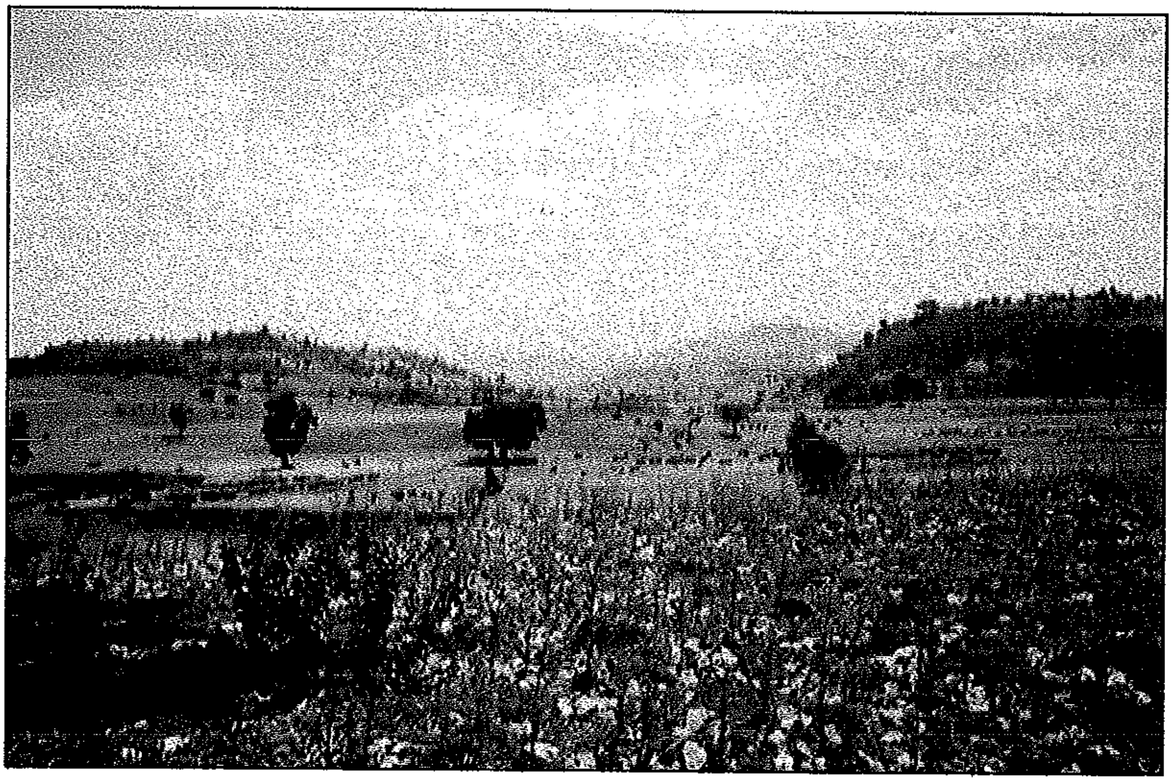
ADDIS ABABA UNIVERSITY
School of Graduate Studies
Faculty of Science

Engineering Geological Study on Rib Dam Project, South Gonder (North-Western Ethiopia)



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GETINET ASFAW

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ABSTRACT

Rib Dam project is located in the North-Western part of the Ethiopia, defined by the geographic coordinate of 12° 02' 30" N and 37° 59' 45" E. The primary purpose of this dam project is to provide irrigation to 19000 hectare of land. The proposed dam height would be 54 m with a crest length of 700 m. The total storage capacity of the reservoir would be about 204 M m^3.

In the present study engineering geological characterization of soils and rocks of the study area has been carried out to evaluate the dam foundation condition and the reservoir water tightness. In addition to this stability condition of the abutment and reservoir slopes has also been studied. Besides, the suitability of the construction material available in the vicinity of the project area has also been studied.

The engineering geological appraisal of the dam project has been after carrying out insitu and laboratory testing on soils and rocks and determining the index and engineering properties.

Geological maps of the reservoir area and the dam site has been produced at the scale of 1:25000 and 1:10,000, respectively. Besides, based on the engineering characterization of rocks and soils of the area Engineering geological maps were also produced for the dam site and the reservoir areas on the scale of 1:2000 and 1: 10,000, respectively.

Thus, based on the engineering geological appraisal of the site the dam foundation area has problem of seepage at the abutments and settlement at the valley floor area. Hence certain recommendations to improve the foundation strength and seepage conditions have been made. Besides recommendations on the construction material and the water tightness of the reservoir have been made. In addition, further detailed studies on the dam foundation conditions and the reservoir area have been recommended.

CHAPTER I

INTRODUCTION

1.0 GENERAL

Among the various features which directly influence the design of a dam, geology is the most important, because this factor controls not only the character of the foundation, but it also governs the construction material available for the construction. It is worth mentioning that most unanticipated difficulties are related to the geological setup of the site rather than the engineering design and the workmanship. Nearly up to first half of the 20th century dams were designed and constructed without proper investigations and systematic studies of the geological environment on which the dam and its appurtenant structures would be founded. In recent years the role of engineering geology has been wide spread with its application in all stages of development of civil engineering projects. Since, the knowledge of this science provides substantially towards the economy and stability of any civil engineering project.

The primary objective of geological site investigation for a dam project is to provide the information and realistic data for the dam design which will be essential for the safe design and will be utilized to estimate, with reasonable accuracy, how much the dam is going to cost. The aim of the dam designer is to build the dam for the lowest cost consistent with the currently accepted standard of safety. This can be done through site investigation program that uses wide range of field data gathering techniques such as, outcrop mapping, sampling, field testing joint survey, laboratory testing of rock and soil samples etc. The geological and physical data as obtained are then integrated to form the geo-mechanical model of the site which provides the design engineer with reasonably realistic and quantitative basis on which he may design the dam and its associated structures. This process of collecting relevant geological data and presenting the processed data, to be used for design purpose, is the main function of the engineering geologist. Hence, this study is planned and designed to address such issues on Rib River dam project.

1.2 THE STUDY AREA

The study area for the present research work includes Rib dam site, its reservoir area, its appurtenant structures and its construction material sites. Rib dam site is located in Amhara regional State, South Gondar zone in Ibnat District. The dam site is located at the geographic coordinates 12° 02' 30" N and 37° 59' 45" E. The site is accessible by Addis Zemen - Ibnat road, diverting at Zaha village, some 20 km from Addis Zemen and further it is 10 kms by dry weather road. Figure 1.1 shows the location of the study area. The project site is 676 km from Addis Ababa. The topography of the area is characterized by broad and flat flood plains, old bench forming terraces and low to high relief basaltic hills with steep to moderately steep slopes which are found bordering the proposed reservoir area. The elevation range in the river basin area varies from 4000 m to 1800 m msl.

1.3 PREVEOUS WORKS

The engineering geology of the North Western high lands of Ethiopia of which the study area forms a part is not well known. Most of the past works have been done on the general geology of the region. European geologists and later Ethiopian Institute of Geological Survey have undertaken geological studies in this region since1940s. Some of the prominent studies are:

- The geology of Ethiopia by Mohr (1971)
- Explanation of the geological map of Ethiopia by Kazmin(1975)
- Ethiopian flood basalts by Mohr (1988)
- The North Western Ethiopian plateau flood basalts by Pick et al (1988)
- Unpublished MSc Thesis on Groundwater Potential Assessment by Mola (2004)
- Abay Basin Master Plan, Ministry of Water Resources (1996)

Based on these researches the project area is represented by tertiary trap series basalts. Lithologically two different basalt flows are identified, the aphanitic basalt and the amygdaloidal basalt. Soils of the study area are represented by recent and old flood plain deposits, residual and colluvial deposits.

1.4 OBJECTIVES OF THE PRESENT RESEARCH WORK

For the present study the following objectives were planned ;

General objective

- Engineering Geological appraisal of Rib dam project.
- To study the engineering characteristics of the rocks and soils at the dam site reservoir. area, spillway site, borrow area and quarry sites.
- To evaluate the stability of dam abutments.

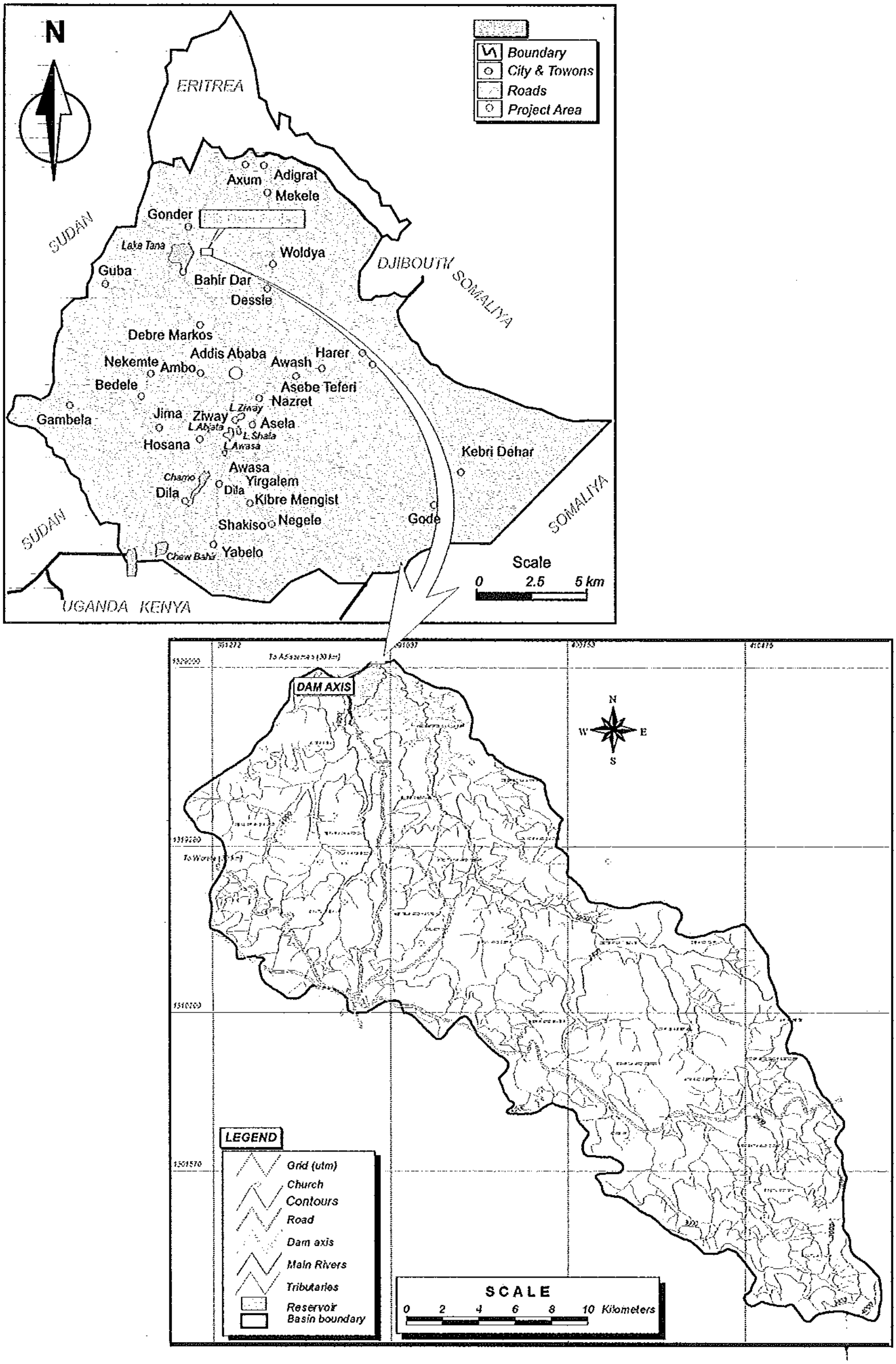


FIG. 1.1 Location map of Rib Dam Project

- Reservoir rim stability assessment
- To produce engineering geological map for the dam site and reservoir area.

* Based on the investigations and findings, to suggest suitable remedial measures.
* Recommendations.

