohesion 'C' for the rock mass of the study area is 3.03 Mpa and the average angle of friction for the rock mass is 38.75^0 as determined from Hoek and Brown Failure criteria whereas the average value of Cohesion 'C' for the rock mass of the study area is 2.9 Mpa and the average angle of friction for the rock mass is 34.44^0 as determined from Bieniawiski's relation. The overall average cohesion (C) is 3.1 MPa and angle of friction for the rock mass is 36.6^0.

4.3.3 Rock quality designation (RQD)

The rock quality designation (RQD) was devised by Deere (Deere et al. 1967) to provide qualitative estimate of rock mass quality from drill core logs. RQD is a important parameter to describe the engineering properties of rock mass. RQD is defined as the percentage of intact core pieces longer than 10 cm in the total run.

$$RQD = \frac{\sum \text{length of core pieces} > 10 \text{ cm long} * 100}{\text{Total length of core run}}$$

The smaller pieces < 10 cm length in the core (core loss), are assumed to result from closely spaced discontinuities, shearing, faulting or weathering, all of which decrease rock quality. RQD values of 100 % does not imply an unjointed rock mass but pieces greater than 10 cm. (Bell, 1992).

The project Authority has been collected the RQD for the boreholes in different depths. Thus, table 4.5 shows the RQD values and description of the RQD value in the bore holes at different depths.

4.4 The Deformability of the Rock mass

Deformability means the capacity of rock to strain under applied loads or in response to un-loads on excavation. If a dam is founded over various types of rocks having varying deformation properties, shear and diagonal tension stresses will develop due to the uneven deflection of the foundation.

If such properties of the foundation rocks are known in advance, dam structure can be designed accordingly, to handle such deflections. The deformability properties of the rocks are critical when they form the foundation for concrete dam, tower buildings, big bridges or any major engineering structure. Stress strain relationships can be observed in static and dynamic tests conducted in the laboratory or in the field. Deformability properties can then be obtained from the data, assuming that some idealized model describes the rock behavior in the test configuration. Deformability properties can also be back calculated from instrumental data on the movement of a structure or excavation, if the initial and final states of stress are known. The most widely used testing procedures for deformability measurements are laboratory compression and bending tests, wave velocity measurements in the lab or field, field loading tests using flat jacks or plate bearing apparatus and borehole expansion test (Goodman, 1989).

Table 4.5 RQD values of Rocks in foundation area and quality

Borehole name	Location	RDQ collection Depth(m)	RQD values	Description of RQD Value (quality)
BH-GD3	368800E 1298137N	37-38.35	100	Excellent
		38.35-39.85	30	Poor
		39.85-42	90	Excellent
BH-GD4	368606E 1298195N	47-49.50	10	Very Poor
		49.50-53.00	60	Fair
BH-GD5	368446E 1298244N	18.35-19.20	35	Poor
		33.90-34.00	40	Poor
		34.00-34.80	25	Poor
		34.80-36.30	50	Fair
		36.50-37.00	80	Excellent
		37.00-38.30	68	Fair
		38.30-40.00	93	Excellent
BH-GD6	368250E 1298273N	2.55-3.00	66	Fair
		3.00-4.00	48	Poor
		4.00-4.80	45	Poor
		5.40-5.80	50	Fair
		5.80-6.90	20	Very Poor
		6.90-8.20	100	Excellent
		8.20-11.00	95	Excellent
		11.00-12.00	67	Fair
		12.00-13.10	65	Fair
		13.10-14.20	100	Excellent
		14.20-15.00	40	Poor
		15.00-16.00	65	Fair
		16.00-17.50	78	Good
		17.50-18.60	60	Fair
		18.60-20	20	Very Poor
BH-GD7	368278E 1298298N	1.80-2.10	12	Very Poor
		2.10-3.00	100	Excellent
		3.00-3.50	40	Poor
		6.00-6.50	57	Fair
		6.50-9.00	50	Fair
		9.00-12.20	30	Poor
		12.20-13.10	68	Fair
		13.30-15.70	55	Fair
		15.70-17.10	85	Good
		17.10-20.00	60	Fair

Table 4.6 Comparison of shear strength parameters, cohesion 'C' and angle of internal friction 'φ' as determined by Bieniawiski's RMR system and Hoek and Brown Failure criteria

Location	Average RMR Value	Shear Strength Parameters from RMR (Range)				Shear Strength Parameters from Hoek and Brown Failure Criteria	
		Cohesion (C)	Specific Cohesion (C)*	Angle of Friction (φ)	Specific Angle of Friction (φ)**	Cohesion (C) (Mpa)	Angle of Friction (φ)
L1	50	200-300	2.5	25-35	30	2.55	33°
L2	64	300-400	3.2	35-45	37	3.0	40°
L3	58	200-300	2.9	25-35	34	3.5	39°
L4	65	300-400	3.25	35-45	37.5	3.25	40°
L5	60	200-300	3.0	25-35	35	3.5	37°
L6	58	200-300	2.9	25-35	34	2.5	42°
L7	54	200-300	2.7	25-35	32	3.0	39°
L8	56	200-300	2.8	25-35	33	3.0	40°
Average Values	58.125	225-325	2.9	27.5-37.5	34.44	3.03	38.75˚

C* is the specific value of Cohesion for a given RMR as determined using Eq. 4.6
Φ** is the specific value of angle of friction for a given RMR as determined using Eq.4.7

For the present dam site no field test data is available therefore an attempt has been made to empirically estimate the deformability characteristics of the rock mass exposed at the dam site. For this purpose empirical relation proposed by Hoek and Brown (1997) has been employed. Hoek and Brown (1997) proposed an empirical relation to work out modulus of deformation of rock mass .This relation works good for rock mass having uniaxial compressive strength (UCS) less 100 MPa. At the present dam site the UCS value is less than 100, thus condition for application of this empirical relation is satisfied. The Hoek and Brown empirical relation for determination of modulus of deformation is given as eq. 4.8.

$$Ed = (qc)^{0.5} / 10 * 10^{(GSI-10)/40} \qquad \text{……eq. 4.8}$$

Where; 'qc' is the UCS of intact Rock at natural moisture content, GSI 'Geologic Strength Index' (GSI). For $RMR_{76} > 18$, $GSI = RMR_{76}$ and $RMR_{89} > 23$, $GSI = RMR_{89} - 5$

Thus, the computed modulus of deformation of the rock mass at dam site is presented in Table 4.7.

Table 4.7 The modulus of deformation for various localities at the dam site

Location (UTM)	**Rock Mass Rating RMR Value**	**Uniaxial Compressive Strength 'qu ' (UCS) (MPa)**	**Modulus of deformation 'Ed' (GPa)**
L1 (1298326N, 368351E)	50	57.54	0.0101
L2 (1298330N, 368349E)	64	59.76	0.046
L3 (1298355N, 368349E)	58	71.8	0.071
L4 (1298361N, 368114 E)	65	66.06	0.0457
L5 (1298420N, 368251 E)	60	44.73	0.050
L6 (1298459 N, 368300 E)	58	67.46	0.069
L7 (1298450 N, 368425 E)	54	78.19	0.0936
L8 (1298266 N, 368315 E)	56	77.37	0.083
Average Value	58.125	65.36	0.0585

Perusal of Table 4.7 indicates that the value of modulus of deformation of the rock mass at proposed dam site varies from 0.0101 to 0.0936 GPa with an average value of 0.0585 GPa.

4.5 Permeability of foundation rocks

The permeability of rocks is due to water percolation along bedding planes and other surfaces of separation such as joints. In order to obtain test results as reliable as possible, drill holes and water pressure tests must be distributed so as to determine the permeability of all rocks type that occur in the bottom and flanks of the dam site.

In dam foundations the role of water is destructive and in all respect it must be checked not to enter in the dam foundation area. The adverse effect of water in the dam foundation are; (i) it lubricates the rock surface and decreases the dry coefficient of friction, (ii) within the rock surface it erodes the rock by mechanical action and form caverns and (iii) within the rock strata it develops the pore water pressure which is proportional to the hydraulic head present in the reservoir. This pore water pressure tends to lift dam thus decreasing shearing strength of the rock.

The permeability of rocks is assessed on the basis of criteria established by various authors like Lugeon, Jahde, Terzagi, Holsby etc. Lugeon's criterion is most commonly used as it gives reliable values. According to Lugeon, in dams higher than 30 m the water loss in the pressure tests should not exceed 1 liter in 1 min. per m of the borehole at 10 atmospheric Pressure, which should act at least for10 minutes. The permeability results are normally described in terms of Lugeon units, one lugeon is equal to a flow of 1 lit/m/min at a pressure of $1MN/m^2$. A lugeon unit is approximately equal to coefficient of permeability of 10^{-7} m/s. According to lugeon (1933) a rock absorbing less than one lugeon unit can be considered as watertight. Lugeon (1933) also suggested that grouting beneath concrete gravity dams is necessary when the permeability exceeded 1 lugeon unit (Bharat Singh, 1995).

Holsby, 1977 proposed an approach of assessment of the permeability of the foundation rocks, according to this approach, five consecutive tests are performed, for 10 min, each at pressures in order A, B, C, B, A. Based on the Lugeon values, thus obtained, the flow conditions in the foundation rocks are defined as shown in Fig. 4.3.

In order to assess the permeability of rocks in the foundation area of Gumara dam site water pressure tests has been conducted by the Project Authorities. For this purpose water pressure tests were conducted in five-bore holes using Holsby approach and Lugeon criteria. These boreholes were selected in the valley floor and on the left abutment. The summarized results of these water pressure tests are presented in Table 4.8.

A perusal of Table 4.8 indicates that the bedrock in the valley floor is permeable upto a depth of 42m where it demonstrates turbulent flow. Under turbulent flow condition the permeability decreases, as the pressure increases from minimum to maximum and it increases when pressure is decreased from maximum to minimum. It implies that as the pressure is increased in the initial steps of the water pressure test the infilled material within the joints/ fissures starts blocking the openings by wedging. Consequently when pressure is decreased from maximum to minimum the infilled particles within the joints/ fissures are washed away. Thus, there is a need to provide proper treatment of the foundation rocks. The possible treatments of foundation rocks to check any possible seepage after the construction of dam is discussed in detail in Chapter 5. Further, in the valley floor rocks at a depth below 51m are non-permeable indicating that the joints are tightly closed.

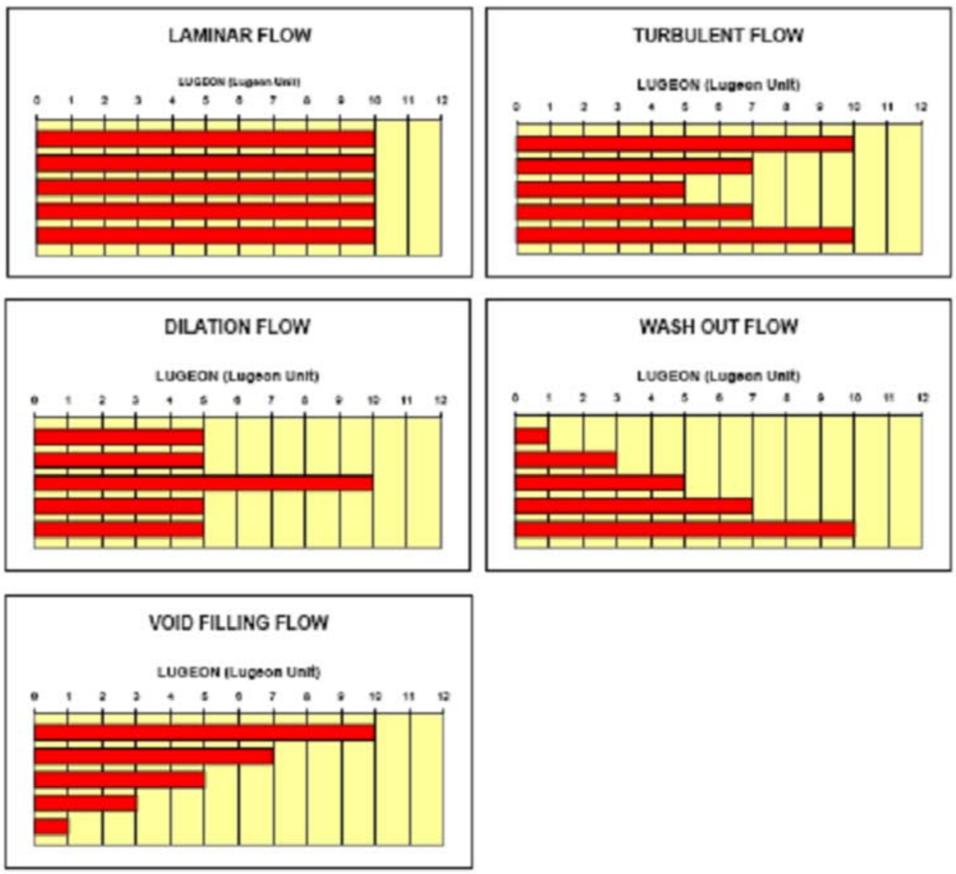

Fig. 4.3 Lugeon test pattern and the associated flow conditions

In the lower reaches of left abutment the rocks are permeable upto a depth of about 35m where these rocks demonstrate turbulent to washout flow conditions with a permeability value of 30 Lugeon. In the intermediate section of the left abutment the rocks are moderately permeable and showed laminar to turbulent flow conditions. In this zone the permeability ranges from 2 to 13 Lugeon upto a depth of 20m. In the top reaches of the left abutment the rocks are highly permeable upto a depth of 13m where these rocks demonstrates dilation to turbulent flow conditions. This may be a result of high weathering grade of rocks due to which the joints are wide open and extends upto a depth of 13m. Further below in this section of the left abutment the permeability condition improves and it reduces to 4 Lugeon. It implies that the joints are comparatively tight below 17m depth. However, the overall permeability condition in the left abutment indicates a need for proper treatment, which is discussed in detail later in Chapter 5.

Table 4.8 Results of water pressure tests conducted in valley floor and left abutment

Bore hole Number	Location	Elevation (m)	Test Section Depth (m)	Type of flow	Permeability (Lugeon)	General Remark for permeability
BH-GD3	Valley floor	1925	37-42	Turbulent	13	Medium Permeability Needs Treatment
BH-GD4	Valley floor	1927	51-54	No Flow	No flow	Non Permeable Rocks
BH-GD5	Left abutment	1933	19-26	Turbulent	12	Medium Permeability Needs Treatment
			29-35	Wash out	30	
			35-40	No Flow	No flow	Non Permeable Rocks
BH-GD6	Left abutment	1950	6-11	Turbulent	13	Medium Permeability Needs Treatment
			11-16	Laminar	10	Low Permeability Needs Treatment
			16-20	Laminar	2	Impervious Needs No Treatment
BH-GD7	Left abutment	1970	3.5-9.0	Dilation	41	High Permeability Needs Treatment
			9-13	Turbulent	62	Very High Permeability Needs Treatment
			13-17	Laminar	4	Low Permeability Needs Treatment
			17-20	Laminar	12	Low Permeability Needs Treatment

** 0–3 Lugeon impervious, 3–10 Lugeon low permeability, 10–30 Lugeon medium permeability, 30–60 Lugeon high permeability, and > 60 Lugeon very high permeability

Further, so far no water pressure tests has been conducted on the right abutment. However, during the present fieldwork attempt has been made to qualitatively assess the permeability condition of the right abutment. The geological condition on right abutment is similar to that of left abutment. The rocks on the surface are highly weathered and joints are wide open. Thus, it may safely be presumed that the permeability conditions will also be similar to that of the left abutment. In this regard the exact permeability condition will only be assessed quantitatively when more water pressure tests will be conducted on the right abutment.

4.6 Slope stability of abutments

Design of stable cut slope is the prerequisite for the safe construction and functioning of dam project. It is essential to recognize the potential slope stability problems in the initial stages of investigation, so that proper remedial measures can be adopted for the slope stabilization. For the present study slope stability studies has been carried out for the abutment slopes.

4.6.1 Discontinuity analysis

Discontinuities or weakness planes are those structural features, which separate intact rock blocks with in a rock mass. The discontinuity planes are mainly bedding planes, tension joints, shear joints, fault plane, shear zone and dyke. These discontinuity planes play an important role in defining the stability condition of the slope face. Thus it is important to collect the discontinuity orientation data and to workout there preferred orientation.

During the present study the stability analysis has been carried out for left abutment slope of the proposed Gumara dam site. Due to the non-accessibility and steep slope gradient discontinuity data could not be collected from the right abutment slope.

At the Gumara dam site young aphanitic basalt dikes intrude the feldspar rich basalt on both the abutments. In addition joints also affect the feldspar rich basaltic rock out crop on both the abutments. During the present field work discontinuity data were collected from the left abutment slope and later stereographically analyzed to get the preferred orientation. The preferred orientation of discontinuity planes as observed on left abutment is already discussed in the previous paragraphs (Fig.4.1, Table 4.1).

4.6.2 Geometry and Geology of the critical slope section

For the detailed stability analysis cross sections has been prepared along the left abutment slope sections. The geology and the geometry of slope sections in terms of slope direction and inclination, upper slope direction, inclination and the height of the slope are presented in Fig.4.4.

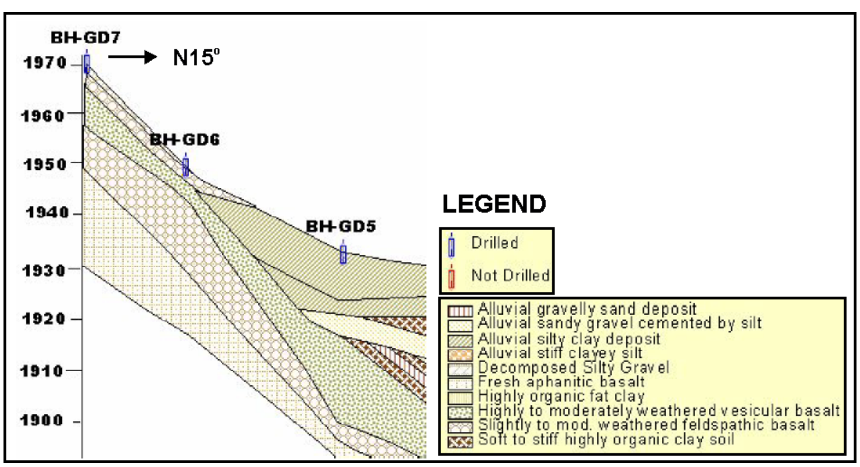

Fig. 4.4 Slope Geometry and Geological section of Left Abutment

4.6.3 Kinematics check

In order to work out the possible mode of failure Markland test has been applied. Constructing the daylight envelope or "phi circle" is useful method of identifying kinematically possible failure modes. This envelope is the locus of all the poles representing planes whose dip direction lie in the plane of the slope face. Poles lying with in the envelope represents joints which daylight in the slope (Bell, 1978).

Thus, in rock slope the failure will occur if the following conditions are satisfied;

Plane failure $\alpha_f > \alpha_p > \Phi$ ….. eq. 4.9

Wedge failure $\alpha_f > \alpha_i > \Phi$ ….. eq. 4.10

Where; 'α_f' is the slope angel, 'α_p' is the dip of the potential failure plane, 'α_i' is plunge of the line of intersection and 'φ' is the angel of internal friction of two wedges forming planes.

The structural data, along with slope inclination and a 'phi circle' corresponding to angel of friction of the rock mass has been plotted on equal area projection 'Schmidt Net'. The angel of friction has been estimated from RMR data (37^o). Fig. 4.5 shows the kinametic plot for the left abutment slope.

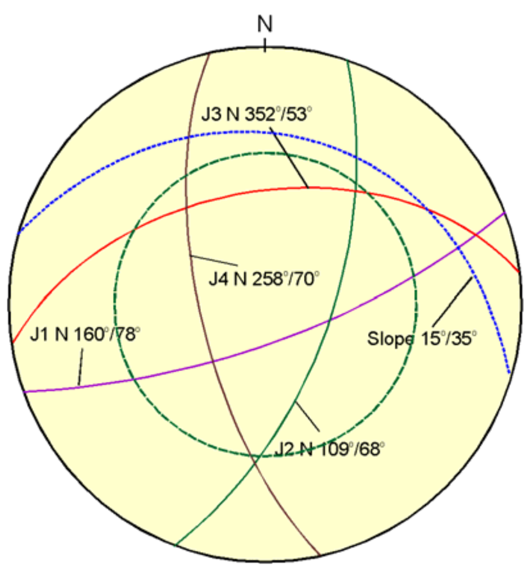

Fig. 4.5 Stereoplot showing Kinametic check for left abutment slope

Perusal of Fig. 4.5 indicates that the left abutment slope is not kinametically unstable as it do not satisfy the condition set by Markland. Thus it may safely be presumed that the left abutment slope is stable for present geometric configuration. However, Plane J3 (N352^{o}/53^{o}) may be daylighted during the stripping operations if the slope is cut at a steeper angles greater than 53^{o}. Therefore it is suggested that the left abutment slope should not be cut at steeper angles during stripping operations.

4.7 Characterization of foundation soils

Soils are used as construction materials and foundations for engineering structures. The wide range of soil properties and conditions affect their performance and use. Various engineering classifications of soils have been adopted throughout the world. The most commonly used with wide application is the Unified Soil Classification. For the present study, unified soil classification system is used to classify the soils in the foundation area. An engineering soil classification system indicates engineering soil properties and provides preliminary understanding of the behavior of the soil under various engineering conditions. It is used to communicate this information in simple notation and brief description.

For the present study two test pits were excavated along the dam axis and three soil samples were collected for the determination of index property. The location details of these test pits is shown in Table 4.9.

Table 4.9 Details of test pits and soil sample collection from the dam foundation area

Test Pit No.	Sampling Details				
	Location	**UTM**	**Surface Elevation (m)**	**Depth (m)**	**Type of Sample**
TP-1	River bed section at Dam Axis	368457 E 1298273 N	1923	1.0 - 1.30	Disturbed
TP-1	River bed section at Dam Axis	368457 E 1298273 N	1923	1.30 - 2.50	Disturbed
TP-2	River bed section at Dam Axis	368618 E 1298250 N	1933	1.00 - 1.52	Disturbed

4.7.1 Physical properties of the soils of the dam Foundation Area

Index properties

The tests required for the determination of engineering properties are generally elaborate and time consuming. Sometimes index properties are used to have some rough estimation of engineering properties without conducting elaborate tests. Soils are classified and identified based on the index tests. The main index properties of coarse-grained soils are particle size and relative density whereas, for fine-grained soils the main index properties are Atterberg limits and consistency (Arora, 1997).

Grain size analysis

It is determined by the gradation analysis of soil and presented in the form of a cumulative grain size curve in which particle sizes are plotted on a linear scale. The sizes as 10 %, 30% and 60% (D_{10}, D_{30}, D_{60}) are used in defining the gradation characteristics of soil. The permeability and the angel of friction of soil are mostly proportional to the grain size of the soil.

For the present study, Grain size analysis has been carried out for the representative samples collected from the river bed section along the dam axis. The gradation curves for various samples is shown in Fig. 4.6.

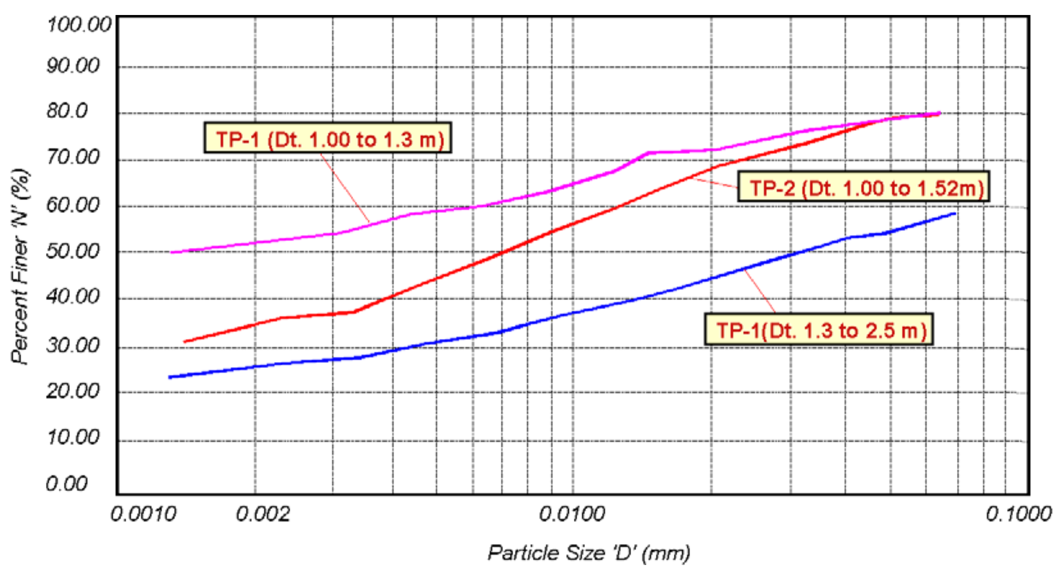

Fig.4.6 Gradation of the soils present in the dam foundation area

Consistency of the soils

The consistency of a soil in the remoulded soil varies in proportion to the water content. At higher water contents, the soil-water mixture behaves a liquid, at lower water contents it behaves plastically and at still less water contents it behaves as a semi solid and solid. An important component of soil classification is to determine the water contents at which these phase/behavior changes occur.

Determination of these limits helps in predicting the potential engineering behavior and use of the soils. Therefore, these limits are important index properties of soil.

In order to determine the Atterburg limits of fine-grained soils of the study area, disturbed samples has been collected. The samples were collected from the dam site and later tested at Amhara Water Resource Development Soil Laboratory, Bahir dar. The results of Atterburg limits of the soil samples from the dam site area are presented in Table 4.10.

Table 4.10 Atterburg Limits and Classification of soils of the dam foundation area

Test Pit	Location (UTM)	Depth (m)	Liquid limit	Plastic limit	Plasticity index	Soil Classification (USCS)
TP-1	368457 E 1298273 N	1.0-1.30	81	31.38	49.62	Inorganic clays of high plasticity **CH**
TP-1	368457 E 1298273 N	1.30-2.50	56.23	32.42	23.81	Inorganic silt of high plasticity **MH**
TP-2	368618 E 1298250 N	1.00-1.52	48.73	27.79	20.94	Inorganic silt of low plasticity **ML**

Characterization of foundation soils from secondary data

The soil samples collected during the present study were collect from test pits at shallow depth. These samples can only characterize surfacial soils. However, for the dam foundation analysis it is essential to characterize soils upto much depth. For this purpose secondary data from the previous test reports has been utilized. The foundation soils in the river valley section are very thick which extends up to a depth of 47 meters as observed in boreholes BH-GD4 and BH-GD3. From the core samples of the bore holes the Atterberg limits, free swell, and the percent organic content of the soil has been determined by the Project engineers. The test results on Atterberg limit, free swell and percentage of organic contents of the soils are present in Table 4.11. The plasticity chart prepared from the secondary data is presented as Fig. 4.7.