Specific objectives

- To determine and analyze important engineering geological properties of soils and rocks in the study area.
- To identify and classify soils and rocks of the study area by their index and engineering geological properties.
- Measurement of discontinuity characteristics in the field
- Determination of preferred orientation and dip angle of joint systems
- To characterize the rocks and soils of the study area for foundations and constructions materials.
- To produce engineering geological maps at scales of 1:10,000 for the reservoir, 1:2000 for the dam site.
- Testing index and engineering properties of soils in the field and laboratories
- Determination of factor of safety of the abutment slopes.

1.5 JUSTIFICATION

The Amhara region, particularly the study area, is known for its surface water potential. However, this resource is yet to be efficiently utilized for different purposes. The rapid population growth has resulted in shortage of farm land and it has become serious problem to satisfy the growing demand of farm land.

In addition to the above mentioned problem productivity in the area is getting reduced from time to time because of crop failure associated with uneven distribution of rainfall and adequate irrigation facilities. Therefore, it is an urgent need to provide the area with water resource project, which can meet out the irrigation needs of the area. Such water resource project will not only provide a sustainable resource but, in general will improve the overall socio-economic status of the local people.

The proposed water resource project in the area may benefit the local people in the following ways;

- cultivation at least two times in a year.
- optimum utilization of the farm land.
- To supplement rain fed agriculture with irrigation water from irrigation scheme when there is shortage of rainfall.
- Dam project will minimize the flood hazard in the area.

The construction of dam require extensive geological and engineering geological studies before the actual design and construction activities. Therefore, giving special attention to the problems the Ministry of Water Resource has come up with a proposal of dam project on Rib valley. Hence, this research will provide data/information on the engineering geological characteristics of rocks and soils of the proposed dam site, reservoir area and construction material sites.

1.6 METHODOLOGY FOR THE PRESENT STUDY

In order to achieve the objectives of the present study following systematic methodology has been adopted;

i) Literature review from both published and unpublished repots of geology, geomorphology, engineering geology and hydrogeology of the study area and its vicinity.

ii) Preparation of geological map, soil map and land use map based on the previous studies and the data collected from the field during the present study.

iii) Collecting, interpreting and analyzing relevant hydro meteorological data.

iv) Conducting fieldwork to collect relevant data on geology, structural geology, engineering geology and groundwater condition of the area. This has been done through surface and subsurface investigation with the help of test pits, trenches and existing bore holes data.

v) Laboratory testing and analysis of collected samples to determine the various engineering geological and geotechnical properties of rocks and soils in the study area.

vi) Data processing, and result interpretations for recommendations.

1.7 APPLICATION OF THE RESULTS

The results of this research will be primarily vital for safe and economical design of the dam and other appurtenant structures. The result of the research will be helpful for the construction and post construction phase of the dam. The present research may provide useful data/ information for the future researchers intended to work in the similar areas. The result of the research may contribute in upgrading the data in the area as well as in the region. It may also be used by the scientific community.

1.7 LIMITATIONS OF THE PRESENT STUDY

All efforts are being made to carry out the present study in a systematic way, well supported with the actual field data and the laboratory tests. However, these efforts were being made under the limitation of resources, time and the financial constraints.

CHAPTER II

GENERAL OVER VIEW OF THE STUDY AREA

2.1 TOPOGRAHY AND DRAINAGE OF THE BASIN

The basin characteristics such as the area, land surface topography and other morphological properties vary only with respect to geological time and this may be treated as constant, accept for the conditions when catastrophic landslides take place in the basin. Topography of an area plays a significant role for subsurface runoff, ground water percolation, and soil erosion rate. The sloppy area is more susceptible for erosion and surface run off rather than infiltration and deposition.

Topography of Rib river basin ranges from flat to undulating. The flat areas are dominantly present in the lowland that are part of Fogera plain, however the gentle slopes occupies the areas between the lowland and the highland. The very steep (cliff) areas are mainly along the border of Rib river basin and mainly on Mount Gunna. Deeply dissected Gullies are present along Ribb river and at the immediate foot of mount Gunna.

Different plugs and domes characterize the basin at gently foot slope and at the immediate termination of the flat area. Southern and Eastern part of the basin are dominantly undulating and hilly. The combined effects of climate and geology on the Catchment topography yield an erosion pattern which is characterized by a network of channels or streams. Due to this fact, the drainage pattern and density in the study area varies from place to place or can be grouped into two categories. The first categories (northern and northeast of the basin) are characterized by dense drainage with dendrtic pattern. This is governed by shallow weathering layer and uniform geology of the area. Most of the precipitation in this category is not allowed to infiltrate and then to percolate in enormous amount.

The second category is present in South-East of the catchment having dendritic type of drainage and moderate drainage density compared to the first category. Geology of south-eastern area is dominated by jointed basalt with an intercalation of pyroclastic materials. Figure 2.1 presents the drainage pattern in the study area.

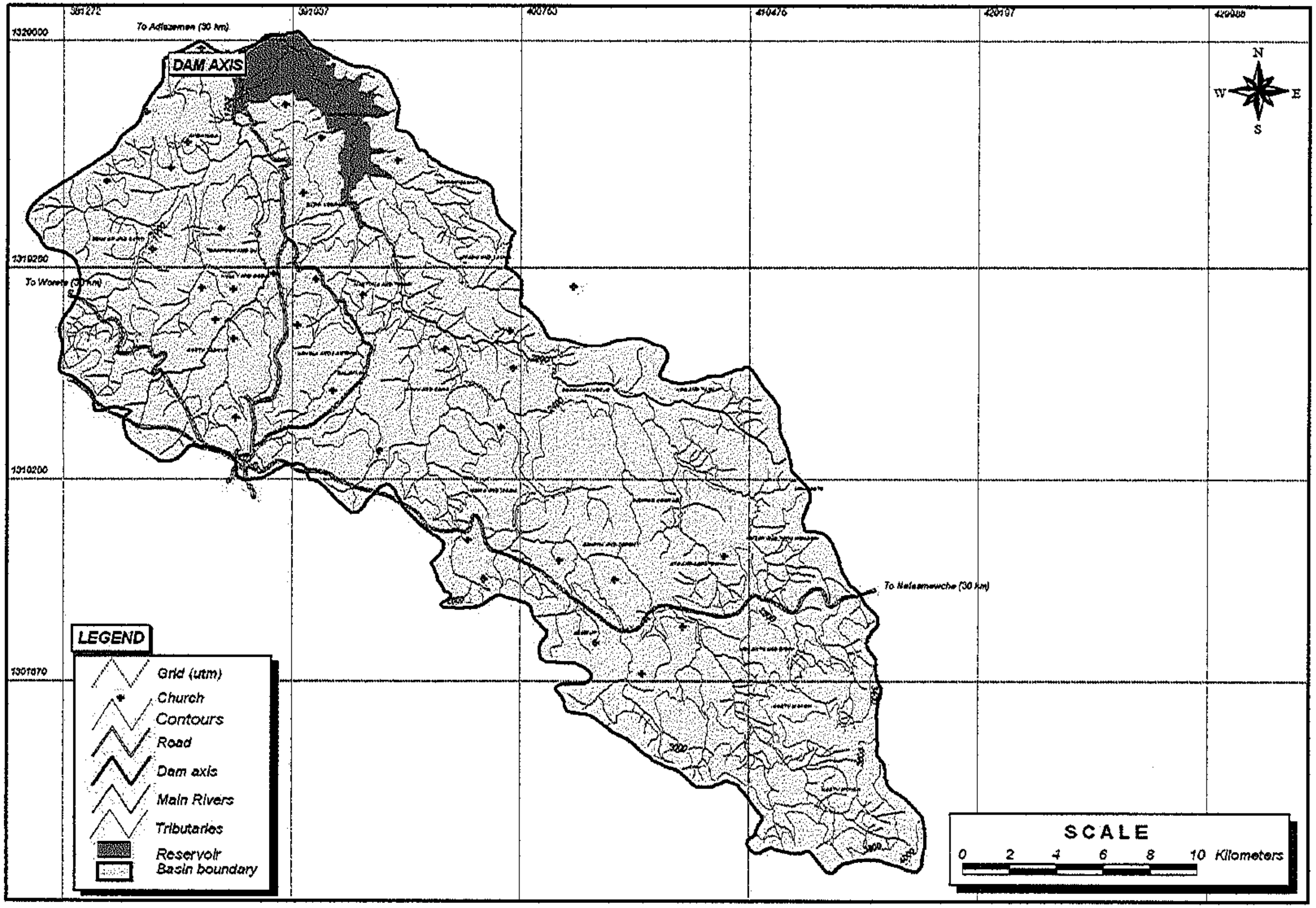


Fig. 2.1 Drainage Pattern - upstream of Rib Dam Site

2.2 LAND USE AND LAND COVER

Based on the type of covering material of the area with their present uses, major land cover/land use units of the study area are described in five units. i) Extensively cultivated , ii) Moderately cultivated with shrubby grass land, bush, open shrubby grass land and rock out crop, iii) Plantation with cultivation, grasses, and significant shrubs, iv) Afro-Alpine and v) Towns (Abay Basin Master Plan, 1996).

Extensively cultivated area covers generally, lowland areas and highland area having flat to gentle slope, whereas moderately cultivated areas cover the area with moderately steep slopes. Shrubby grass land with scatter cultivation and open wood land with bush covers the Northern periphery of the study area, which is not suitable for cultivation due to dissected terrain. Afro alpine occupies generally, high land area. In the study area Gunna and its surroundings are covered with Afro-Alpine vegetation. Generally, a large part of the basin is covered by cultivated land which is followed by shrubby grass land, bush, shrub and shrubby wood land, whereas, plantation and Alpine vegetation takes the least coverage. Table 2.1 and Figure 2.2 present the aerial coverage of land use/land cover in the study area.

Major crops grown in the Rib basin are, teff, maize, potato, bean, barley, wheat, millet and others. Common vegetations are classified according to their climate. Highland areas are dominated by-highland Eucalyptus, Juniperus, Asta and other bushes. Moderate to lowland areas are covered dominantly with, Acacia species, Lowland Eucalyptus, Crotonmarostach (Bisana), Dokma, Ficus species (Shola, Warka and Bamba), and other bushes.

Table 2.1 Aerial coverage of land use/land cover in the study area.

No	Land use/Land cover	Coverage (km^2)	Weighted Area (%)
1	Extensively cultivated	397.54	71.3
2	Moderately cultivated with shrubby grass land, bush, open shrubby grass land and rock out crop.	117.52	21.1
3	Plantation with cultivation, grasses, and significant shrubs	8.93	1.6
4	Afro alpine	32.12	5.8
5	Town	1.09	0.2
	Total	557.2	100