Perusal of Table 4.11 indicates that the liquid limit for the foundation soil varies from 45.05 to 60.5 with plastic limit varying from 28.45 to 41.22. The Plasticity index varies in the range of 3.83 to 28.40. Thus, according to Holtz and Gibbs Classification (1956) (Table 4.12) from the results it may be concluded that the soil possess medium to high potential for swelling as far as its liquid limit is concerned. Whereas, on the basis of plasticity index the soil falls in low to high potential for swelling. The samples also show organic content in a range of 14.26 to 23. The free swell of the foundation soils varies in the range of 30 to 60 %.

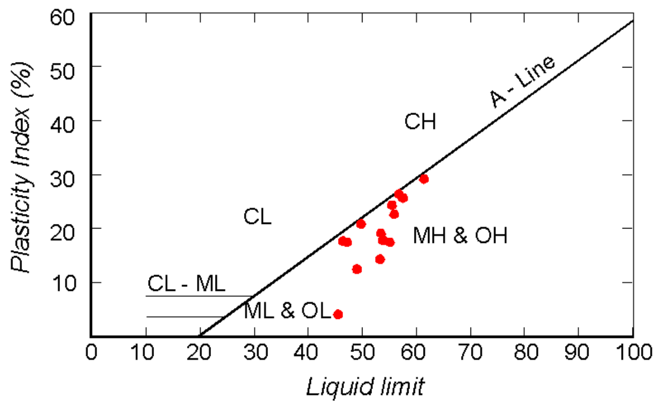

Fig. 4.7 Plasticity chart for soil samples from secondary data

According to Arora (1997) Soils with free swell less than 50% are not likely to show expansive property, while soils with free swells in excess of 50% could present swell problems. Values of 100% or more are associated with clay, which could swell considerably, especially under light loadings. Thus most of the soil samples from the foundation soils show potential to swell.

Table 4.11 Atterberg Limits, free swell, percent of organic content of soils of the dam foundation

Bore Hole No	Location (UTM)		Depth in (m)	Attebeurg limit			Free swell (%)	Organic content
	Northing	Easting		Liquid limit	Plastic limit	Plasticity index		
BH-GD3	1298137	368800	6.0	60.50	32.10	28.40	50	16.25
BH-GD3	1298137	368800	6.2	45.50	28.45	17.05	60	17.85
BH-GD3	1298137	368800	8.0	49.35	38.28	11.07	30	16.62
BH-GD3	1298137	368800	10	49.7	29.33	20.37	40	16.89
BH-GD3	1298137	368800	12	51.9	32.49	19.41	30	16.79
BH-GD3	1298137	368800	14	47.05	29.59	17.46	60	18.00
BH-GD3	1298137	368800	16	54.00	36.82	17.18	50	19.25
BH-GD3	1298137	368800	18	55.10	33.08	22.02	50	17.88
BH-GD4	1298195	368606	3.5	52.80	34.63	18.17	50	19.25
BH-GD4	1298195	368606	8.8	57.60	32.35	25.25	50	23.00
BH-GD4	1298195	368606	13	54.35	29.85	24.50	50	16.72
BH-GD4	1298195	368606	18	57.25	30.72	26.53	50	19.24
BH-GD4	1298195	368606	25	45.05	41.22	3.83	50	14.26
BH-GD4	1298195	368606	30	52.00	37.40	14.60	30	17.24

Table 4.12 Classification of potential swell based on plasticity (Holtz and Gibbs, 1956)

Classification of potential swell	Liquid limit (LL),%	Plasticity Index (PI),%	Shrinkage Limit (SL), %
Low	20-35	<18	>15
Medium	35-50	15-28	10-15
High	50-70	25-41	7-12
Very high	>70	>35	<11

Also samples collected and tested during the present study from the shallow depth demonstrates liquid limit in the range of 48.73 to 81 with a plasticity index ranging from 20.94 to 49.62. Thus, indicating medium to very high potential for swelling.

From the results it may safely be deduced that the foundation soils posses a potential to swell in varied degree upto a depth of 47 m, which by all means is an unfavorable condition for the embankment dam. Further, the presence of organic matter will also increase the compressibility of the soils. Such compressibility is also undesirable property for the dam foundation as this may lead to development of cracks in the core. Thus, some form of remedial measures is required to improve the properties of the foundation soils, which are discussed in detail later in Chapter 5.

4.7.2 Engineering Classification of the soils

Soil classification is the arrangement of soils into different groups such that the soils in a particular group have similar behaviour. It is a sort of labelling of soils with different labels. As there is a wide variety of soils covering earth, it is desirable to systematize or classify the soils into broad groups of similar behaviour. It is more convenient to study the behaviour of groups than that of individual soils.

For the present study the soils of the dam site area has been classified by United Soil Classification system (USCS). The USC system is a textural-plasticity classification scheme. Paired letter symbols are used for each soil group in this system. The first symbol refers the predominance particle size. The second symbol for coarse-grained soils refers to gradation for clean (little or no fines) soils and the presence of silt and clay size particles for soils with appreciable amounts of fines. The second symbol for fine-grained soils sub-divided on the basis of low (L) or high (H) plasticity.

Laboratory determination of liquid limit and plasticity indexes for a soil sample permits assignment of fine-grained soils (including the fine fraction of coarse grained soils) to the proper group by use of 'plasticity chart', or 'A - Line diagram' (Casagrande, 1948).According to USCS, the soils present at the dam foundation area are classified as 'Inorganic clays of high plasti-city' (CH), 'Inorganic silts of low plasticity (ML) and Inorganic silt of high plasticity (MH). Table 4.12 shows the general engineering properties in qualitative terms. From Table 4.13 it can be noted that both CH, ML, MH soils possess medium to high compressibility and fair to poor shear strength.

Engineering Properties such as compressibility and shear strength are important as far as dam foundation is concerned. High compressibility is associated with differential settlements and formation of cracks in the core. Whereas, poor shear strength of the foundation soils will result into poor bearing capacity of the soils.

Table 4.13 General engineering properties of 'CH', 'ML' and 'MH' Group soils

Soil Group	Permeability	Compressibility	Shear strength	Workability
ML	Semi-pervious to Impervious	Medium to High	Fair	Good to Fair
MH	Semi-pervious to Impervious	High	Fair to poor	poor
CH	Impervious	High	Poor	Poor

(Arora, K.R., 1997)

However, based on these limited samples, from shallow depth of upto 2.5 m, overall assessment on the engineering performance of these soils as foundation material can not be made. Therefore, an attempt has been made to utilize the secondary data to classify the soils of the foundation area. No data on particle size distribution was available for the present study. However, data on Atterberg limits has been utilised to plot plasticity chart (Fig.4.7). As from the chart (Fig.4.7) it can be clearly seen that all the soil samples falls below 'A-line'. Also, the samples show organic content in a range of 14.26 to 23. Thus, it may safely be concluded that the soils falls in organic silts of low plasticity 'OL' and organic clays of medium to high plasticity 'OH' groups (Table 4.14).

From the results (Table 4.14) it can be seen that most of the foundation soils at depth fall into organic clays of medium to high plasticity 'OH' group. According to Arora (1997) the soils falling in OH group possess high compressibility and poor shear strength. Thus, these soils may not be suitable for the foundation as these may result into high compressibility and may be responsible for the development of cracks in the core.

Activity of Soils

Activity (A) of the soil is the ratio of the plasticity index and the percentage of the clay fraction (soil fraction passing 2μ sieve).

$A = I_p / F$ ……eq. 4.11

Where, I_p is plasticity index and F is clay fraction.

The activity of soils of the dam foundation area has been worked out and is presented in Table 4.15 The result from Table 4.16 indicates that the soil samples collected during this study falls into normal to inactive class of activity.

Table 4.14 Classification of soils of the foundation area

Bore Hole No	Location (UTM)		Depth in (m)	Attebeurg limit			Free swell (%)	Organic content	Classification of soils based on Plasticity*
	Northing	Easting		Liquid limit	Plastic limit	Plasticity index			
BH-GD3	1298137	368800	6.0	60.50	32.10	28.40	50	16.25	OH
BH-GD3	1298137	368800	6.2	45.50	28.45	17.05	60	17.85	OL
BH-GD3	1298137	368800	8.0	49.35	38.28	11.07	30	16.62	OL
BH-GD3	1298137	368800	10	49.7	29.33	20.37	40	16.89	OL
BH-GD3	1298137	368800	12	51.9	32.49	19.41	30	16.79	OH
BH-GD3	1298137	368800	14	47.05	29.59	17.46	60	18.00	OL
BH-GD3	1298137	368800	16	54.00	36.82	17.18	50	19.25	OH
BH-GD3	1298137	368800	18	55.10	33.08	22.02	50	17.88	OH
BH-GD4	1298195	368606	3.5	52.80	34.63	18.17	50	19.25	OH
BH-GD4	1298195	368606	8.8	57.60	32.35	25.25	50	23.00	OH
BH-GD4	1298195	368606	13	54.35	29.85	24.50	50	16.72	OH
BH-GD4	1298195	368606	18	57.25	30.72	26.53	50	19.24	OH
BH-GD4	1298195	368606	25	45.05	41.22	3.83	50	14.26	OL
BH-GD4	1298195	368606	30	52.00	37.40	14.60	30	17.24	OH
* 'OL' organic silts of low plasticity				'OH' organic clays of medium to high plasticity					

Table 4.15 Classification of soils based on activity

S.No	Activity	Soil type
1.	A < 0.75	Inactive
2.	A = 0.75 to 1.25	Normal
3.	A > 1.25	Active

Table 4.16 Activity of the soils of the dam foundation area

Soil Sample	Liquid Limit	Plastic Limit	Plasticity Index	Clay Fraction	Activity
TP-1	56.23	32.42	23.81	25	0.952
TP-1	81	31.38	49.62	53	0.936
TP-2	48.73	27.79	20.94	35	0.598

4.8 Engineering property of foundation soils

The suitability of soil for a particular use depends on it's response to that use. Suitability usually depends on one or more engineering properties of soil. The performance of engineering work will depend on the correct assessment of engineering properties to determine suitability and to predict performance of a soil for intended use. The three important engineering properties are the degree to which soil will change volume under applied load this is termed as compressibility, the resistance of soil to sliding of one mass against another termed as shear strength and the permeability of the soil (Arora, 1997).

4.8.1 Compressibility

Compressibility is the decrease in volume of a soil mass due to a change in the volume of voids and to a lesser extent; it can also result from a change in the volume of solids. Consolidation is the form of compressibility that occurs under a static load such as a weight of structure after construction. Consolidation is the process of driving water from the voids. The consolidation of the soil should be considered during design otherwise it might lead to settlement that may seriously damage the structure being founded on soil. The consolidation of soil is the process of compression of soil by gradual reduction of pores under a steady applied pressure. The main purpose of the consolidation test is to obtain the soil data required for the predicting the rate and amount of settlement of the structure. Two basic coefficients are very important which relates to the consolidation, these are ' Compression index 'Cc' and ' Coefficient of volume change 'mv'.

<u>Coefficient of volume Change(m_v) and Compression Index (c_c)</u>

Coefficient of volume compressibility (m_v) is important compressibility parameter of soil required for settlement analysis. Coefficient of volume compressibility of soils of the study area is calculated by using following relation;

$$m_v = \left(\frac{H_1 - H_2}{H_1}\right) x \left(\frac{1}{P_2 - P_1}\right) \qquad \text{.......eq.4.12}$$

where, m_v is the coefficient of volume change (m^2/kPa), P_1 and P_2 are pressure increments (kPa), H_1 and H_2 are specimen height at equilibrium pressure (cm).
Compression Index (c_c) is also an important parameter used in settlement analysis. For the present study compression Index is calculated by using the following relation;

$$c_c = \frac{e_i - e_f}{\log_{10}\left(\frac{\sigma_f}{\sigma_i}\right)} \qquad \text{.........eq. 4.13}$$

where, e_i is the void ratio at initial pressure, e_f is the void ratio at final pressure, σ_i is the initial pressure(kPa) and σ_f is the final pressure(kPa).

The value of Cc is determined between 2 points where curve gets straightened in pressure versus void ratio curve. Fig. 4.9 shows the e-log p curves for four represented soil samples from the study area. The calculated Coefficient of volume compressibility (m_v) and

Compression Index (c_c) as determined from one dimensional consolidation test are presented in Table 4.17

The Project Geo-technical Engineers has conducted a consolidation test on soils from two boreholes at different depths to know the consolidation characteristics of the foundation soils. Table 4.17 shows the results of the consolidation tests. Fig. 4.9 shows the void ratio verses Pressure curves for all the tested samples.

From the results (Table 4.17) it can be noted that compression index (Cc) varies in the range of 0.185 to 0.249 for the pressure variation of 640 to 1280 KN/m^2, whereas coefficient of volume change varies from 5.33 x 10^{-5} to 8.33 x 10^{-5} for the same pressure range.

4.8.2 Shear strength of soils

The shear strength of a soil is its maximum resistance to shear stresses just before the failure. Soils are seldom subjected to direct shear. However, stresses develop when the soil is subjected to direct compression. Although, shear stresses may also occur when the soil is subjected to direct compression.

Shear stresses may also develop when the soil is subjected to direct tension, but these shear stresses are not relevant. In field, soils are seldom subjected to tension, as it causes opening of cracks and fissures. Thus, the shear failure of a soil occurs when the shear stresses induced due to the applied compressive loads exceed the shear strength of the soil.

Table 4.17 Consolidation of Foundation Soil

S.No	Borehole No	Depth in (m)	Void Ratio		Pressure Applied (KN/m^2)		Compression Index 'C$_C$'	Coefficient of volume change 'mv' (m^2/kN)
			Initial void Ratio	Final Void Ratio	Initial Pressure	Final Pressure		
1	BH-GD3	6.0	0.828	0.780	160	320		1.65 x 10^{-4}
			0.727	0.659	640	1280	0.228	6.21 x 10^{-5}
2	BH-GD3	16.0	0.678	0.634	160	320		1.64 x 10^{-4}
			0.580	0.520	640	1280	0.199	5.94 x 10^{-5}
3	BH-GD4	3.5	0.760	0.696	160	320		2.28 x 10^{-4}
			0.630	0.575	640	1280	0.185	5.33 x 10^{-5}
4	BH-GD4	8.8	0.515	0.459	160	320		2.33 x 10^{-4}
			0.395	0.320	640	1280	0.249	8.39 x 10^{-5}

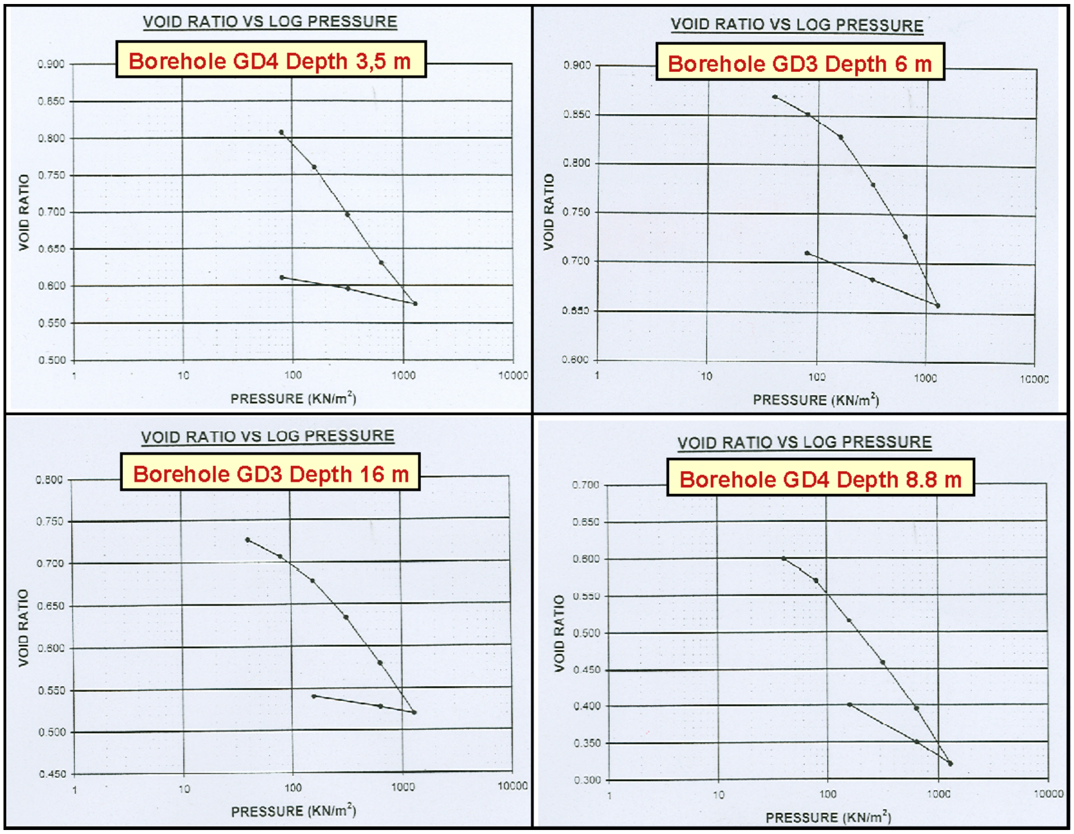

Fig. 4.8 Void ratio Vs Log Pressure curves for soil samples from foundation area

Shear strength is the principal engineering property, which controls the stability of a soil mass under loads. The shear strength from the secondary data from the project Authority is presented in Table 4.18.

Perusal of Table 4.15 indicates that the angle of shearing resistance for the foundation soils in the tested section varies from 25^o to 35^o, whereas the cohesion varies from 31 to 35 KN/m^2.

Table 4.18 Shear strength of foundation material

Sample No	BH-Name	Depth of Sampling (m)	Type of test	Sample type	Cohesion KN/m^2	Angel of shearing resistance(φ)
1	BH-GD3	6.0	Triaxial UU	Remolded	35	29˚
2	BH-GD3	12	Triaxial UU	Remolded	31	25˚
3	BH-GD4	3.5	Triaxial UU	Remolded	31	30˚
4	BH-GD4	13	Triaxial UU	Remolded	33	35˚

4.8.3 Permeability of soils

The property of a soil, which permits flow of water through it, is called the permeability. A soil is highly pervious when the water can flow through it easily. In an impervious soil, the permeability is very low and the water cannot easily flow through it. However, such completely impervious soil does not exist in nature, as all the soils are pervious to some degree. Practically, a soil is termed impervious when the permeability is extremely low (Arora, 1997).

Permeability is a very important engineering property of soil. In the dam projects, the permeability of the foundation soils is used to calculate the seepage through foundation and later to give remedial measure to solve the problem. The permeability of soils is also required in the design of filters used to prevent piping in hydraulic structure such as dams.

The flow of water through soil is governed by Darcy's law. In 1956, Darcy demonstrates experimentally the laminar flow in homogeneous soils.

The velocity of flow (V) is given by;

$V = K*I$ ……….eq 4.14

Where, 'k' is the coefficient of permeability, 'i ' is the hydraulic gradient.

The velocity of flow is also known as the discharge velocity eq 4.14 is known as Darcy law. The discharge can be obtained by multiplying the velocity of flow (V) by the total cross-sectional area (A) normal to the direction of flow.

Thus,

$q = V*A = K* i* A$ ………eq 4.15

The coefficient of permeability is measured in mm/sec, cm/sec, m/sec, m/day or any other velocity units. According to USBR, the soils having the coefficient of permeability greater than 10^{-3} mm/sec are classified as pervious and those with a value less than 10^{-5} mm/sec as impervious. The soil with a coefficient of permeability between 10^{-5} to 10^{-3} mm/sec are designated as semi pervious (Bharat Singh, 1995).

To determine the coefficient of permeability of soils in the dam foundation soils a variable head permeability test has been conducted by the Project engineers in samples collected from two bore holes (BH-GD4 & BH-GD-5). The variable head permeability test is used for relatively less permeable soils where as the constant head permeability test method is used to determine the coefficient of permeability of relatively more permeable soils. Table 4.16 shows the coefficient of permeability of foundation soils.

Table 4.19 Coefficient of permeability of foundation soils

Bore hole name	Depth of Test section (m)	Coefficient of permeability (cm/Sec)	Permeability
BH-GD4	0.00-3.00	4.33×10^{-4}	Semi Pervious
BH-GD4	2.90-6.00	7.56×10^{-4}	Semi Pervious
BH-GD4	6.00-8.85	5.75×10^{-4}	Semi Pervious
BH-GD4	8.90-12	2.61×10^{-4}	Semi Pervious
BH-GD4	11.90-15	2.57×10^{-4}	Semi Pervious
BH-GD4	14.90-18.5	1.85×10^{-4}	Semi Pervious
BH-GD5	0.00-3.00	1.36×10^{-4}	Semi Pervious
BH-GD5	3.00-6.00	3.47×10^{-4}	Semi Pervious
BH-GD5	8.80-11.8	9.04×10^{-4}	Semi Pervious
BH-GD5	10.00-15	3.09×10^{-4}	Semi Pervious

From table 4.19 the foundation soils in borehole GD4, starting from the depth 0 up to 18.5 m depth, soils fall in the semi pervious range as per the USBR classification. Similarly, in borehole GD5 up to a depth of 15 m the soils are also semi pervious in nature. This indicates that the foundation soils may allow the flow of water from the reservoir. Also, as per the general suitability of the soils for engineering purposes, as proposed by USCS classification, the foundation soils, which falls in OH and OL group, show semi pervious to pervious nature. Thus it shows that foundation soils may allow considerable seepage after the impoundment of reservoir. Thus, to minimize the seepage through the foundation proper method of seepage control should be used. These methods are discussed in detail later in Chapter 5.

4.9 Engineering Geological Mapping of the Dam Site

An engineering geological map is a type of geological map, which provides a generalized representation of all those components of geological environment, which are significant in land use planning, design, construction, and maintenance, as applied to civil engineering applications. These maps may play an important role in planning and decision making in the early stages of development of a project. Another use of the map is to identify potential problems or favorable conditions at the proposed project site (Krynine & Judd, 2001).

The engineering geological mapping of the Gumara Dam site has been carried out on the basis of engineering characterization of rock and soils. For mapping purpose the rocks have been characterized based on the strength, RQD and the quality of the rock. Whereas the soils of the dam site area have been classified based on the unified soil classification system, compressibility, shear strength. Fig.4.9 presents the engineering geological map of the dam site. Table 4.17 shows the engineering geological classes, which forms the basis for the engineering geological map. Engineering geological variation in vertical section along the dam axis is shown through Fig. 4.10.

Table 4.20 Engineering geological units used for the preparation of Engineering Geological map

Engineering Geological Units	Description
Soil Units	
GP	Poorly graded gravel, gravel–sand mixtures, little or no fine Good Shear strength, Negligible compressibility, pervious permeability characteristic, good workability.
ML	Inorganic silts of low plasticity, very fine sands. Shear strength fair, medium compressibility, fair workability, semi- pervious to impervious permeability.
CH	Inorganic clays of high plasticity, fat clays. Poor shear strength, high to very high compressibility, poor workability, impervious permeability.
Rock Units	
Class II Rock	Good rock Shear strength 100-200 Mpa, cohesion of rocks mass 300-400 Kpa, friction angle of rock mass 35^0-45^0.
Class III Rock	Fair rock Shear strength 50-100 Mpa, cohesion of rock mass 200-300 Kpa, friction angle of rock mass 25^0-35^0.

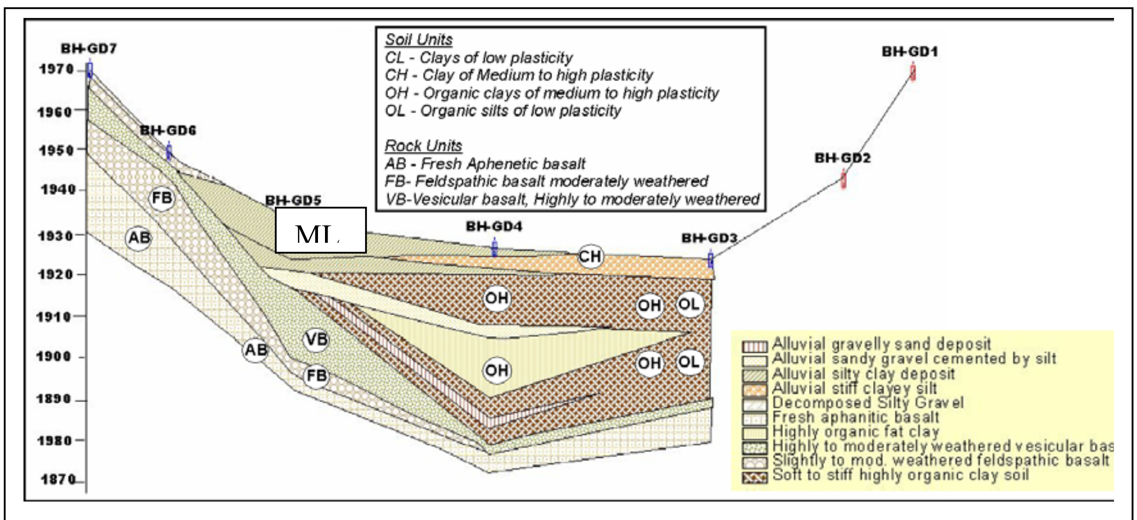

Fig. 4.10 Engineering geological variation in vertical section along the dam axis