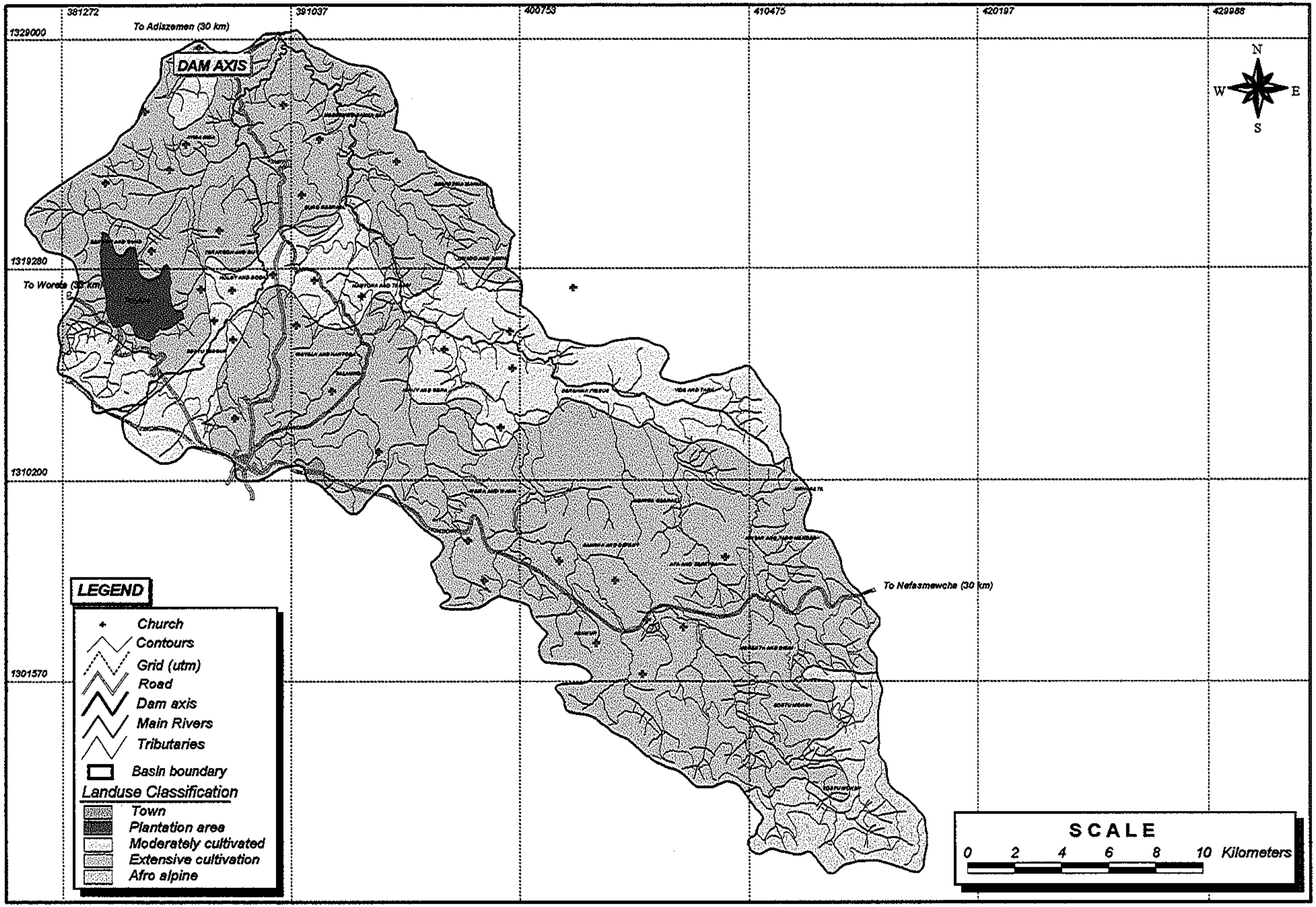


Fig. 2.2 Landuse and Landcover Map - upstream of Rib Dam Site

2.3 AGRICULTURAL SOIL UNITS OF THE RIB BASIN

2.3.1 Soil classification

Classification of soil units is based on the combined description of geomorphic and soils characteristics. The dominant soil units of the study area fall into 4 main soil groups. These classifications are taken from the study of Abay basin land resource development master plan (BCEOM, 1999) and supplemented by field visit. The main soils are silty clay, clay loam, sandy loam, and fractured rock with big boulders and cobbles. Figure 2.3 and Table 2.2 demonstrates the distribution of agricultural soil units in the study area.

Silty clay- This type of soil is found on almost flat surface on which during high rainy season, water will stay for certain days as a stagnant water and is called inundic soil. It is situated on the western extreme of the study area (Fig2.3).

Clay loam- These soils are developed in fluvial lacustrine deposit area. Clay fraction is dominated by expandable clay. These soils are shallow to deep and very poorly to moderately well drained. In the study area this soil occupies the lowland area of the basin as a continuation of Fogera plain.

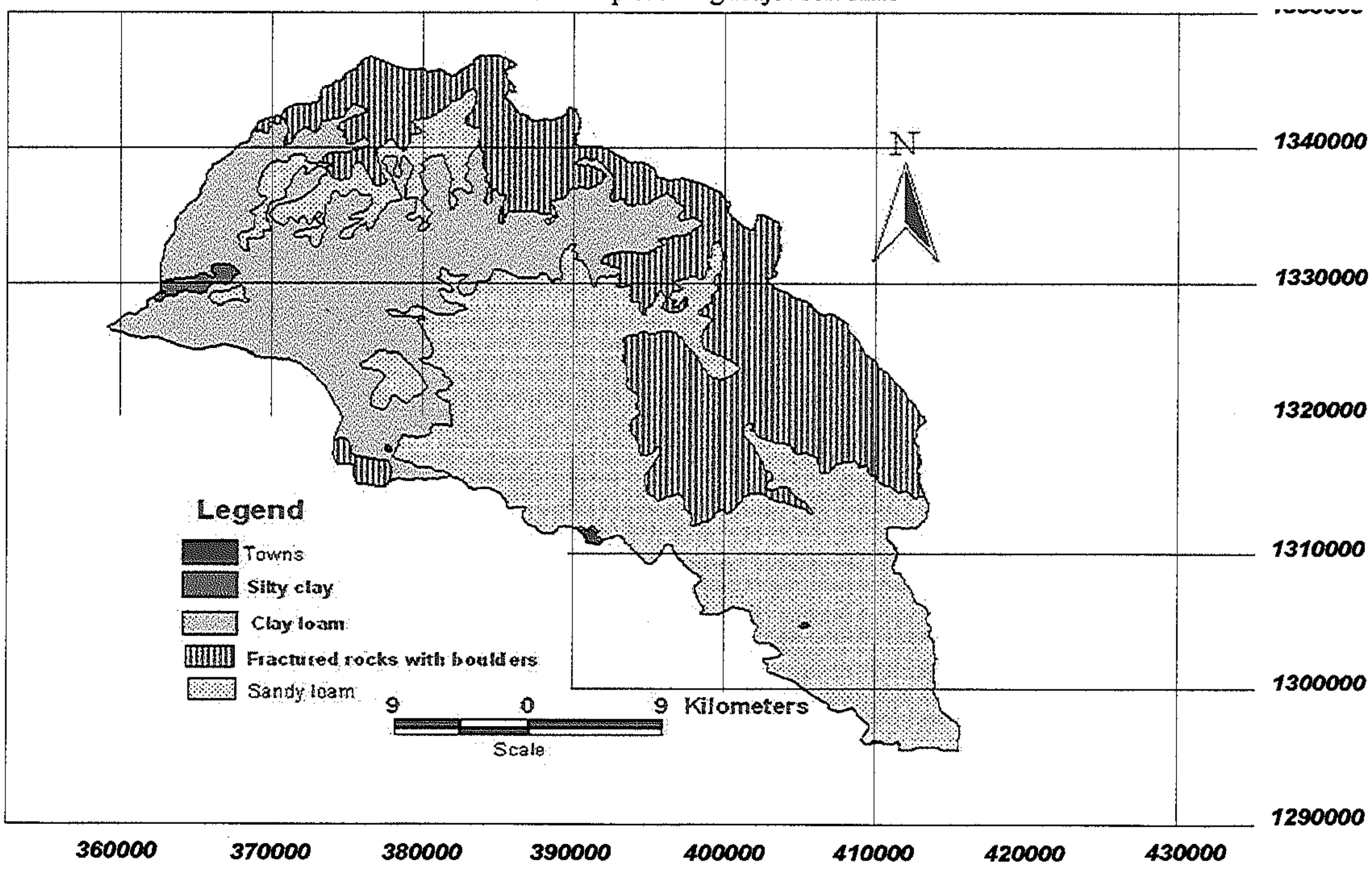
Sandy loam- These soils are developed on slightly to moderately dissected undulating to rolling low relief hill side slopes and plantation areas. The soils are moderately deep to deep, well drain, dark yellowish brown or dark grayish brown in colors. The Southeastern part of the basin is dominantly covered with this soil unit.

Fractured rock with big boulders and cobbles: These soils occur only, when severely dissected rolling to steep, low relief hill side slopes are prominent giving rise the active erosion. The soils are shallow to very shallow and genetically they are young. They mainly support scattered land cover of scattered wood. Northern and Northeast periphery of the basin is covered by this soil.

Table 2.2 Soil type and its coverage, upstream of the Rib dam site

No	Soil type	Area coverage (km^2)	Weighted area (%)
1	Clay loam	374.0	27.1
2	Silty clay	6.1	0.5
3	Sandy loam	590.5	42.8
4	Fractured rock with big boulders and cobbles	408.7	29.6

FIG 2.3 Soil map showing major soil units



2.4 CLIMATE OF THE STUDY AREA

The altitude of the basin ranges from 1800 to 4100 m.a.s.l, the climate of Rib river basin ranges from temperate (Weynadega) to cool (Kur) climatic zone (Tenalem Ayenew and Tamiru Alemayehu, 2001). For over view of the study area the temperature and rainfall are discussed briefly in this section. However, the data and detailed discussion is made in Chapter3 of this report.

Monthly mean maximum temperature in the highland is 23.9° C in the month of April at Debre Tabor Town whereas, the monthly mean minimum temperature in the same zone is 8.3°C in the month of December. In the lowland areas around Woreta, the temperature attains its mean minimum and maximum monthly temperature in the months of December and March, with values of 8 and 30.4 °C, respectively. As most of the meteorological stations (5 stations) confirmed that in the vicinity of the study area, the minimum and maximum mean monthly temperature falls within the months of August and April with values of temperature 17.2°C and 20.9°C, respectively. This condition relates with rainy season as temperature falls with the beginning of rainy season and increases with the dry reason. Temperature within the study area shows altitudinal variations with a general trend of increasing rainfall with increasing altitude.

The average annual rainfall ranges from about 1540 mm in the highlands (Debre-Tabor), South-Eastern part of the study area and 938 mm in the Northern part of the area. Rainfall distribution within the area is unimodal, which has only one peak period of rainfall and one lowest period. The lowest period of rainfall covers the months of December to February whereas, the rainy season covers the months of June to September (Molla, 2004).

2.5 SEDIMENTATION POTENTIAL OF THE AREA

The amount of sediment eroded and transported from the sources is expressed in terms of weight of sediment per unit drainage area per unit of time. The sedimentation problem is one of the important consideration in dam design. Sedimentation in the Rib area is considerably high because of steep catchment slopes, high percentage of agricultural land, less forest cover and low terracing practice in the area. The sedimentation discharge data of the Rib river is 69 ton/km^2/year whereas, from the total catchment 47.1 tone of soil is lost every year through Rib river (Abay Basin Master Plan, 1996).

CHAPTER III

HYDROMETEOROLOGY AND GROUND WATER CONDITION

3.1 GENERAL

Hydrology is the study of three important phases of a hydrological cycle, namely rainfall, runoff and evaporation (Mutreja, 1986). Engineering hydrology, however, includes those segments of hydrology that are important for the design and operation of engineering projects responsible for the control and use of water.

In planning water resource projects detailed study of hydrology is very important, For many water resource projects, determination of the peak and magnitude of flood that has to be adopted in the design of the project is of great importance. Too high design flood results in unnecessary cost, while adoption of a low design flood can (if higher flood occurs) result in loss of structure with human causalities.

Climate, topography and geology determine hydrology of a given region. Precipitation, humidity, temperature, wind speed, evaporation and sunshine duration are important factors. Engineering works require the input of hydrogeological study to assess the suitability of a site for the proposed project.

3.2 PRECIPTATION

From the hydrological view point precipitation is any form of moisture reaching the earth's surface from the atmosphere (rain, ice. etc). Precipitation data is of utmost important, as it forms the basis of planning and management of water resource projects and is used to extend records of runoff both in time and space. Hence, proper network of rain gauge is necessary to collect precipitation data for the water shed. All forms of precipitation are expressed as the vertical depth of water that would accumulate.

For the present study precipitation data for Rib River is procured from National Meteorological Service Agency. This data is available from six meteorological stations for a maximum of 33 years for Woreta station. These stations are Debre Tabor, Amedber, Woreta, Yifag, Addis Zemen and Ebenat. The location details of these stations are given in table 3.1. Table3.2 presents the mean annual precipitation of the area, as summarized from the observed data recorded at six stations (Molla, 2004). The calculated mean annual precipitation for the Rib catchment from the observed data comes out to be 1258 mm.