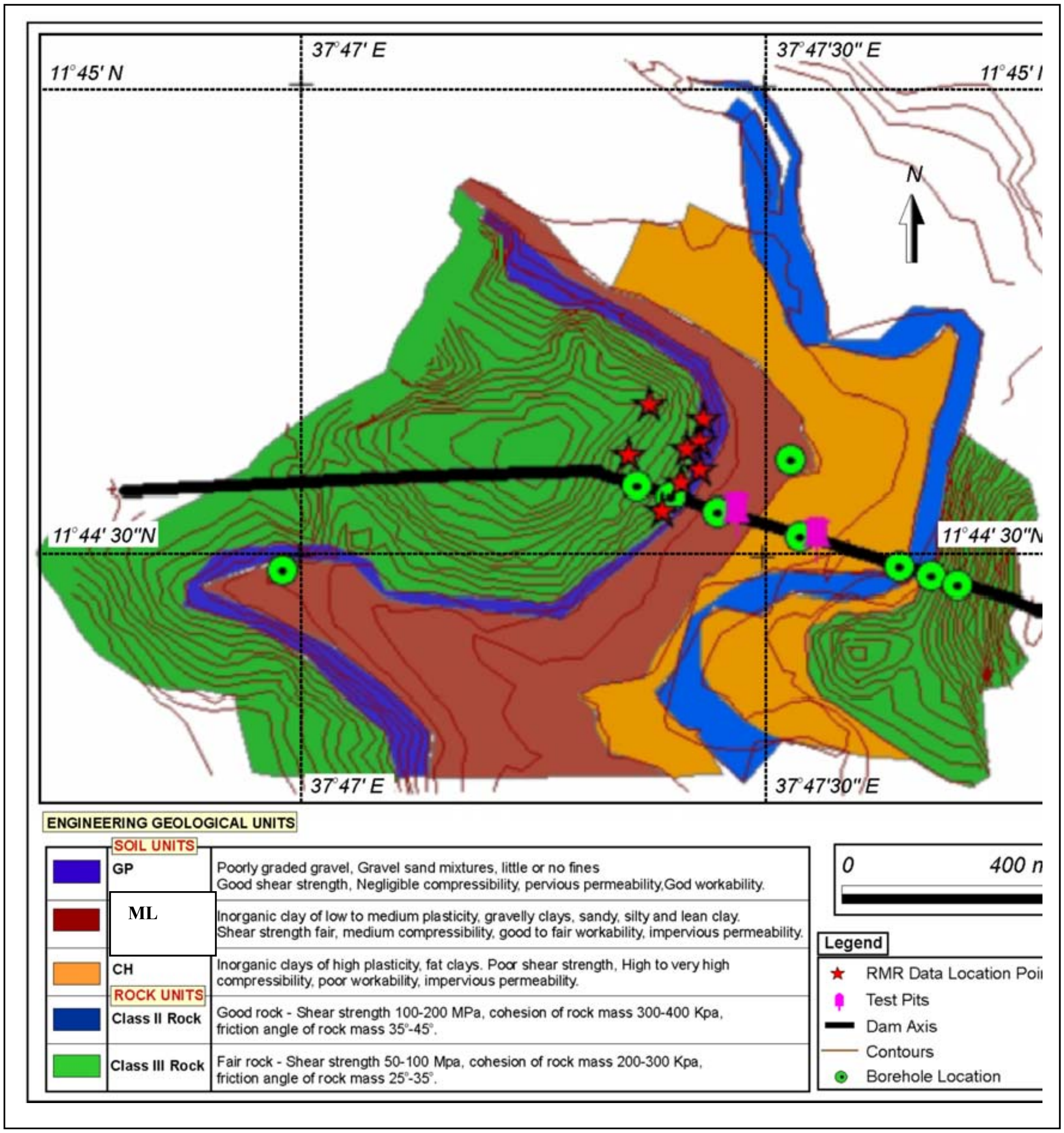

Fig. 4.9 Engineering Geological Map of Gumara dam site

4.10 Engineering Geological Appraisal of the Dam Foundation

For the present study dam foundation appraisal has been made by secondary data analysis and through field investigations. Besides, representative soil samples were also collected from the dam foundation area. Later, soil testing was done to workout index and engineering properties. Thus, an overall dam foundation assessment has been done which is discussed in the following paragraphs.

The dam foundation is composed of both soils and rocks. Both abutments are composed of basaltic rocks whereas; the river valley section is covered by thick alluvial soil deposit. Joints and dykes affect the rocks on abutments. In addition rocks at abutments are also affected by varied degree of weathering.

As per the rock mass rating the rock exposed at the dam foundation area falls in fair to good quality class. The cohesion of the rock mass at the dam site as determined from Hoek and Brown failure criteria varies from 2.5 to 3.5 MPa with an average value of 3.03 MPa. Similary, angle of friction of the rock mass at the dam site varies from 33^o to 42^o with an average value of 38.75^o.

The left abutment slope is not kinametically unstable, as it does not satisfy the condition set by Markland. Thus it may safely be presumed that the left abutment slope is stable for present geometric configuration. However, Plane J3 (N352^o/53 ^{o}) may be daylighted during the stripping operations if the slope is cut at steeper angles greater than 53^o. Therefore it is suggested that the left abutment slope should not be cut at steeper angles during stripping operations.

The foundation area is characterized by wide valley floor of thick alluvial deposits. This unit consists of silt and clay locally with pockets of medium grade and rounded river gravels along the river channel and banks. The top part of alluvial deposit including the present river channel deposit which has a minimum thickness of 5 m on the left bank of the river channel. At the foot of the left bank this alluvial deposit have a maximum thickness of about 16 m and 7 meter at the center of the dam axis. The deeper portion of the alluvial deposit starts from about 7 meter to a maximum depth of 47 m consisting of alternate layers of thick organic black silty clay and thin layers of sandy gravel of river deposit.

The soils present at shallow depth of upto 2.5m at dam foundation area are classified as ‘Inorganic clays of high plasticity’ (CH) and ‘Inorganic clays of low to medium plasticity (CL) This was deduced from the soil testing carried out during the present study. As per the general engineering suitability of soils ML, CH and MH soils possess medium to high compressibility and fair to poor shear strength. Which is undesirable for dam foundation. Further, from the secondary data analysis, all the soil samples falls below 'A-line' on the plasticity chart. Also, the samples show organic content in a range of 14.26 to 23%. Thus, it may safely be concluded that the soils falls in organic silts of low plasticity 'OL' and organic clays of medium to high plasticity 'OH' groups.

From the results of secondary data analysis (Table 4.13) it can be seen that most of the foundation soils at depth fall into organic clays of medium to high plasticity 'OH' group. According to Arora (1997) the soils falling in OH group possess high compressibility and poor shear strength. Thus, these soils may not be suitable for the foundation as these may result into high compressibility and may be responsible for the development of cracks in the core.

The tests conducted by project engineers for consolidation of soils indicates that compression index (Cc) varies in the range of 0.185 to 0.249 for the pressure variation of 640 to 1280 KN/m^2, whereas coefficient of volume change varies from 5.33×10^{-5} to 8.33×10^{-5} for the same pressure range. Also, the angle of shearing resistance for the foundation soils in the tested section varies from 25^o to 35^o, whereas the cohesion varies from 31 to 35 KN/m^2.

The bedrock in the valley floor is permeable upto a depth of 42m where it demonstrates turbulent flow. Further, in the valley floor rocks at a depth below 51m are non-permeable indicating that the joints are tightly closed. Thus, there is a need to provide treatment to improve the permeability condition of the foundation rocks. In the lower reaches of left abutment the rocks are permeable upto a depth of about 35m where these rocks demonstrate turbulent to washout flow conditions with a permeability value of 30 Lugeon. In the intermediate section of the left abutment the rocks are moderately permeable and showed laminar to turbulent flow conditions. In this zone the permeability ranges from 2 to 13 Lugeon upto a depth of 20m. In the top reaches of the left abutment the rocks are highly permeable upto a depth of 13m where these rocks demonstrates dilation to turbulent flow conditions. This may be a result of high weathering grade of rocks due to which the joints are wide open and extends upto a depth of 13m. Further below in this section of the left abutment the permeability condition improves and it reduces to 4 Lugeon. It implies that the joints are comparatively tight below 17m depth. Further, so far no water pressure tests has been conducted on the right abutment. The geological condition on right abutment is similar to that of left abutment. The rocks on the surface are highly weathered and joints are wide open. Thus, it may be presumed that the permeability conditions will also be similar to that of the left abutment. In this regard the exact permeability condition will only be assessed quantitatively when more water pressure tests will be conducted on the right abutment.

The permeability test conducted by the project engineers on the soil samples from foundation area indicates that the foundation soils in general are semi-pervious (1.36 x 10^{-4} to 7.56 x $^{10-4}$ cm/s) which may allow the flow of water from the reservoir. Also, as per the general suitability of the soils for engineering purposes, as proposed by USCS classification, the foundation soils, which falls in OH and OL group, show semi pervious to pervious nature. Thus it shows that foundation soils may allow considerable seepage after the impoundment of reservoir. Thus, to minimize the seepage through the foundation proper method of seepage control should be used.

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Foundation Evaluation and Treatment Chapter V

5.0 Introduction

Foundation investigations and the interpretation of the data obtained are required to ascertain whether a safe and economical structure can be built at a selected site. Experience has shown that many of these failures were the result of poor foundations or a lack of knowledge of the site conditions. A considerable number of dam failures probably originated from ineffective foundations. Thus, investigations of foundation condition leads to the selection of safer site or to the adoption of the design and construction provisions necessary to overcome foundation defects. Surface and sub surface investigation in addition to the laboratory tests at the foundation area should be adequate to determine the suitability of the foundation material, the required foundation treatment and the excavation of stable slope. The required foundation treatment may be the major factor in determining project feasibility. These investigations should cover classification, physical properties, location and extent of soil and rock strata and in piezometeric levels in ground water at different depths (Bell, 1978).

For the present study based on the surface, subsurface investigations and laboratory test results an overall appraisal of foundation condition of Gumara Dam has been made. In order to over come the various problems associated with the dam foundation suitable remedial measures have been worked out.

5.1 Evaluation of Foundation Suitability for Embankment Dam

For a stable and technically sound embankment dam the foundation and abutment must be stable during the construction and reservoir operation and seepage through the embankment, foundation and abutments must be collected and controlled to prevent excessive uplift pressures, piping, sloughing, and removal of material by solution.

In the present study the suitability of the dam foundation is evaluated according to:

(i) The sub-surface index and engineering properties of the soil strata, the bed rock conditions and its depth.
(ii) Abutment slope stability condition.
(iii) The water-tightness or the potential for the seepage through the foundation materials.
(iv) The compressibility and settlement condition of the foundation soils.

5.1.1 Sub surface condition

The Gumara dam site foundation is composed of both soils and rocks. The valley part is covered by thick alluvial soil deposit and the banks are composed of basaltic rocks. Joints and dykes affect the rocks on abutments. In addition rocks at abutments are also affected by varied degree of weathering.

The foundation comprise of the left abutment, the valley floor, the river channel and the right abutment. Top reaches of the left abutment is composed of slightly to moderately weathered feldsphatic basalt and highly to moderately weathered vesicular basalt upto a depth of 20 m. The lower reaches of left abutment is covered by an alluvial poorly sorted sandy silt deposit, as observed in borehole (BH-GD5), for 14 m thickness. Further, below from elevation 1921m to 1917m an alluvial sandy gravel cemented by silt deposit is present. This may indicate the past depositional sequence in the river channel of Gumara. The valley floor of the foundation is composed of 7 m thick organic clay soil deposit found below the alluvial silty clay soil deposit. This thick black organic soil extends to a depth, as observed in BH-GD4, from 1920 m elevation to 1880m elevation. In addition, this organic soil contains thin layer of gravelly sandy deposit. The bed rock in the valley floor are found at a depth of 47 m and about 38 m thickness of the over burden in boreholes BH-GD4 and BH-GD3, respectively. The river channel is covered by recent alluvial sandy gravel deposit. Both, left and right abutments are covered by feldspar rich basaltic rocks. The exposed part of this layer is affected by intense widely spaced joints and varied degree of physical weathering.

The foundation area is characterized by wide valley floor of thick alluvial deposits. This unit consists of silt and clay locally with pockets of medium grade and rounded river gravels along the river channel and banks. The top part of alluvial deposit including the present river channel deposit which has a minimum thickness of 5 m on the left bank of the river channel. At the foot of the left bank this alluvial deposit have a maximum thickness of about 16 m and 7 meter at the center of the dam axis. The deeper portion of the alluvial deposit starts from about 7 meter to a maximum depth of 47 m consisting of alternate layers of thick organic black silty clay and thin layers of sandy gravel of river deposit.

The soil testing conducted during the present study reveals that the soils present at shallow depth of upto 2.5m at dam foundation area are classified as 'Inorganic clays of high plasticity' (CH) and 'Inorganic clays of low to medium plasticity (CL). As per the general engineering suitability of soils both CL and CH soils possess medium to high compressibility and fair to poor shear strength. This is undesirable for dam foundation. Further, from the secondary data analysis, all the soil samples falls below 'A-line' on the plasticity chart. Also, the samples show organic content in a range of 14.26 to 23%. Thus, it may safely be concluded that the soils falls in organic silts of low plasticity 'OL' and organic clays of medium to high plasticity 'OH' groups. According to Arora (1997) the soils falling in OH group possess high compressibility and poor shear strength. Thus, these soils may not be suitable for the foundation as these may result into high compressibility and may be responsible for the development of cracks in the core. Fig. 5.1 shows the geological cross section along the dam axis.

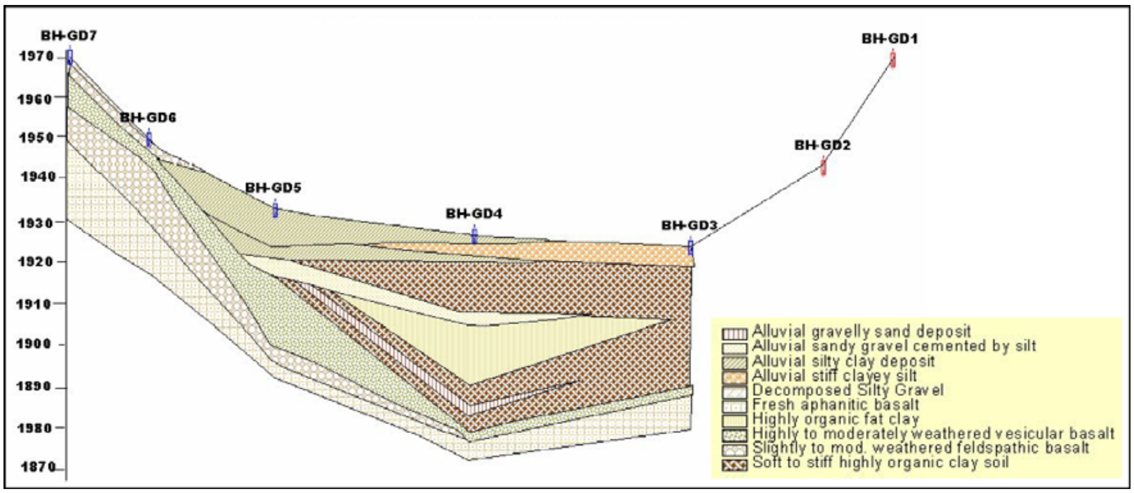

Fig. 5.1 Geological cross-section along the dam axis

5.1.2 Abutment Sections

Both left and right abutments are covered by feldspar rich basaltic rocks. The exposed rock in this layer is affected by intense widely spaced joints and varied degree of physical weathering. On left abutment four joint sets are present which are J1(N160^{o} / 78^{o}), J2(N109^{o} / 68^{o}), J3(N352^{o} / 53^{o}) and J4(N258^{o} / 70^{o}). The rock mass exposed on the left abutment falls into Class-II and Class-III as per the Bieniawiski's Rock mass rating system. Thus the rock mass is of Fair to good quality. The overall average cohesion (C) is 3.1 MPa and angle of friction for the rock mass is 36.6^{0}, as determined from Hoek and Brown failure criteria.

The left abutment slope is not kinametically unstable, as it does not satisfy the condition set by Markland. Thus it may safely be presumed that the left abutment slope is stable for present geometric configuration. However, Plane J3 (N352^{o}/53^{o}) may be daylighted during the stripping operations if the slope is cut at steeper angles greater than 53^o. Therefore it is suggested that the left abutment slope should not be cut at steeper angles during stripping operations.

5.1.3 Stripping limit of the abutments

The foundation rock should be resistant to disintegration and erosion. The foundation of the dam should be laid down on the fresh rock. For this stripping of abutment slope is done. In order to know the stripping limit, proper geological mapping should be done to know the depth up to which gliding cracks are present. The stripping of the abutment slope is done up to the lateral depth beyond the glide cracks zone.

For the present study attempt has been made to work out the stripping limit from the borehole logs for left abutment. For this purpose RQD values from borehole data for BH-GD6 and BH-GD7 were examined. From the RQD values it has been observed that the fresh rock appears at a depth of 7m in BH-GD6 whereas in BH-GD7 the fresh rock appeared at a depth of 13m. Therefore, it will be safe to strip off weathered rock upto a depth of 7m along BH-GD6 section whereas in BH-GD7 section the stripping limit may be kept upto a depth of 13m.

For right abutment no drilling is conducted so far and no sub-surface data is available. Therefore, stripping limit for right abutment could not be worked out during the present study. However, from the surface geological conditions it appears that the geological conditions are similar to that of left abutment. Thus, it may be presumed that the stripping limit may also follow the same trend as what estimated for left abutment. However, the stripping limit on right abutment has to be evaluated only after the availability of actual sub-surface investigation data. Fig. 5.2 shows the stripping limit for the abutments.

5.1.4 Seepage and Leakage Condition

Seepage through the abutments

In order to know the permeability of the left abutment water pressure test has been conducted by the project engineers in two boreholes BH-GD6 and BH-GD7. In the top reaches of the left abutment the rocks are highly permeable upto a depth of 13m where this rock demonstrates dilation to turbulent flow conditions. This may be a result of high weathering grade of rocks due to which the joints are wide open and extends upto a depth of 13m.

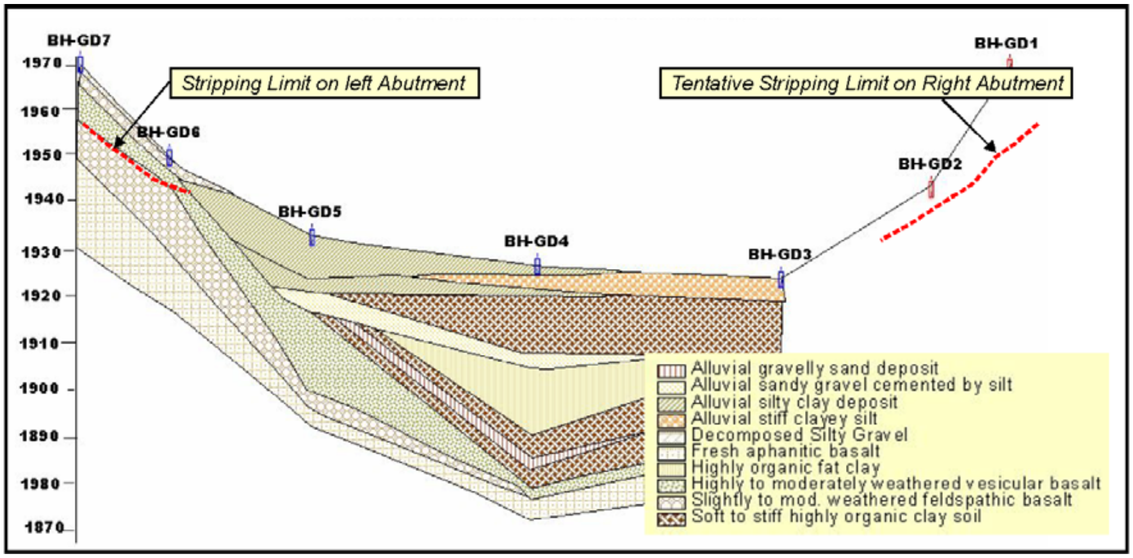

Fig. 5.2 Stripping limit for the abutments

Further below in this section of the left abutment the permeability condition improves and it reduces to 4 Lugeon. It implies that the joints are comparatively tight below 17m depth. In the intermediate section of the left abutment the rocks are moderately permeable and showed laminar to turbulent flow conditions. In this zone the permeability ranges from 2 to 13 Lugeon upto a depth of 20m. In the lower reaches of left abutment the rocks are permeable up to a depth of about 35m where these rocks demonstrate turbulent to washout flow conditions with a permeability value of 30 Lugeon. The overall permeability condition in the left abutment indicates a need for proper treatment, which is later discussed in this chapter.

For right abutment, so far no water pressure test has been conducted. However, during the present field-work attempt has been made to qualitatively assess the permeability condition of the right abutment. The geological condition on right abutment is similar to that of left abutment. The rocks on the surface are highly weathered and joints are wide open. Thus, it may safely be presumed that the permeability conditions will also be similar to that of the left abutment. In this regard the exact permeability condition will only be assessed quantitatively when more water pressure tests will be conducted on the right abutment. Fig. 5.3 shows the seepage condition of the dam foundation

Seepage in the River section

The bedrock in the valley floor is permeable upto a depth of 42m where it demonstrates turbulent flow. Further, in the valley floor rocks at a depth below 51m are non-permeable indicating that the joints are tightly closed. Thus, there is a need to provide treatment to improve the permeability condition of the foundation rocks. In the lower reaches of left abutment the rocks are permeable upto a depth of about 35m where these rocks demonstrate turbulent to washout flow conditions with a permeability value of 30 Lugeon.

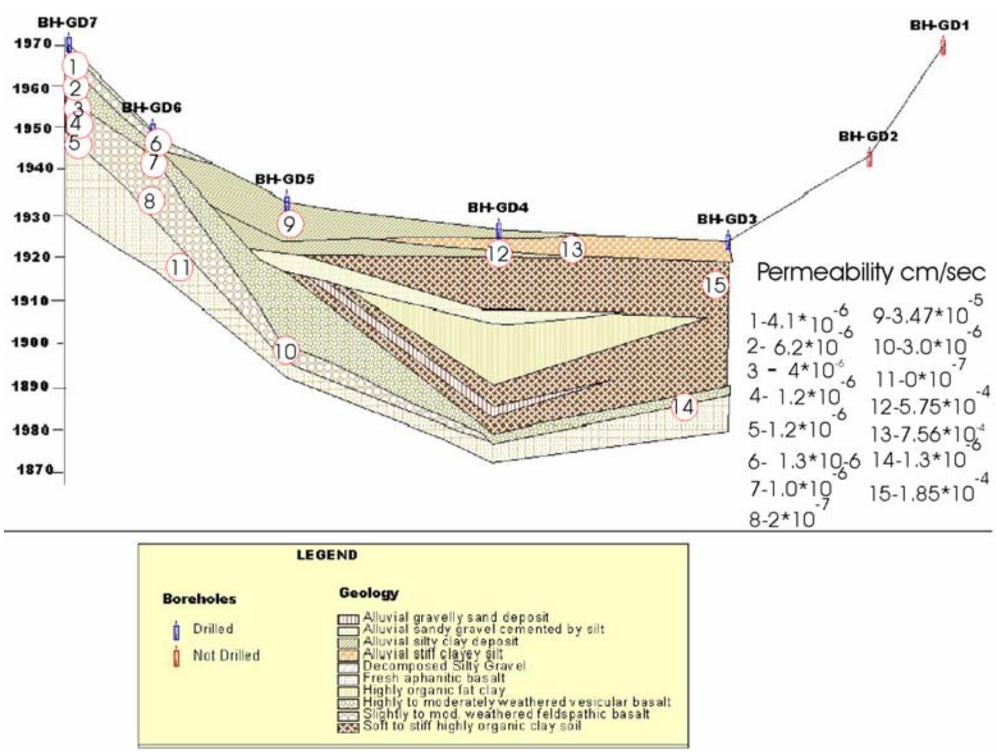

Fig. 5.3 Seepage condition along the dam foundation area

The permeability test conducted by the project engineers on the soil samples from foundation area indicates that the foundation soils in general are semi-pervious (1.36 x 10^{-4} to 7.56 x $^{10-4}$ cm/s) which may allow the flow of water from the reservoir. Also, as per the general suitability of the soils for engineering purposes, as proposed by USCS classification, the foundation soils, which falls in OH and OL group, show semi pervious to pervious nature. Thus it shows that foundation soils may allow considerable seepage after the impoundment of reservoir. Thus, to minimize the seepage through the foundation proper method of seepage control should be used, which are discussed later in this chapter.

5.1.5 Compressibility and settlement of foundation soils

Compressibility is the decrease in volume of a soil mass due to a change in the volume of voids and a lesser extent; it can also result from a change in the volume of solids.

Consolidation is the form of compressibility that occurs under a static load such as a weight of dam after construction.

The foundation soils in the river valley section are very thick which extends up to a depth of 47 m in boreholes BH-GD4 and 37 m in BH-GD3, respectively. On an average, 7 m onwards from the ground surface, the foundation soils are organic in nature where the organic content varies from 14.26 to 23% for these soils. Also, from the classification of these soils using plasticity chart all the samples fall below A-line in the groups OL and OH. Organic soils (OL& OH) have poor bearing capacity and usually exhibit large settlement under load. Due to this the foundation soils can lead to excessive settlement that may seriously damage the core by developing cracks.

The tests conducted by project engineers for consolidation of soils indicates that compression index (Cc) varies in the range of 0.185 to 0.249 for the pressure variation of 640 to 1280 KN/m^2, whereas coefficient of volume change varies from 5.33×10^{-5} to 8.33×10^{-5} for the same pressure range. Also, the angle of shearing resistance for the foundation soils in the tested section varies from 25^o to 35^o, whereas the cohesion varies from 31 to 35 KN/m^2.

Thus, from the experimental results and the general classification of the foundation soils it may be concluded that the foundation soils, which extends upto a depth of 47m are highly organic in nature and posses high degree of compressibility. Such compressible soils in the dam foundation are undesirable owing to a potential for excessive settlement. Since there is a lateral and vertical variation in the organic content of these soils therefore it may result into uneven settlement of the embankment. Such uneven settlement may result into development of cracks in the central core. This by all means is a very unsafe situation for an embankment dam.

5.2 Foundation Treatment

The following criteria must be satisfied for a stable and technically sound embankment dam;

(i) The foundation, abutment and embankment must be stable during construction and reservoir operation.

(ii) Seepage through the foundation, abutments and embankment must be collected and controlled to prevent excessive uplift pressures, piping, sloughing, removal of material by solution, formation of cracks, joints and cavities. The design should consider seepage control measures such as foundation cutoffs, adequate and non-brittle impervious zones, transition zones, drainage blankets, upstream impervious blankets and relief wells.

(iii) Freeboard must be sufficient to prevent overtopping by waves and include an allowance for the normal settlement of the foundation and embankment.

(iv) Spillway and outlet capacity must be sufficient to prevent overtopping of the embankment.