Table 3.1 Location of Meteorological Stations in and around study area

No	Stations	Location UTM		Altitude m.a.s.l	Recording period(year)	
		Latitude	Longitude			
1	Debre Tabor	394750	1312750	2500	1974-----2001	*1
2	Amed Ber	373100	1314000	2150	1976-----2003	*2
3	Woreta	356850	1317600	1805	1969------2002	*2
4	Yifag	360606	1334400	1835	1978------2001	*2
5	Addis Zemen	376800	1339850	2065	1975------2002	*2
6	Ebenat	407450	1341300	2340	1970------2002	*2
*1	*Records temperature, wind speed, relative humidity, sunshine hours and rainfall*					
*2	*Records temperature and rainfall*					

Table 3.2 Mean annual precipitation of the area, as summarized from the data

S.No	Station	Mean annual rainfall(mm)	Area of influence(km^2)	Weighted area (%)	Weighted rainfall(mm)
1	Debre Tabor	1540	651.59	47.24	727.5
2	Amed ber	1535	133.64	9.69	148.74
3	Woreta	1279	0.62	0.05	0.64
4	Yifag	997	88.73	6.43	64.11
5	Addis zemen	1259	383.79	27.82	350.25
6	Ebenat	938	120.94	8.77	82.26
	Total	*7548*	*1379.31*	*100*	*1373.5*

3.3 TEMPRATURE

For the present study, temperature is measured at 5 stations and the mean monthly temperature is computed as the arithmetic average of the mean daily temperature of all the days in the month. The minimum temperature of 5 stations is recorded in the month of December whereas, the maximum temperature recorded is during the months of March and April. Temperature varies spatially with altitude with a general trend of decreasing with altitude, though there is some variations in some stations at certain months. Table 3.3 gives the monthly mean maximum and minimum temperature variability, as observed at 6 meteorological stations. (ABMP, 1996)

3.4 RELATIVE HUMIDITY AND SUNSHINE HOURS

Sunshine hours are the duration of sunshine in a day. It plays as an important evaporation factor and it has a direct relationship with evaporation. This data is available only from Debre

Tabor and Yifag stations. The area attains its maximum and minimum sunshine hours during January and July, respectively (Table 3.4). Sunshine hours variation in the area is a result of cloud cover during summer and winters.

The state of the atmosphere in relation to the amount of water vapor it contains, is called humidity (Mutereja, 1986). Humidity is closely related to temperature; the higher the temperature, more vapor it can hold. Humidity is extremely important factor since atmospheric water is the source of precipitation and controls the rate of evaporation. Relative humidity is the ratio between amount of water vapor actually contained per unit volume and the maximum amount of moisture it can hold when saturated at the same temperature.

Table 3.3 Monthly mean maximum & mean minimum temperature variability

Station	Month	Jan	Feb	March	April	May	Jun	July	Aug	Sept	Oct	Nov	Dec
Amedber	Minimum	10.4	11.7	13.2	13.6	13.7	12.8	12.1	11.7	11.6	11	10.3	10
	Maximum	27.1	28.2	29.3	29.2	28.3	25.9	23.3	23.1	24.4	26.1	26.9	27
	Average	18.8	20	21.3	21.4	21	19.4	17.7	17.4	18	18.6	18.6	19
Yifag	Minimum	10.7	12.3	14.4	15.4	15.8	14.8	14.3	13.7	13.7	11.5	10.5	10
	Maximum	28.5	29.1	30.7	30.9	29.4	26.8	24.4	23.7	25.3	27	27.8	28
	Average	19.6	20.7	22.5	23.1	22.6	20.8	19.3	18.7	19.5	19.2	19.2	19
Addis Zemen	Minimum	9.9	10.8	11.4	11.7	12	11.7	11.5	11.4	11.2	10.7	10.3	10
	Maximum	29.4	30.2	30.4	30.7	30.1	28	25.3	25.4	26.1	27.6	28.5	29
	Average	19.6	20.5	20.9	21.2	21	19.8	18.4	18.4	18.7	19.2	19.4	19
Debre Tabor	Minimum	8.9	9.7	10.6	11.4	11.4	10.4	9.9	9.7	9.3	9	8.5	8.2
	Maximum	22	23.1	23.6	23.9	23.3	21.5	18.4	18.3	19.5	20.3	21	21
	Average	15.4	16.4	17.1	17.7	17.4	16	14.2	14	14.4	14.7	14.7	15
Woreta	Minimum	8.4	9.9	11	11.5	12.3	11.9	10.9	10.6	10.6	9.7	8.6	8.3
	Maximum	28.6	29.7	30.4	30.4	29.2	27.1	24.9	24.2	25.3	26.9	28	28
	Average	18.5	19.8	20.7	20.9	20.7	19.5	17.9	17.4	18	18.3	18.3	18

3.5 EVAPORATION

Water molecules are continually being exchanged between liquid and atmospheric water vapor. If the number passing to the vapor state exceeds the number joining the liquid the result is evaporation. Evaporation of water takes place from free water surface i.e lakes, reservoirs, etc. The rate is dependent upon factors such as the water temperature and the temperature and absolute humidity of the layer of air just above free water surface, solar radiation and wind especially over land. **Table 3.5 presents the evaporation from open water body using 'Penman combined method' for Ribb basin (Molla, 2004).**

Table 3.4 **Maximum and minimum sunshine hours and Mean monthly sunshine hours of the two stations**

Station	Month	Jan	Feb	Mar	Apr	May	Jun	July	Aug	Sep	Oct	Nov	Dec	total
							RELATIVE HUMIDITY							
Debre Tabor	minimum	7.5	5.2	7.1	5.8	5.8	5.8	3.5	4.7	6.4	5.6	4.4	6.6	68.4
	maximum	9.1	8.6	8.6	8.7	8.6	6	5.2	6	7.6	8.3	9.2	9	94.9
	Average	***8.3***	***6.9***	***7.85***	***7.25***	***7.2***	***5.9***	***4.35***	***5.35***	***7***	***7***	***6.8***	***7.8***	***81.65***
Yifag	minimum	8.2	8.1	8.4	8.1	7.4	5.3	4.4	4.5	5.6	7.7	6.4	5.3	79.4
	maximum	10.7	10.8	10.6	10.2	9.6	8.8	6.5	9.2	9.2	9.9	10.8	10.7	117
	Average	***9.45***	***9.45***	***9.5***	***9.15***	***8.5***	***7.05***	***5.45***	***6.85***	***7.4***	***8.8***	***8.6***	***8***	***98.2***
							SUNSHINE HOURS							
Debre Tabor		8.21	7.71	7.60	7.00	7.19	5.90	4.35	5.20	6.96	7.11	7.75	7.98	82.97
Yifag		9.57	9.60	9.17	9.04	8.39	7.28	5.94	6.54	7.71	9.18	9.58	8.40	100.39
Average		***8.89***	***8.66***	***8.38***	***8.02***	***7.79***	***6.59***	***5.15***	***5.87***	***7.33***	***8.15***	***8.67***	***8.19***	***91.68***

3.6 RUN-OFF CHARACTERSTICS OF THE CATCHMENT

Run-off is the residual of precipitation after the demands of interception, infiltration, depression storage and evapotranspiration are met and constitutes overland flow (surface run off) and inter flow (subsurface run-off (depends on geology) and base flow (ground water flow that contributes to the river) based on the characteristics of the streams. Surface run-off is water that flows over the soil surface when precipitation rate exceeds the infiltration capacity (Fetter, 1987).

Run-off characteristics of a basin are affected by the rainfall input, physical characteristics of the area, vegetative, and climatic characteristics of the basin. The catchment characteristics are greatly determined by its geology, geomorphology, area, slope, and drainage basin dynamics. Generally, runoff is the portion of the precipitation that makes its way towards stream channels and lakes consisting the three components of runoff and will be used in this context in this section. Rib River is a perennial river fed by a number of intermittent tributaries. The river is gauged down stream of the dam site, about 3 km from the dam axis. The mean monthly flow data is given in Table 3.6. and Figure 3.1 shows the Hydrograph of Rib River based on daily discharge data.

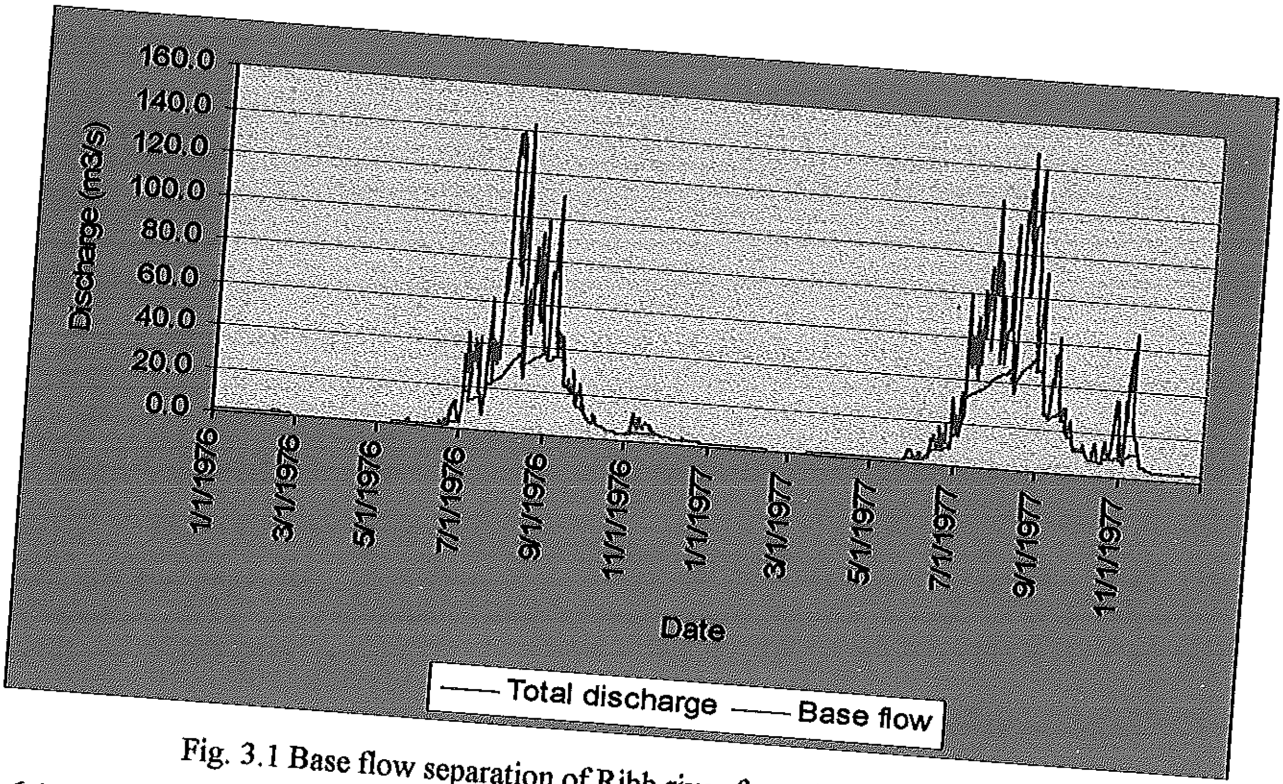


Fig. 3.1 Base flow separation of Ribb river from daily discharge

3.6.1 FLOW DURATION OF RIB RIVER

The characteristics of a basin can be established by analyzing the shape of the Flow duration curve. A flow duration curve is obtained by plotting percentage of time as the abscissa and the flow value as the ordinate. In dam projects the flow duration curve is important to calculate the total amount of the sediment transported to the reservoir site if a sediment rating curve is available for the given stream. A flow duration curve can also shows the percentage of time a specified discharge is equal to or exceeded.

For the present study flow duration has been worked out for Rib river from the observed discharge data for a period of 1994 – 2003. The computed flow duration is presented in table 3.7 and shown through Figure 3.2

The curve has steep slope up to 30% of time followed by relatively flat slope. Curves having steep slope indicate that there is significant run-off while relatively flat slopes indicate that there is substantial storage either on the surface or as ground water.

3.7 RESERVOIR STORAGE CAPACITY

The size of a reservoir is based on number of factors like topographic suitability of the area, the size of the cultivated area to be developed by irrigation, the runoff harvest potential of the adjacent catchment area and the scale of the project .etc

Table 3.5 Evaporation from open water body using penman combined method for Ribb basin

Month	Temp (Oc)	e_a (mm/day)	RH (%)	Ed (mm/d)	U2 (mile/d)	Tk (0k)	n (hr/d)	N (hr/d)	n/N	fa(n/N)	Ra (mm/d)	RI(1-r) (mm/d)	σTa^4 mm/d	Ro mm/d	H	Ea mm/d	Δ/γ	Eo mm/m
Jan	18.38	15.93	0.47	7.48	86.96	291.4	8.9	11.5	0.77	0.64	12.2	6.95	14.06	3.49	3.46	4.05	2.06	113.24
Feb	19.47	17.05	0.44	7.5	87.49	292.5	8.7	11.7	0.74	0.62.	13.6	7.62	14.27	3.43	4.19	4.59	2.18	120.84
March	20.49	18.17	0.47	8.54	89.11	293.5	8.4	12.0	0.70	0.59	14.7	7.82	14.47	3.14	4.68	4.69	2.31	145.17
April	20.85	18.58	0.51	9.47	105.75	293.8	8.0	12.4	0.64	0.56	15.2	7.65	14.53	2.78	4.87	4.96	2.36	146.9
May	20.54	18.23	0.63	12.03	98.77	293.5	7.8	12.7	0.61	0.54	15.1	7.46	14.47	2.33	5.13	3.23	2.32	141.72
Jun	19.09	16.66	0.76	12.66	91.79	292.1	6.6	12.8	0.51	0.48	15.0	6.56	14.14	1.89	4.67	1.98	2.14	114.39
July	17.50	15.07	0.88	13.26	71.39	290.5	5.2	12.7	0.41	0.41	15.1	5.88	13.88	1.51	4.37	0.77	1.96	97.76
Aug	17.19	14.78	0.87	12.86	67.10	290.2	5.9	12.5	0.47	0.45	15.1	6.31	13.83	1.72	4.59	0.78	1.93	101.98
Sept	17.69	15.25	0.8	12.2	63.34	290.7	7.3	12.1	0.60	0.53	14.5	7.03	13.92	2.18	4.85	1.21	1.98	108.85
Oct	17.99	15.55	0.69	10.73	63.88	290.9	8.2	11.8	0.69	0.59	13.2	7.02	13.96	2.24	4.78	1.92	2.02	114.98
Nov	18.03	15.58	0.61	9.5	69.78	291.0	8.7	11.5	0.76	0.63	11.7	6.56	13.98	2.49	4.07	2.55	2.02	110.56
Dec	18.06	15.61	0.56	8.74	83.20	291.1	8.2	11.4	0.72	0.61	10.8	5.95	14.0	2.56	3.39	3.2	2.02	99.81
Total																		**1416.2**

e_a = saturation vapor pressure (mmHg)
e_d = actual vapor pressure (mmHg)
RH = relative humidity (%)
U_2 = wind speed (mile/day)
n =daily mean bright sunshine hour (hr/day)
N =maximum possible sunshine hours determined by latitude and season (12° N latitude is for the study)from standard tables.
fa =a function of sun shine hour
σ =Stephan-Boltzman constant ($5.67 \times 10^{-8} \times Wm^{-2} T^{-4}$)
T =temperature (°c)
TK =temperature in Kelvin
Ro =out going solar radiation (mm/day)

Ra =solar radiation which depends on latitude and season (mm of water /day)
R_I =incoming solar radiation (mm/day)
r = albedo (reflection coefficient for incident radiation =0.05 for water)
σT^4 =theoretical black body radiation (mm/day)
Δ =slop of saturation vapor pressure plotted against temperature
γ =hygrometric constant (0.27mmHg/°c)
Eo =open water evaporation (mm/month)
Ea =energy for evaporation (mm/day)
H =available heat (mm/day)

Table 3.6 Mean Monthly Flow Data of Rib River

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept.	Oct .	Nov	Dec
1980	0.471	0.324	0.2	0.753	0.16	1.25	36.867	63.811	22.653	-	-	-
1981	0.831	0.619	0.412	0.366	0.692	-	-	73.926	31.874	5.989	2.73	1.747
1982	0.317	0.184	0.052	0.012	0.205	1.757	19.383	56.814	15.437	3.507	2.606	1.325
1983	0.007	0.162	0.651	-	0.198	6.238	21.774	34.528	17.667	0.943	0.081	0.033
1984	-	-	-	0.132	1.068	1.413	30.594	35.51	30.597	1.659	0.262	0.104
1985	0.232	0.112	0.027	0.043	0.004	8.713	54.586	82.68	50.7	7.709	1.568	0.937
1986	0.434	0.251	0.25	0.166	2.317	3.317	6.927	34.953	12.705	1.78	0.73	0.322
1987	0.99	0.572	0.495	0.506	0.639	4.782	26.931	68.057	22.126	6.911	2.673	1.806
1988	-	0.771	0.508	0.471	0.393	1.043	34.324	55.889	45.929	7.833	3.019	2.495
1989	-	0.699	0.508	1.301	-	-	-	-	70.417	6.227	2.032	1.149
1999	-	0.138	0.081	2.032	1.000	1.102	49.123	123.223	42.275	10.361	21.071	0.991
2000	-	0.541	0.389	1.613	4.189	4.807	41.08	54.382	44.819	10.456	2.341	0.62
2001	-	0.087	0.053	0.074	0.764	7.221	66.892	95.26	50.059	1.992	0.709	0.445
2002	-	0.243	0.231	0.66	0.565	1.42	-	71.473	29.139	1.838	1.032	0.828
2003	-	0.419	0.602	1.467	7.904	29.167	65.58	83.992	22.321	7.225	2.951	1.368
Annual mean	**0.410**	**0.341**	**0.269**	**0.640**	**0.660**	5.557	**28.152**	66.750	25.470	5.316	**1.521**	**1.012**

Source: Abay Master Plan, Ministry of Water resources, 1996

Table 3.7 Flow duration for Rib river based on 10 daily discharge data (1994 -2003)

S.No.	Percent time equaled or exceeded	Discharge (M m^3)	S.No.	Percent time equaled or exceeded	Discharge (M m^3)
1	2.78	76.8743	19	52.78	2.9449
2	5.56	73.4394	20	55.56	2.6548
3	8.33	66.7321	21	58.33	2.4408
4	11.11	66.5156	22	61.11	2.2270
5	13.89	48.5523	23	63.89	2.0655
6	16.67	46.3726	24	66.67	1.7902
7	19.44	28.7140	25	69.44	1.0725
8	22.22	27.2228	26	72.22	0.5730
9	25.00	17.2208	27	75.00	0.5701
10	27.78	15.1400	28	77.78	0.5275
11	30.56	10.4427	29	80.56	0.4441
12	33.33	10.3894	30	83.33	0.4283
13	36.11	9.32204	31	86.11	0.4261
14	38.89	6.59620	32	88.89	0.4225
15	41.67	5.15375	33	91.67	0.4062
16	44.44	4.31108	34	94.44	0.3497
17	47.22	3.72482	35	97.22	0.3492
18	50.00	3.32802	36	100.00	0.2965

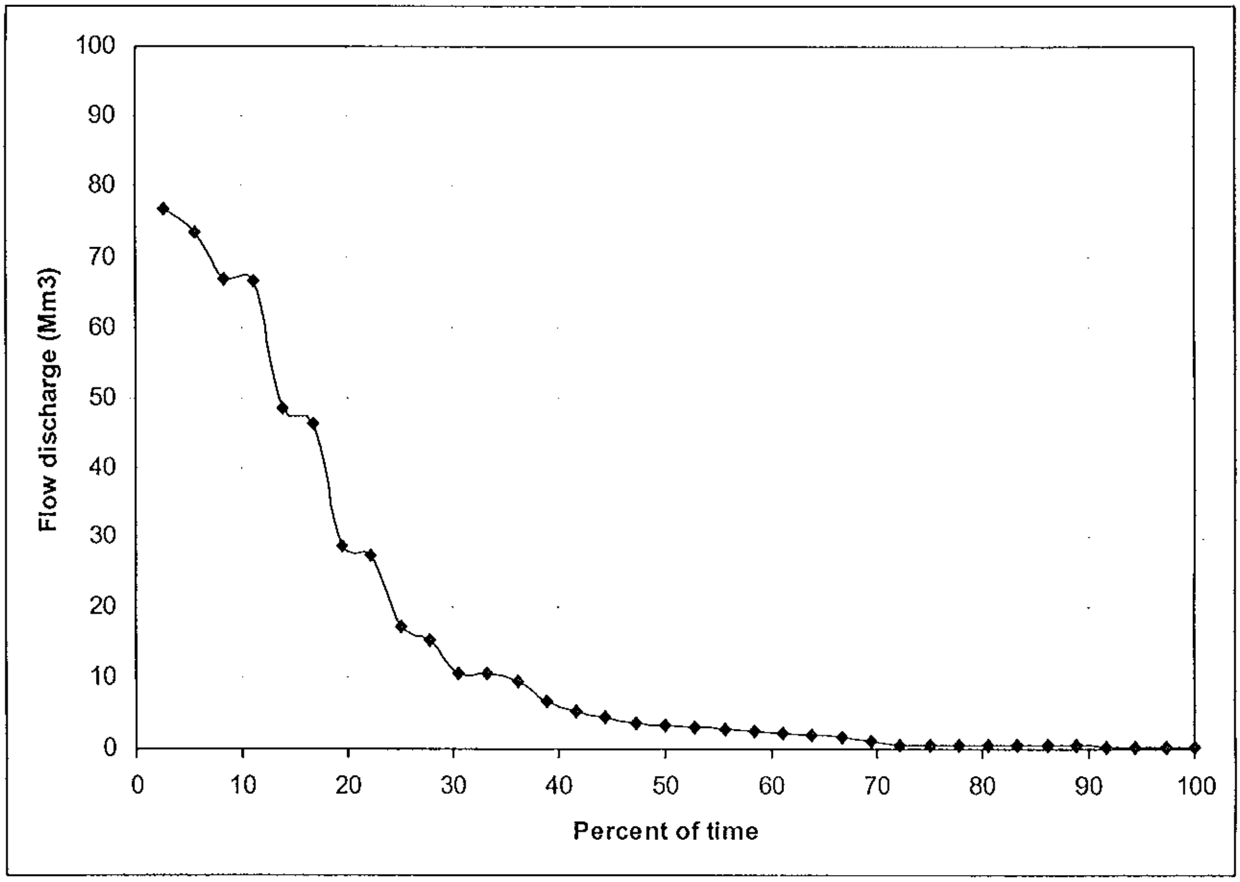


Figure 3.2 Flow duration curve for Rib River based on 10 daily observed discharge data (1994 -2003)

One or more of the above factors may dictate to keep the reservoir at a limited capacity. In addition to this, the capacity of the reservoir of a dam is directly influenced by the height of the dam.

It is only after deciding the dam height, the storage capacity of the reservoir can be computed. For the Rib dam project the maximum reservoir level is proposed to be kept at 1920 m, thus the maximum reservoir depth, at the dam axis, would be 50m. In order to calculate the storage capacity of the proposed reservoir topographic map at the scale of 1:20,000 was used. Further, digitization of elevation contours at 5m steps has been carried out using AutoCad software. Storage capacity was worked out using area elevation and capacity elevation curves. To workout the capacity elevation curve the datum elevation was set at 1870m above the river bed and the area was measured by AutoCAD. Thus, finally five contours at equal intervals of 5m above the datum were worked out. Table 3.8 and Figure 3.3 shows the computed storage capacity for the proposed reservoir.