Special attention should be given to possible development of pore pressure in foundations, particularly in stratified compressible materials, including varved clays. High pore pressure may be induced in the foundation, beyond the toes of the embankment where the weight of the dam produces little or no vertical loading (Bharat Singh & Varshney, 1995).

For the present study based on the surface, subsurface investigations and laboratory test results, generated during present study and from the secondary data, an overall appraisal of foundation condition of Gumara Dam has been made. In order to overcome the various problems associated with the dam foundation suitable remedial measures have been worked out.

5.2.1 Treatment for Abutments

Both left and right abutments are covered by feldspar rich basaltic rocks. The exposed rock in this layer is affected by intense widely spaced joints and varied degree of physical weathering. Through the borehole data on left bank it has been noted that the rock is highly weathered and jointed upto a depth of 7m in BH-GD6 borehole section, whereas along BH-GD-7 borehole section the fresh rock appear at a depth of 13m. Thus rock upto these depth possess a high potential for excessive seepage and may undergo deformation when subjected to loads imposed by the embankment. Therefore, it is necessary to strip of these rocks upto these depths.

Further, through the stability analysis it has been found that the left abutment slope is not kinametically unstable for present geometric configuration. However, Plane J3 (N352^o/53 ^{o}) may be daylighted during the stripping operations if the slope is cut at steeper angles greater than 53^o. Therefore it is suggested that the left abutment slope should not be cut at steeper angles during stripping operations.

As discussed earlier, for right abutment no drilling is conducted so far and no sub-surface data is available. However, from the surface geological conditions it appears that the geological conditions are similar to that of left abutment. Thus, it may be presumed that the stripping limit may also follow the same trend as what estimated for left abutment. However, the stripping limit on right abutment has to be evaluated only after the availability of actual sub-surface investigation data.

5.2.2 Seepage Controls in Abutment and Foundation Area

The primary objective of any dam project is to impound water behind it. This water if seeps through the abutment, foundation or through the embankment in excessive quantity may cause the failure of the dam partially or fully. Therefore, it is very important to control the seepage through embankment dam. Two approaches are followed to control the seepage through the foundation and the embankment. The first approach is preventive were as, the other approach is curative. In earth dam design practice both the approaches are combined followed. In preventive approach efforts are made in keeping the water out in so far as possible while in curative approach a safe out let is provided to water, which has entered in spite of the preventive measures.

In foundation the commonly used preventive measures are cutt off trench, grout curtain, concrete diaphragm walls, sheet piles, other thin cutoffs and impermeable upstream blankets. In curative approach the requirements is drainage system such that seepage forces will not be able to cause soil migration and their magnitude and direction is such that they can not cause embankment sliding or foundation blow out. For foundation horizontal drainage blankets, partially penetrating toe drains, vertical sand drains or relief wells may be provided (Bharat singh and Varshney, 1995).

During the present study it has been evaluated that the foundation soils in the river channel section, which have a thickness of 47m, in general, are semi-pervious (1.36×10^{-4} to 7.56×10^{-4} cm/s) which may allow the flow of water from the reservoir. Also, the bedrock in the valley floor is permeable upto a depth of 42m where it demonstrates turbulent flow. Further, in the valley floor rocks at a depth below 51m are non-permeable. Thus, there is a need to provide treatment to improve the permeability condition of the foundation rocks.

Also, it has been found that in the top reaches of the left abutment the rocks are highly permeable upto a depth of 13m where this rock demonstrates dilation to turbulent flow conditions. Further below in this section of the left abutment the permeability condition improves and it reduces to 4 Lugeon upto a depth of 17m. In the intermediate section of the left abutment the rocks are moderately permeable and showed laminar to turbulent flow conditions. In this zone the permeability ranges from 2 to 13 Lugeon upto a depth of 20m. In the lower reaches of left abutment the rocks are permeable up to a depth of about 35m where these rocks demonstrate turbulent to washout flow conditions with a permeability value of 30 Lugeon.

Thus, there is a need to provide seepage control both in the abutment and the river channel section. The following seepage controlling methods are suggested to minimize the seepage through the foundation and to provide safe passage to the uncontrolled flows.

Curtain Grouting

In order to check the seepage through the bed rock and the abutment section it is suggested to provide curtain grouting in the upstream portion of the central core of the dam and it should be extended up on both the abutment sections.

Initially Lugeon postulated that foundation with a permeability of more than 1 Lu should be grouted below dams of more than 30 m height, and a permeability of more than 3 Lugeons was considered critical for smaller dams. One Lu degree of permeability is encountered in nearly tight foundation, which requires almost no grouting (Bgarat Singh &Varshney, 1995). For Gumera dam foundation it has been noted that the seepage in all cases is more than 1 Lugeon. Thus, it shows the necessarily of the curtain grouting.

Curtain grouting is performed to cut off seepage under dam or reduce it to a point that it can be controlled economically by the drainage installations. Control is accomplished by drilling and grouting one or more lines of grout holes in the foundation in the upstream portion below the central core, usually parallel to the alignment of the dam or normal to the direction of water movement. A barrier to the movement of water in the foundation is constructed by filling the voids or water passageways with grout. In theory, the barrier needs only to be a curtain of moderate width. In practice, however, the barrier obtained will be wider than necessary in some places and levels at others and possibly not wide enough (Bharat Singh & Varshney, 1995).

The depths to which grout holes are drilled should be governed by the hydrostatic head to which the foundation will be subjected and by the geologic conditions in the foundation such as the depths of impervious rock. Depths for a grout curtain should be such that the seepage path is long enough to offer sufficient resistance to seepage and to prevent the occurrence of high exit gradients near the downstream toe or excessively high uplift pressure under the downstream portion of the dam. Grout holes should bottom in sound, relatively impervious rock where possible. Final depths should never be based on precedent. A rule of thumb often used for preliminary planning of hole depth is two-thirds the hydraulic height of the dam. However, for the present case the depth upto which curtain grouting must be provided has been worked out by following Ewert's relation.

According to Ewart (1985) the depth of grout curtain can be estimated as:

$$D = 1/3\ H + C \qquad \text{..........eq.5.1}$$

Where, 'D' is the depth of curtain grout, 'H' is the height of dam and 'C' is constant depends on the foundation condition mainly, foundation material, joint intensity and the permeability, which can have a values between 8 to 25.

Thus, the computed depth for the grout curtain for the Gumera dam foundation comes out to be 37.5 m. Where the dam height is taken equal to 60 m and the foundation condition are considered to be average for which the value of C is taken equal to 17.5. The grouting in the river section may be done upto a depth of 37.5 m, however, in the abutment sections this depth can be decreased depending upon the rock condition.

The curtain grout for the present case may be provided in multiple rows. Houlsby (1976) recommended that in multiple rows curtain the outer row should be completed first. A spacing of 1.5 m between rows should be maintained, upstream row should be the tightest.

Impervious cutoff Trenches

As already discussed that the dam foundation in the present case comprises of 40 m thick organic soil strata. These soils in general, are semi-pervious (1.36×10^{-4} to 7.56×10^{-4} cm/s) which may allow the flow of water from the reservoir. Therefore, these soils require proper treatment. One of the effective treatment for such foundation condition is to provide impervious cutoff trench below the central core portion of the dam. These compacted back fill cutoff trenches are constructed from the base of an earth dam structure through the upper pervious layer down to impervious soil layers or to the bed rock. The cutoff trench is an extension of the impervious zone of the dam core. In the study area the depth of the soil strata is deep due to this constructing impervious cutoff wall may not be economically feasible. However, more detailed study is required to workout an effective depth of partial cutoff trench.

Upstream Impervious Blanket

In addition to cutoff trench an upstream impervious blanket would also be an effective measure in controlling the seepage through the foundation soils. An up stream blanket will frequently advantageous for soil founded dams, as it increases the length of the seepage paths and there by reduce uplift pressure under structures and a potential for seepage problems. The blanket may be of clay material. Typically, the blanket will be a minimum of 5 ft in thickness to reduce risk of punctures and tears and will extend upstream. From the variable falling head test of soils around the foundation area the permeability falls in the semi pervious range. Thus, to minimize the seepage through the foundation an upstream impervious blanket should be done in the upstream side of the reservoir area.

Rock Toe, Toe Drains and Filter drains

Besides partial cutoff trench and up-stream blanket some additional considerations may also be given while designing the embankment. For sure what ever precautions are taken as seepage control water will find its way through the dam foundation, therefore it is essential to provide some form of curative measures to provide safe passage to the water. This may be accomplished by providing Rock toe, toe drain, filter drain in between the foundation and the embankment. Such curative measures will eliminate the possibilities of foundation blowouts and controls the development of excessive water pressures

The downstream toe of a homogeneous dam is the most critical region in respect of seepage instability. The entire seepage through the embankment or through the foundation tends to concentrate around downstream toe if internal drainage is not adequate. The soil mass in this region is subjected to excessive seepage force. This may cause heaving and sloughing of the toe if not properly protected. Rock toe, drainage blanket and filter drains are provided on the downstream of the dam to provide controlled out let to seepage, lower the seepage line and keep it with in the downstream face, prevent heaving and piping at the downstream toe and thus improves the stability of the dam against seepage (Bharat Singh & Varshney, 1995).

For the present case rock toe of 1/4 to 1/3 the height of the dam can be provided. This rock toe should be protected by transition filter, so as to check the migration of fine particles into the rock toe. Toe drains are useful in collecting foundation seepage. The depth of the toe drains is usually 1.5 m with a minimum bottom width of 1m and side slopes 1:1. A suitable slope is to be provided to this drain so that water can drain in to the natural stream.

Relief Wells

Another additional curative measure which can be provided are pressure relief wells. These relief wells are vertically installed wells consisting of a well screen surrounded by filter material designed to prevent washout of foundation materials in to the well.

Relief wells are used extensively to relieve excess hydrostatic pressures in pervious foundation strata overlain by more impervious top strata, conditions which often exist in the downstream of dams and various hydraulic structures. Placing the well outlets in below-surface trenches or collector pipes serves to dry up seepage areas downstream of dams. Relief wells are often used in combination with other under seepage control measures, such as upstream blankets, and grouting.

5.2.3 Compressible Soils in the Foundation Area

The most sever problem investigated at the Gumera dam foundation is the presence of thick compressible organic soil strata in the central valley floor. The average thickness of this compressible soil layer is about 40m. Based on the classification test result the foundation soil of Gumara dam site is OL and OH group. According to group classification these soils, as per the UCS suitability for engineering use, posses poor bearing capacity and high compressibility. Such compressible soils in the dam foundation are undesirable owing to a potential for excessive settlement. Since there is a lateral and vertical variation in the organic content of these soils therefore it may result into uneven settlement of the embankment. Such uneven settlement may result into development of cracks in the central core. This by all means is a very unsafe situation for an embankment dam. Therefore it is necessary to provide adequate treatment for such foundation conditions. The two possible treatments which may be provided are;

(i) Removal of undesirable soils from foundation area and backfilling it with non-compressible soil.

(ii) To provide broader and flatter slopes for embankment, this means increasing the base area of the embankment.

Removal of compressible soils

The most commonly practiced mitigation method for compressible soils is removal of undesirable compressible soils from the foundation area and backfilling it with non-compressible soils. This method would not be feasible as the thickness of these compressible soils at Gumera dam site is around 40m. Removal of these soils to its full depth will require an equal quantity of non compressible fill material. This will not be economically viable for a medium height dam (60m). However, more detailed studies would be required to work out the minimum thickness upto which these soils may be removed and backfilled by non-compressible soils to result into relatively less compressible effect.

Embankment design consideration

Other alternative may be to provide broader and flatter slope for embankment, so that the dam base area is increased and the loads imposed by the dam are distributed over a wider area. This method will reduce comparatively the effective settlement of the structure. Thus, reducing possibilities of development of cracks in the central core. This method has to be supplemented by providing more core thickness so that the seepage path through the core is increased and to provide adequate transition downstream filter zone. These additional curative measures are to be provided to minimize the adverse effect of the seepage through the possible cracks in the central core. In addition to these, the seepage controlled measures discussed in the previous paragraphs may also be provided.

Moreover, such flatter and broader slope section increases the volume of embankment which requires more construction material. Thus the cost for construction and time will be increase. Thus, there is a need to carry out elaborate techno-economic feasibility study before taking any final decision on it.

Conclusion and Recommendations Chapter VI

6.1 Conclusion

The proposed research study was planned to investigate the engineering geological suitability of Gumara dam foundation. The study was aimed to determine the suitability of the dam foundation conditions against seepage, and deformation of the foundation material in response to the loads imposed by the dam. Thus, through the present study an attempt has been made to identify the adverse foundation conditions, which may pose problems during or after the construction stage. Based on the foundation conditions certain remedial measures have been worked out. This contributes towards the safety as well as the economic planning and execution of the Gumara irrigation projects.

For the present study based on the surface, subsurface investigations and laboratory test results an overall appraisal of foundation condition of Gumara Dam has been made. In order to over come the various problems associated with the dam foundation suitable remedial measures have been worked out.

The Gumara dam site foundation is composed of both soils and rocks. The valley part is covered by thick alluvial soil deposit and the banks are composed of basaltic rocks. Joints and dykes affect the rocks on abutments. In addition rocks at abutments are also affected by varied degree of weathering.