Table: 3.8 Computed storage capacity for the proposed Rib dam reservoir

Parameters	Dimensions					
Consecutive Average contours Elevation(m)	1877.5	1887.5	1897.5	1907.5	1917.5	1920
Average area(m^2)	882231.4	2537852.5	4053933.9	5685802.65	8614255.55	19134826.37
Storage capacity(Mm^3)	4.411	12.6892	20.2697	28.4290	43.0713	95.67413185
Incremental Storage Capacity(Mm^3)	4.411	17.1002	37.3699	65.7989	108.8702	204.544

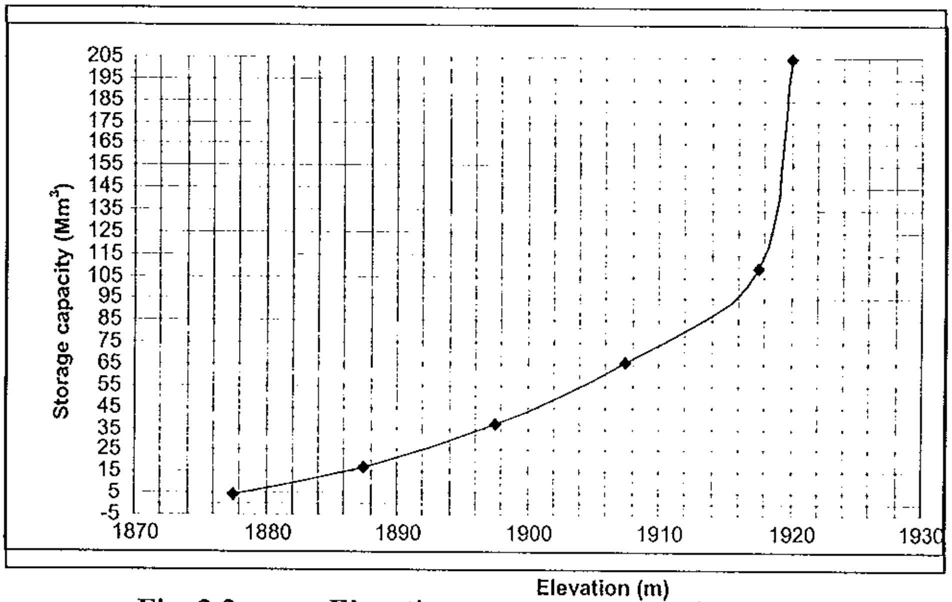


Fig. 3.3 Elevation vs. storage capacity curve

3.8 GROUND WATER CONDITION OF THE PROJECT SITE

The principal source of ground water is precipitation. The amount of water that infiltrates into the ground depends upon the dispersion of precipitation, i.e. on what proportions immediate runoff and evapo-transpiration takes place and what is left, it contributes to infiltration/percolation.

The major engineering significance of ground water conditions are listed below;

(i) The retention of water in the reservoir basin is determined by the piezometric level and ground water divide position with respect to the reservoir level.

(ii) The presence of ground water in rock mass causes water pressure within the discontinuities which results in reduction of cohesion and may cause problems of instability of reservoir rim slopes and the abutments.

(iii) Ground water may pose problems in construction.

(iv) Ground water may be an erosion agent that may damage the structure.

(v) Ground water fluctuation may cause uplift problem.

3.8.1 GROUND WATER LEVEL AND FLUCTUATION

The ground water level fluctuates in position, particularly in those climates where there are marked seasonal changes in rain fall. Thus, permanent and intermittent water tables can be distinguished, where the later is an expression of the fluctuation of ground water level.

The location and probable fluctuation are important for foundation work, deep excavations and water logged areas. The position of ground water table can be estimated through observations of open wells at the site or in the vicinity. The seasonal variation in ground water levels can be ascertained by piezometrers which can be installed suitably (Gopal Ranjan ,1993)

The ground water observations of the site has been made by the Project Authorities during the drilling operation and test pitting. The ground water level was observed by placing piezometres in drill holes and by direct observations in hand dug test pits, however, the data collected represents a short period of time and does not account for fluctuations within the seasons.

The ground water level recorded on each bore hole varies because of topographic location of the bore holes the maximum depth of ground water level is recorded in bore hole No.BH-1 which is located at the left abutment. The minimum depth is recorded in BH-2 which is located in the river bed. The depth to groundwater as observed in various drill holes and the test pits at dam site is shown in table 3.9.

Table 3.9 Depth of Groundwater as observed in bore holes and Test pits

S.No	Bore holes/Test pits	Depth of Ground water below ground surface (meters)
1	BH-1	40
2	BH-2	2.5
3	BH-3	39.3
4	DTPR-2	5.7
5	DTPR-3	8.2

The perusal of table 3.9 indicates that the ground water table is shallow, particularly in the dam foundation area therefore during trench excavation for the preparation of the dam foundation pumps will be required for dewatering.

3.8.2 CHEMISTRY OF THE RESERVOIR WATER

In engineering structures water is an agent of solution by virtue of its acidity and in soils it is an agent of piping in embankment by virtue of its total dissolved solids. Carbonic acid attacks concrete structures by stripping of Ca^{++} and Mg^{++} which are then carried off in solution and sulfuric acid attacks concrete by driving of the Ca^{++}. This phenomenon occurs in concrete dams however, in the case of Rib dam, the dam is an earth dam which is not susceptible to acidity of the water.

In earth dams the effect of water chemistry is on piping problem of the embankment. The dispersive nature of embankment soils depends on the exchangeable sodium percentage, however values of ESP causes serious piping when the ESP value of embankment soil is greater than 15 and the reservoir water has ESP value is lower than 0.5. The chemistry of the Rib river water is presented in table 3.10 (Molla, 2004). Thus, from table 3.10 ESP of the rib river water is calculated as; ESP = Na/ Na+K+Ca+Mg*100 =0.78/3.02=26% .

Table 3.10 Chemestry of Rib river water

Ions and Ctaion	meq/l
Na+ and K+	0.78
Ca++	1.32
Mg++	0.92
Cl	0.26
So4	0.10
Hco3	2.68

CHAPTER IV

GEOLOGICL SETTINGS

4.1 REGIONAL GEOLOGICAL SETTING

The visible geological make up of north western Ethiopia is predominantly composed of tertiary volcanic rocks. The Ethiopian plateau does not fit the popular image of a continental flood basalt province rather than consisting of a thick pile of monotonous, rapidly erupted theoloitic basalts. It is made up of several distinct volcanic centers of different ages and magmatic affinities (Kieffer et al., 2004). The recent studies have revealed the great complexity of the volcanic succession (Kazmin, 1962). In the northern part of Ethiopia based on their age, volcanic rocks can be classified into four units.

4.1.1 Ashangi Basalt

The Ashangi Formation represents the earliest fissural flood basalt volcanism on the North western plateau. The basalt flows are several hundreds of meters to a kilo meter thick of strongly weathered, crushed tilted basalts which lie below the major pre-Oligocene unconformity (Zanttin et al, 1980). The Ashangi Formation consists of predominantly mildy alkaline basalts with inter bedded pyroclastics and rare rhyolytes and is commonly injected by dolerite sills and dykes .The upper part of the Ashangi Formation is more tuffaceous and contains inter bedded lacustrine deposits lignite seems. It was believed that these early flood basalt flows are restricted to the North western plateau (Zanettin and Jusefin Visefine, 1975, Merla et al.1973,1979) until a group of early flood basalt were found in South western Ethiopia (Devidson, 1983) with K/Ar ages between 49 to 36 Ma. Among these the Akobo basalt (49 to 46 Ma) are here considered to be analogous with the basalts of the Ashangi Formation.

The age of the Ashangi Formation in the type area remains uncertain (Zanettin et al ,1980). The oldest reported age for a volcanic rock on the North western plateau is 54 Ma (Kazmin, 1979). The general consensus remains that the Ashangi Formation have Eocene to Oligocene age in the Northern western Ethiopia.

4.1.2 Aiba Basalt

The Aiba basalts represent the second major pulse of fissural basalt volcanism on the North western plateau. They are generally aphyric compact rock ,in place showing stratification and contain rare inter bedded basic tuffs .The Aiba basalt un conformably overlay the Ashangae formation and attain a thickness of 200 to 600 m .The basalts show a distinctive tholetic nature with transition to mildy alkaline varieties. The absolute age of the Aiba basalts ranges from 34 to 28 Ma placing the in Oligocene (Zanetin et al,1980: Kazmine,1979)

4.1.3 Alaji Formation

The Alaji Formation mainly consists of aphyric flood basalts associated with rhyolytes (ignimbrites) and subordinate trachytes. This formation ranges in age between 36 to 13 Ma (Kazmine, 1979, Zanettine et al, 1980). A migration of Alaji type volcanism from North to South is indicated by the occurrence of the older volcanics of this formation on the Northern part of the North western plateau.

Alaji formation makes the bulk of the volcanic succession on North western plateau. On the North-western plateau the Alaji Formation rests conformably on the Aiba basalts .The Alaji Formation contains basalts transitional to theoletic in nature and an increase in alkalinity is observed in the younger members of the formation. Thus the Miocene member of the Alaji Formation are more alkaline and are associated with sub alkaline acidic members.

4.1.4 Tarmaber Formations

Tarmaber Formation on the North western plateau represents Oligocene to Miocene basaltic shield volcanism. The central type Tarmaber Formation basaltic volcanism was followed by fissural eruptions, particularly along the escarpments of North-western and South-eastern plateaus.

Basalts of the Tarmaber Formation in contrast to the theoletic and mildy alkaline nature of the earlier flood basalts typically have an alkaline affinity on the North-western plateau the Tarmaber shield volcanoes become progressively younger from North to South. The classification of Tarmaber - Gussa Formation for the shield volcanoes of the Northern Ethiopia plateau with an absolute age of 26 to 16 Ma and the name Tarmaber - Megezez Formation for the younger shield volcanoes with an absolute age range from 16 to 13 Ma in the southern part of the North-western plateau and the South-eastern plateau has been widely used and the later is believed to mark the initiation of rifting of the main Ethiopian rift (Kazmine, 1979). The upper age limit of the Tarmaber –Megezez Formation is lowered to 7 to 3 Ma since the large basaltic center.

4.1.5 Quaternary Plateau Basalts

Quaternary alkaline basalts and trachytes were erupted along pre existing structures on the North- eastern and South-eastern plateau, although not dated their relatively unmodified geomorphologic features such as the prevalence of prominence cinder cones and small collapse creators particularly in a region of heavy rain fall and perennial streams indicate their recent age .Alkaline basalts and trachyte lavas prevail in the Tana graven belong to this unit. Field evidence suggests a Pleistocene age to this unit. Volcanic cones and flows of scoraceous basalts are well presented in the Tana lake graven .These quaternary basalt flows are characteristically alkaline and may represent the final pulse of basaltic volcanism on the Ethiopian plateau.

4.2 QUATERNARY SEDIMENTS

Quaternary sediments of different origin (fluvial ,lacustrine, and elluvial) are widely spread all over Ethiopia including the North western part

4.3 GEOLOGIC STRUCTURES

Geologic structures play an important role in any developmental activities. For the safe design, construction and functioning of Engineering structures, mainly dams geological structures may pose problems. Therefore, it becomes very important to identify such structures in the beginning of investigation so that accordingly proper remedial measures can be planned.

These geological structures are resulted because of deep tectonic activities, either regionally or locally, which can change the morphology of the earth through time. In the Rib basin different structures with different orientations have been identified. Plugs of trachyte and phonolite are common features of the basin which mark terminal parasitic activity on the flanks of the Gunna shield volcano (Jepson and Athearry, 1961).

From the areal photographs on 1:50,000 Scale, dominant minor structures can be seen which generally trends NW-SE and at rare occasions ENE-WSW. These structures are faults, plugs, topographic escarpments, and river flow lines.

Lake Tana has previously being indicated by different authors to be located in fault bounded grabens with the Gonder and Debre Tabor Grabens (Mohr and Roger, 1966). Over the eastern

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Lake Tana has previously being indicated by different authors to be located in fault bounded grabens with the Gonder and Debre Tabor Grabens (Mohr and Roger, 1966). Over the eastern side of the Tana basin alluvial cover partly masks the D/Tabor Graben. North of Debre Tabor E-W Striking faults with a morphologic expression indicating southerly down throws can be traced further east, where they turn to ESE-WNW before termination against the Gunna shield volcano. The faulting then reappears on the southern flanks of the shield. On the opposing, southern side of Debe Tabor graben a complementary set of E-W faults veers to the east-southeast farther East and is down thrown to the North (Chorowiz et al., 1998 as described in Mohr and Rogers, 1966).

4.4 GEOLOGY OF THE RIB BASIN

The geology of Ribb basin (sub basin of Lake Tana) is dominated by the huge volcano system named Gunna Terara shield volcano. It corresponds to the eruptive events that occurred during the early Miocene to Pliocene period and classified in the shield group basalt (BCEOM 1999, and Kieffer et al., 2004). The common litho type for this material is basalt with large amount of interbedded scoraceous lava, volcanic ash, and other acidic rocks such as rhyolite and trachyte with rare ignimbrite. Basalt agglomerates and palio-soils are also common. Figure 4.1 shows the Geological settings in the Rib basin.

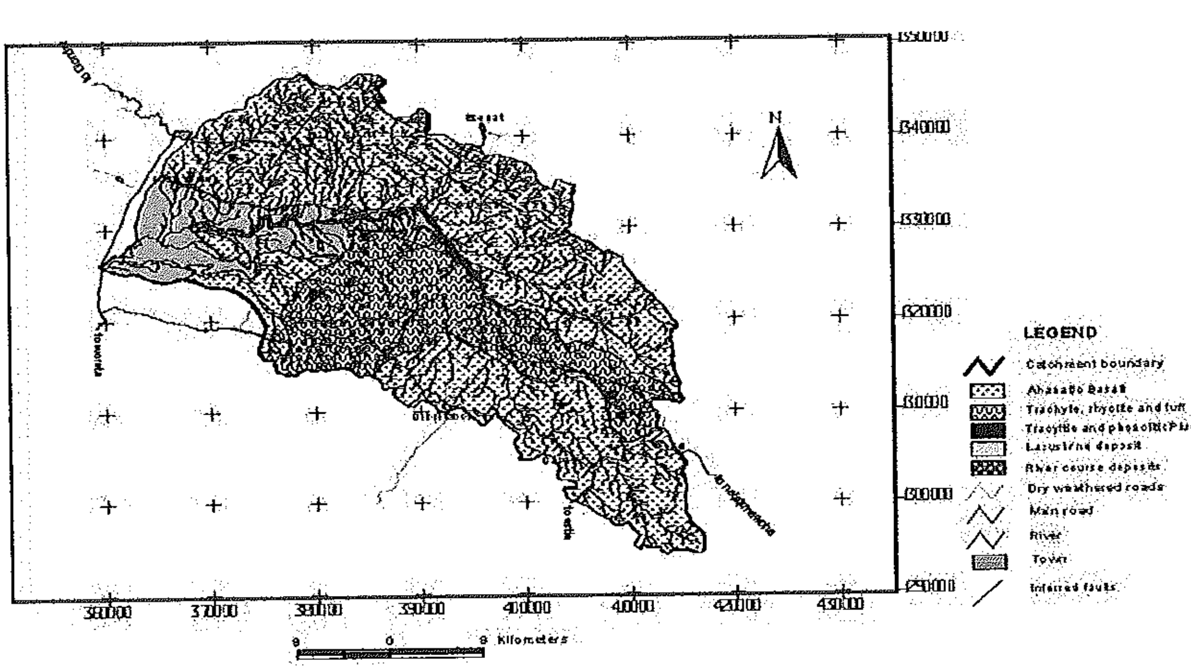


Fig. 4.1 Geological settings in the Rib basin

The other smaller volcanoes located northern part of basin are also considered having been active during the same geological period as Gunna volcano (BCEOM, 1999) .The lava flow from the northern source is toward south where as lava from Gunna in this basin is toward north. Here the river follows the area of confluence. This is clearly indicated by the sudden change of the river direction from S-N- to E-W. Probably this change is the result of blocking of the river by the hard basaltic flows from north side.

At the end of Ribb (near Lake Tana) the area is completely overlain by recent flood materials which are mainly covered by silt to clayey deposits. Recent volcanic flows belonging to the quaternary eruption events have been noted but they appear to be localized in the lower section of Ribb plain. No evidence of such flows has been mapped in the study area of the basin. With in Ribb basin plugs of trachyte and phonolites are common features which mark terminal parasitic activity on the flanks of the Gunna shield volcano (Jepson and Athearry, 1961) as cited in Chorowiz et al. (1998). The lower portion of the basin is totally covered by thick lacustrine deposits (effect of Lake Tana), alluvial deposits, and colluvial formations. The alluvial and colluvial deposits are the derivation of local geology through weathering and transportation by erosion.

4.4.1 Lithologic Description

In general, the geology of the area can be categorized in to 5 main formations. These units are

i) Lacustrine deposits
ii) Shield Group Basalt
iii) Rhyolite, trachyte and pyroclastic
iv) Phonolitic and trachytic plugs
v) River bed deposits

<u>**Lacustrine Deposit**</u>

The Lacustrine deposits of the area are found at the lower part of the basin which is mostly from an altitude of 1800-1870 m.a.s.l. Its source is related with the formation of Lake Tana. It is believed that in the early times Lake Tana covered large areas of present lacustrine deposits and with the regression of the Lake Tana the Lacustrine deposits become terrestrial. This unit is composed mainly, silt, silty sand and clay. Clay, which is present at the top, is the alluvial organic sediment result from the near by area. The depth of clay deposit, as can be seen from different borehole data in its periphery and middle part, is not less than 30 meter.

As borehole evidences show, at the end of the upstream part along Ribb River, its depth is 30 meters and near the periphery (near Woreta town), its depth reaches upto 60 meter. Also, in the northern part of the Ribb River about 3 km from the center of lake, one shallow well proved the presence of such formation with a minimal thickness of 70 meter. There fore, thickness of lacustrine decreases to ward the peripheral and towards the Ribb river.

Shield Group Basalt

Shield Group Basalt rock units are the dominant formation in the basin with degree of weathering that varies from place to place. The high land area around Debre Tabor is covered by highly jointed basalt with intercalation of ash. Jointing is not the result of weathering rather is the result of tensional segregation of mineral crystallization. It is highly jointed and used as dimension stone for buildings in and around Debre Tabor town. The shield Group basalts position is immediately below or at the foot of Mount Gunna elongated along the road Woreta - Woldya through Debre tabor.

The same group of basalt but highly weathered and friable basalt is common in the basin especially northern and North-eastern part of it. This formation covers more area of the basin than other formations. It is distributed on both sides of the river Ribb and is mostly situated on the hilly and ridge parts of the study area and is not common on flat lowland. This rock unit is sensitive to erosion due to its topographic situation and its degree of weathering.

Rhyolite, trachyte and pyroclastic

All these formations are the result of acidic volcanic eruption which is situated around the basaltic eruption center of Mount Gunna. In the study area South of Ribb river immediately below the jointed basalt boulder, these formations are present. Fresh rhyolite formation occurs in most of the area along Ribb and Meher River around their junction. Ignimbrite rock which is highly compacted tuff is also found around the rock hewn church known as Wukro with less extent in coverage. Volcanic ash and tuff (part of the pyroclastic) are common in between Ammed ber and Debre Tabor as exposed clearly along the road Woreta -Woldya. Trachyte formation with high degree of weathering are dominantly occupied the area of Alem saga forest near Ammed ber.

Trachyte and phonolitic plugs

With in the Rib basin plugs of different sizes are present. In origin these plugs are mainly composed of trachyte and phonolite. Trachyte plugs are present near or within the area occupied by the acidic rocks whereas; phonolitic plugs are common in the basaltic terrain. These plugs are dominantly made an alignment or form of lineation at the foot of the elevated ridges.

River bed deposits

In most of the river courses in the basin huge deposition of alluvial materials can be observed along the river banks, which show the intensity of erosion in basin. Rib river which starts from top of the Mount Gunna and flows through Fogera plain caries different sediment load (alluvial formation). Along the course of the river there are three types of river beds (the silty sand, the sandy gravel, and gravel and pebbles with boulders).

The lower part of the river coarse which covers most of the flat area is occupied with silty sand and silt. Next to this formation in upstream direction the sandy gravel type will take the dominancy and then followed by gravel, pebble, and boulders. These deposits are good construction material.

Seismicity in the area

The method of assessing likely earthquake intensity and frequency at a given site are complex, requiring reasonable judgment and collection of geological and seismic data. Due to this complexity, for structures with lesser magnitude, the tendency is to relay upon seismic risk maps. The maps are often published in national building codes which recommend the engineering 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 conditions, immediately beneath the site because thick soil deposits and outcropping bed rock do not have similar response for the same earthquake magnitude i.e. soils are more susceptible to liquefaction before rocks will be forced to collapse. According to Seismic risk map of Ethiopia 100 year return period, 0.99 probability by *Laike Mariam Asfaw, (1986).*
the country is divided into zones of approximately equal seismic risks based on the known distribution of past earthquakes.

According to R.B. Johnson (1988), these seismic intensity zones are related to the ground acceleration as follows;

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

Figure 4.2 indicates that the Rib dam site is located within the zone of no damage which implies that there is no risk from the seismic activity.

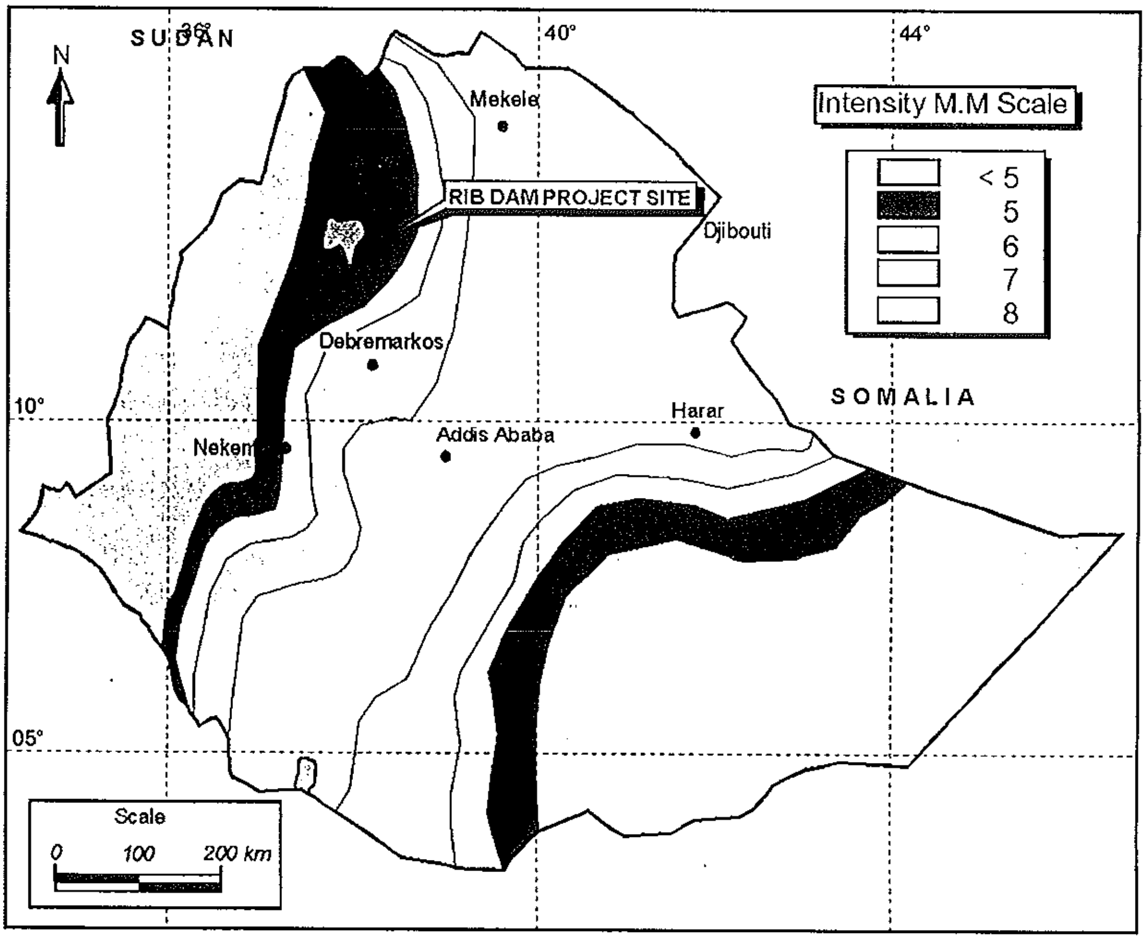


Fig. 4.2 **Seismic risk map of Ethiopia 100 year return period, 0.99 probability.** *(Ater Laike Mariam Asfaw, (1986).*

4.5 GEOLOGY OF THE DAM AND RESERVOIR SITE

Topographicaly, the area is characterized by broad and flat flood plains, old bench forming terraces and low to high relief basaltic hills with steep to moderately steep slopes, which are found bordering the reservoir area especially, at the dam site. The geology of the dam site and reservoir area is represented by tertiary trap series basalts consisting of two different basalt flows of the aphanetic basalt and the porphyritic basalt. The rock out cross are visible only at the sloping flanks of the river valley where as, the broad flat plain area (Fig 4.2 (a), (b))is covered by alluvial and colluvial soil deposits. The alluvial deposits is grouped into three units; i)River bed recent deposits, ii) Flood plain and iii) Terraces deposits.