The foundation area is characterized by wide valley floor of thick alluvial deposits. This unit consists of silt and clay locally with pockets of medium grade and rounded river gravels along the river channel and banks. The top part of alluvial deposit including the present river channel deposit which has a minimum thickness of 5 m on the left bank of the river channel. At the foot of the left bank this alluvial deposit have a maximum thickness of about 16 m and 7 meter at the center of the dam axis. Below the average depth of 7 meter thick alluvial deposit along the valley floor a residual black clay soil extend to depth up to 47 meters.

The soil testing conducted during the present study reveals that the soils present at shallow depth of upto 2.5m at dam foundation area are classified as ‘Inorganic clays of high plasticity’ (CH), inorganic silt of low plasticity (ML) and inorganic clay of high plasticity (MH). Further, from the secondary data analysis, all the soil samples falls below 'A-line' on the plasticity chart. Also, the samples show organic content in a range of 14.26 to 23%. Thus, these soils falls in organic silts of low plasticity 'OL' and organic clays of medium to high plasticity 'OH' groups.

The rock mass exposed on the left abutment falls into Class-II and Class-III as per the Bieniawiski's Rock mass rating system. Thus the rock mass is of Fair to good quality. The overall average cohesion (C) is 3.1 MPa and angle of friction for the rock mass is 36.6^0, as determined from Hoek and Brown failure criteria. The left abutment slope is not kinametically unstable, as it does not satisfy the condition set by Markland. Thus it may safely be presumed that the left abutment slope is stable for present geometric configuration. However, Plane J3 (N352^o/53 ^{o}) may be daylighted during the stripping operations if the slope is cut at steeper angles greater than 53^o. Therefore it is suggested that the left abutment slope should not be cut at steeper angles during stripping operations.

For the present study attempt has been made to work out the stripping limit from the borehole logs for left abutment. From the RQD values it has been observed that the fresh rock appears at a depth of 7m in BH-GD6 whereas in BH-GD7 the fresh rock appeared at a depth of 13m. Therefore, it will be safe to strip off weathered rock upto a depth of 7m along BH-GD6 section whereas in BH-GD7 section the stripping limit may be kept upto a depth of 13m. For right abutment no drilling is conducted so far and no sub-surface data is available. Therefore, stripping limit for right abutment could not be worked out during the present study. However, from the surface geological conditions it appears that the geological conditions are similar to that of left abutment. Thus, it may be presumed that the stripping limit may also follow the same trend as what estimated for left abutment. However, the stripping limit on right abutment has to be evaluated only after the availability of actual sub-surface investigation data.

The water pressure test results indicate a necessity of grouting in the left abutment. The bedrock in the valley floor is permeable upto a depth of 42m where it demonstrates turbulent flow. Further, in the valley floor rocks at a depth below 51m are non-permeable indicating that the joints are tightly closed. Thus, there is a need to provide treatment to improve the permeability condition of the foundation rocks. In the lower reaches of left abutment the rocks are permeable upto a depth of about 35m where these rocks demonstrate turbulent to washout flow conditions with a permeability value of 30 Lugeon.

The permeability test conducted by the project engineers on the soil samples from foundation area indicates that the foundation soils in general are semi-pervious (1.36 x 10^{-4} to 7.56 x10^{-4} cm/s) which may allow the flow of water from the reservoir.

In order to check the seepage through the bed-rock and the abutment section it is suggested to provide curtain grouting in the upstream portion of the central core of the dam and it should be extended up on both the abutment sections. For the present case the depth upto which curtain grouting must be provided has been worked out by following Ewert's relation. The computed depth for the grout curtain for the Gumera dam foundation comes out to be 37.5 m. The grouting in the river section may be done upto a depth of 37.5 m, However, in the abutment sections this depth can be decreased depending upon the rock condition.

In the river section thick soil strata is present which show semi-pervious permeability condition. Seepage through these soil strata may be controlled by providing partial cutoff trench and upstream horizontal impervious blankets. In addition to these Rock toe, toe drain, filter drain and relief wells may also be provided as a curative measure to control the seepage.

The most sever problem investigated at the Gumera dam foundation is the presence of thick compressible organic soil strata in the central valley floor. The average thickness of this compressible soil layer is about 40m. Based on the classification test result the foundation soil of Gumara dam site are OL and OH group. According to group classification these soils, as per the UCS suitability for engineering use, posses poor bearing capacity and high compressibility. Such compressible soils in the dam foundation are undesirable owing to a potential for excessive settlement. Since there is a lateral and vertical variation in the organic content of these soils therefore it may result into uneven settlement of the embankment. Such uneven settlement may result into development of cracks in the central core. This by all means is a very unsafe situation for an embankment dam.

The most commonly practiced mitigation method for compressible soils is removal of undesirable compressible soils from the foundation area and backfilling it with non-compressible soils. This method would not be feasible as the thickness of these compressible soils at Gumera dam site is around 40m. Removal of these soils to its full depth will require an equal quantity of non-compressible fill material. This will not be economically viable for a medium height dam (60m). However, more detailed studies would be required to work out the minimum thickness upto, which these soils may be removed, and backfilled by non-compressible soils to result into relatively less compressible effect.

Other alternative may be to provide broader and flatter slope for embankment, so that the dam base area is increased and the loads imposed by the dam are distributed over a wider area. This method will reduce comparatively the effective settlement of the structure. Thus, reducing possibilities of development of cracks in the central core. This method has to be supplemented by providing more core thickness so that the seepage path through the core is increased and to provide adequate transition downstream filter zone. These additional curative measures are to be provided to minimize the adverse effect of the seepage through the possible cracks in the central core. In addition to these, the seepage control measures may also be provided. Moreover, such flatter and broader slope sections increase the volume of embankment, which requires more construction material. Thus the cost for construction and time will be increase. Therefore, there is a need to carry out elaborate techno-economic feasibility study before taking any final decision on it.

6.2 Recommendations

From the review of the previous investigation reports and the investigations made during the present study it has been found that the Gumara dam foundation has serious engineering geological problems. Thus, through the present study following recommendations are made;

(i) The rocks on left abutment are moderately to highly weathered upto a depth of 7 to 13 m. Therefore it is necessary to strip off these weathered rocks upto a depth of 7 to 13 m to avoid excessive seepage through these weathered rocks. It is recommended to conduct drill holes on the right abutment along the dam axis so that the sub-surface geological conditions are known.

(ii) Since the rocks in the abutment and river section are permeable therefore it is suggested to perform curtain grouting below the central core portion. The grout curtain may be extended upto a depth of 37.5 m in the river section whereas this depth may be reduced in the abutments depending upon the rock condition. The curtain grout for the present case may be provided in multiple rows. Houlsby (1976) recommended that in multiple rows curtain the outer row should be completed first. A spacing of 1.5 m between rows should be maintained, upstream row should be the tightest.

(iii) In the river section thick organic soil strata is present which on an average extends upto a depth of about 40m. These soils are semi-pervious in nature and are highly compressible. In order to control the seepage through this soil strata partial cutoff trench and upstream impervious blanket may be provided. In addition some curative seepage control measures in the form of rock toe, filter drain, toe drains and relief well may be provided.

(iv) One of the serious problems at Gumara dam foundation is the presence of thick organic compressive soil strata in the river section, which extends upto a depth of 47 m. Two possible treatments can be applied to reduce the adverse effect of these soils. The first one is partial removal of undesirable compressible soils from the foundation area and backfilling it with non-compressible soils. The second being to provide broader and flatter slope for embankment, so that the dam base area is increased and the loads imposed by the dam are distributed over a wider area. The second method includes supplementing by providing more core thickness so that the seepage path through the core is increased and to provide adequate transition downstream filter zone. These additional curative measures are to be provided to minimize the adverse effect of the seepage through the possible cracks in the central core. However, there is a need to carry out elaborate techno-economic feasibility study before considering any of the two methods.

(v) The Geophysical investigation methods such as vertical electrical sounding should be conducted before borehole drilling for projects identified for feasibility study around the project area from economic point of view. This is because there is thick thickness of the organic soil around the area as seen in the Gumara dam site A and on the alternative Gumara Dam site A on sendega Gumara which is 6 km far from Gumara dam site B.

(vi) 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.

References

Agarwal, C.K., Mehrotra, V.K. and Mitra, Subhash, 1991, "Need of Long Term Evaluation of Rock Parameters in the Himalayas Proc.", 7th International Congress Rock Mech., Aachen, Germany.

Arora, K.R., 2000, "Soil Mechanics and Foundation Engineering", Delhi.

Barton, N., and Choubey, V., 1977, "The Shear Strength of the Rock Joints in Theory and Practice", Rock Mech., V.10, pp., 1-54.

BECOM (1999): Abbay River Basin Integrated Development Master Plan Project, Addis Ababa

Bell,F.G, 1978 "Engineering Geology and Geotechnics", Newnes-Butterworths, London, UK.

Bell,F.G, 1980 "Engineering Geology and Geotechnics", Newnes-Butterworths, London, UK.

Bieniawski, Z.T., 1976, "Rock Mass Classification in Rock Engineering, Proc. Symp. Exploration for Rock Engineering (Ed. Z.T. Bieniawski), V. 1, pp. 97-106.

Bieniawski, Z.T., 1976, "The Geomechanics Classification in Rock Engineering Applications", Proc. 4th Int. Cong. Rock Mech., Montreux., V.2, pp. 41-48.

Bieniawski, Z.T., 1989, "Engineering Rock Mass Classifications", John Willey and Sons, New York.

Getaneh Assefa (1981): Gohatsion Formation: A new Lias – Malm lithostratigraphic unit from the Abay River Basin, Ethiopia. *Geoscience Journ*., 2: 63-88; Lucknow

Getaneh Assefa (1991): Lithostratigraphy and environment of deposition of the Late Jurassic – Early Cretaceous sequence of the central part of Northwestern Plateau, Ethiopia. *N. Jb. Geol. Palaeont. Abh*. 182(3) 255-284, Stuttgart

Goodman, R.E., 1989, "Introduction to Rock Mechanics", John Wiley and Sons, New York.

Hoek, E., 1976, "Rock Slopes, Rock Engineering for Foundation and Slopes", V. 1, pp. 157-171.

Hoek, E., Bray, J.W. and Boyd, J.M., 1973, "The Stability of Rock Slopes Containing a Wedge Resting on Two Intersecting Discontinuities", Quarterly J. Engg. Geol, V. 6, No.1.

Hoek, E and Bray, J.W., 1997, "Rock Slope Engineering", E&FN Spon and Imprint of Chapman and Hall, 2-6 Boundary Row, London SEI 811 N, UK.

Hoek, E, Kaiser P.K and Bawdern W.F,1995 "Support of underground Excavations in Hard Rock", A.A. Balkema, Rotterdam, Brookfield, VT 05036, USA.

J. Chorowicz et al, 1998 The Tana Basin, Ethiopia; intra-plateau uplift, rifting and subsidence. Tectonophysics, 295: 351-361, Paris

Johnson, R.B and Degraff, J.V, 1991 "Principles of Engineering Geology", John Wiley and Sons, New York..

Kazmin V., (1972): Geology of Ethiopia, Explanatory note to the geological map of Ethiopia 1:2,000,000, Geopolitical survey of Ethiopia, Addis Ababa

Krynine, D. Pand Judd, W.R, 2001 "Principles of Engineering Geology and Geotechnics", Mc Graw Hill Book Co., Inc., New York,U.S.A

Leake Mariam Asfaw, (91996): catalogue of Ethiopian Earth Quakes, Earth quake parameters strain release and seismic risk, Geophysical observatory, Faculty of science, Addis Ababa university.

Mahadevaswamy, G.M, 2002 ' Hydro geologic Appraisal using conventional and remote sensing methodology of Lokapavani Micro water Shed, Mandaya District, Karnataka ,India', Unpuplished Ph.D Thesis, university of Mysore.

Mulugeta Feseha (2002). Sequence Stratigraphy, Petrography and Geochronology of the Chilga Rift Basin Sediments, Northwest Ethiopia. Ph.D. Dissertation, Univ. Texas.

Palmstorm, A. 1982, "The Volumetric Joint Count – a Useful and Simple Measure of the degree of Jointing IV Int. Cong., IAEG, New Delhi, India, pp. V 221-V228.

Raghuvanshi, T. K, 1996 '' Engineering Geological Appraisal of Kishau Dam Project, Garhwal Himalaya, India'', Unpublished Ph.D Thesis, Indian Institute of Technology Roorkee, India

Russo A. Getaneh Assefa & Balemwal Atnafu (1994): Sedimentary evolution of the Abay River (Blue Nile) Basin, Ethiopia. *N. Jb. Geol. Palaeont. Abh.* 5: 291-308, Stuttgart

Serafim, J.L and Pereira, J.P., 1983, "Considerations of the Geomechanical Classification of Bieniawski", Proc. Int. Symp. Engg. Geol, Underground Construction Boston, pp., 33-43.

Sharma, S., Raghuvanshi, T.K. and Anbalayagan, R., 1995, "Plane Failure Analysis in Rock Slopes, Geotech. Geology, Engg.", V. 13, pp. 105-111.

Singh, B, and Goel, R.K., 1998, "Software for Engineering Control of Landslide and Tunnelling Hazard", A.A. Balkema Publishers, Netherlands.

United State Bureau of Reclamation (USBR)(1964): Land and Water Resource Development in Blue Nile Basin.

Water and Power Consultancy Services (WAPCOS) (1990): the preliminary water resource development plan for Ethiopia.

Water Works Design and Supervision Enterprise' and ICT (2006): Geological and Geotechnical Investigations Report.