Based on weathering grades the basaltic rock units are mapped as slightly to moderately weathered, very close jointing basalt and moderately to highly weathered widely jointed basalt. Generally the following soil and rock units are mapped in the reservoir and dam site area.

A/ Soil units : i) River bed recent deposit, ii) Flood plain deposit, iii) Terrace deposit and iv) Colluvial deposits.

RIVER BED RECENT DEPOSIT (Sla_1)

The river bed deposit covers the whole length of the river channel. The compositions consist of bounders, Cobbles and gravel component. The thickness of this deposit is thick (27.18m) as it was seen in the bore hole sunk at he rive bed. The river bed is wide both up stream and down stream of the dam site having an average width of 50 m.

FLOOD PLAIN DEPOSIT (Sal_2)

Mostly the broad concave meanders of the river are made up of flood plain clayey silt deposit which is localized and found invariable size and thickness.

TERRACE DEPOSIT (Sal_3)

On the basis of morphology and stratigraphic position, two terrace levels can be identified both at the dam site and reservoir area. The first level occupies the upstream part of the reservoir area on the right flank and this terrace deposited is of invariable thickness. The average thickness observed is 6m. The second level of terrace deposit occupies the gentle eastern and western slopes of the dam site. As it was indicated by the test pit $DTPR_3$ dug at eastern side of the dam site this terrace deposit has a thickness of 8.20 m, whereas, on the western part of the dam site the terrace has a thickness of 5.7 m. The terrace depots consists of three distinct layers, the top layer consists of black silty clay; the middle layer consists of sandy gravel layer and the lowest layer consist of sandy silt layer having depths 2-6m, 1-2m, and 2-4m in average, accordingly.

Colluvial soil (Sco)

This soil is found at the foot slopes of basaltic hills, composed of clayey sand with angular basaltic rock fragments. This unit is found at the eastern part of dam site at the foot of the hill. However, the thickness of this soil is shallow (<lm).

<u>B/ Rock Units</u>: i) Slightly to moderately weathered aphanetic basalt and ii) Moderately to highly Weathered porphyritic basalt.

<u>SLIGHTLY TO MODERATELY WEATHERED APHANITIC BASALT</u>:

This unit is found covering most of the investigated area occupying higher topography. Morphologically, it forms low relief hills at the valley slopes and flat topography at the western extreme of the dam site. At dam site it is mantled by shallow soil cover and this unit covers both dam abutments and reservoir slopes. As observed in the field the upper part of this basalt is blocky, reddish brown and oxidized.

Three sets of joints are characterizing this unit, the preferred orientation of the major joint sets is; Joint set - 1 =N35^0 W/ 30^0 , Joint set - 2 =N47 ^{0}E/ 65^0 and Joint set - 3 =S72 ^{0}E/ 75^0 . Generally, these joints are planar and smooth with clay filling.

<u>MODERATELY TO HIGHLY WEATHERED BASALT</u>:

This unit is found in the upstream eastern part of the river and at the junction of Rib and Hamus Wonth Rivers, where it forms relatively higher topography and at the river bed sections it overlies aphanetic basalt. The appearance of this unit is blocky, light brown to gray in color, porphyritic texture composed of phenocrysts of probable plagioclases. Fig. 4.3 shows the Geological map of the reservoir and dam site.

4.6 GEOLOGICAL STRUCTURES

In the study area no major geological structures, such as faults and shear zones, were identified. However, well defined three sets of joints and fractures have been observed which are dominantly exposed on the rock out crops at the dam abutment slopes and in the eastern upstream periphery of reservoir. The characteristic of these joint sets is dealt in detail at chapter 5.

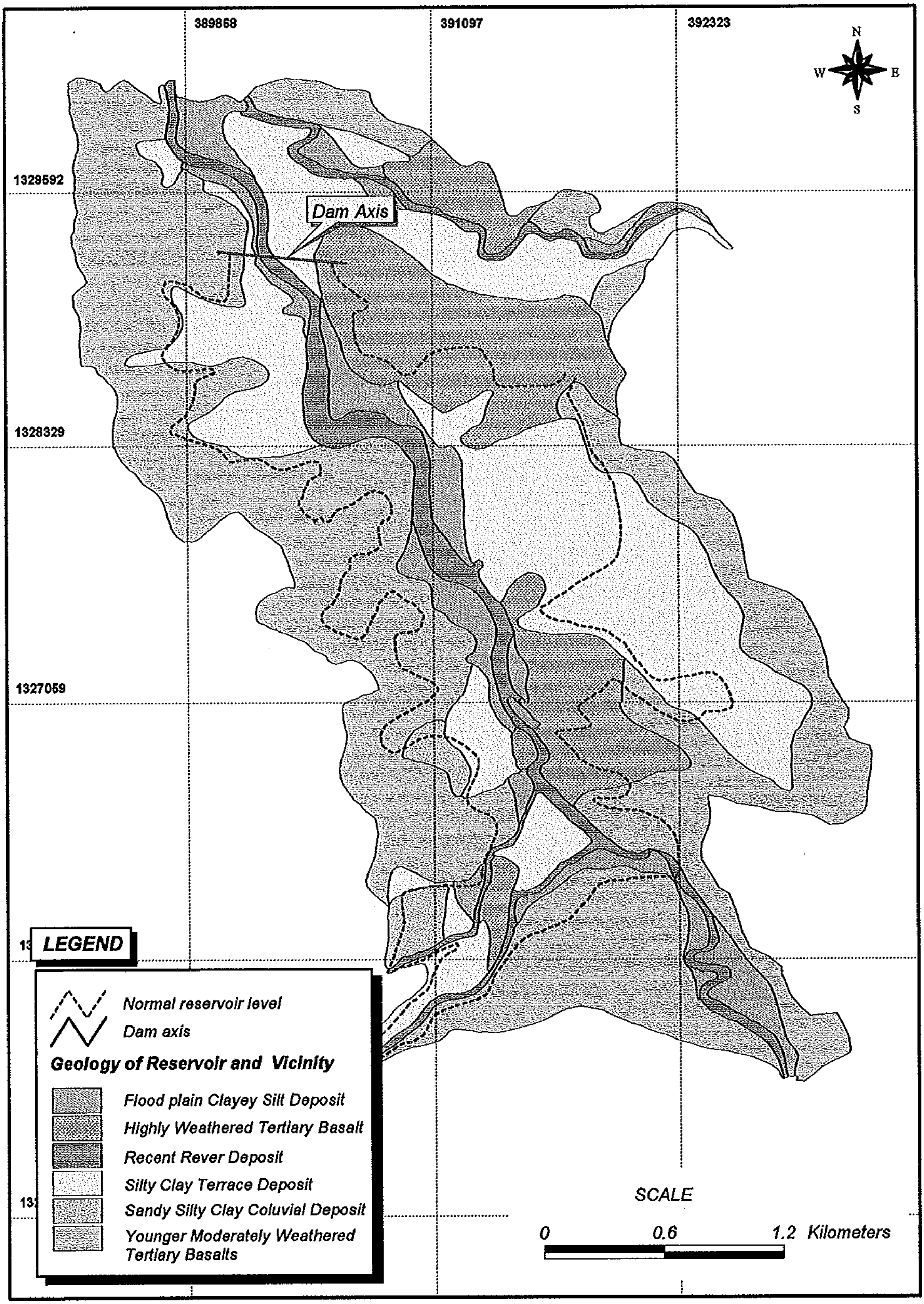


Fig. 4.3 Geology of Reservoir and surrounding Areas.

CHAPTER V

ENGINEERING GEOLOGICAL APPRAISAL

5.1 ENGINEERING CLASSIFICATION OF SOILS OF THE STUDY AREA

Engineering soils are subdivided into two main groups as a function of their predominant sizes and associated plasticity. The coarse grained soils are composed of sand size and larger size particles. They are separated into size ranges by sieving of materials up to cobble size. Except for minor fractions of plastic fines they characteristically are non-plastic. The fine-grained soils consist predominantly of silt and clay sized particles with differing degree of plasticity measured by Atterberg limits, rather than by sieving and settling velocity methods (Jonson,1988).

The most widely used classification schemes are those that divide soils into an orderly, easily remembered system of groups, or classes, that have similar physical and engineering properties and that can be identified by simple and in expensive tests. These groups ideally provide estimates of both the engineering characteristics and performance of soils for design and construction engineers. The description of soils within the group of a given classification typically are represented by alphabetical and alphanumeric symbols for rapid identification in written material, graphic boring logs, and on engineering drawings. The continued use of a few engineering soil classifications is the result of the adaptability of the classification to the variety of soils encountered on engineering practice (Jonson,1988). Even though there are many classification systems the unified soil classification system (USCS) is used in this study since this system is the most popular for use in all types of engineering problems involving soils.

5.2 PHYSICAL PROPERTIES OF SOILS OF THE STUDY AREA

5.2.1. INDEX PROPERTIES

The tests required for the determination of engineering properties are generally elaborate and time consuming. Some times index properties are used to have some rough estimation of the 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 Atterburg Limits and the consistency. (Arora, 2001)

Specific Gravity

The specific gravity of the soil is an important parameter for determination of other properties of soil such as void ratio, degree of saturation, consolidation etc. The specific gravity of the soils of the present study area falls in the general range of 2.35 - 2.76. The smaller values of specific gravity are for the coarse grained soils, sampled from the river terrace deposit in the reservoir area. The average specific gravity of the soils of the study area is presented in the table 5.1.

Table 5.1 Average specific gravity of the soils of the study area

Location	Soil type	Specific gravity
Dam site	Sandy silty clay	2.49
Reservoir area	Silty clay	2.52
Borrow area	Clayey silt	2.54

Moisture contents

The water content of a soil is defined as the ratio of the mass of water to the mass of solids. The moisture content of a soil is an important property because of the characteristics of soils especially, fine-grained soils change to a marked degree with variation of its moisture content. The water content of some of the fine grained soils may be even more than 100% of the total which indicates that more than 50% of the total mass is that of water (Earth Manual, 1998).

In the present study the moisture content is determined for borrow area soils both for the remolded and field density condition. The field moisture content was determined using core cutter method to evaluate the quantity of borrow soils whereas, the moisture content of the remolded soil is determined at optimum moisture content (OMC).

In the present study, the calculated average field moisture content of the soils of the borrow area is found to be 27.17%. Whereas, the average moisture content at maximum proctor compaction dry density or optimum moisture content is 18.8% (Table 5.2). The field moisture content is higher than the compaction density, this was because of rainfall during sampling. This shows that seasonal ground water fluctuation and rainfall variations can result in moisture content variation. The moisture content variation has direct effect on strength, compaction and shrinkage properties.

Table 5.2 Compaction and field moisture contents

Test pit No	Location (UTM)			Field moisture	Optimum moisture
	Location	Northing	Easting		
BA_1	Borrow area	1333150	390250	27.23	17%
BA_2		1333230	390400	28.35	23%
BA_3		1333330	390460	25.74	24%
BA_4		1333420	390510	27.37	21%

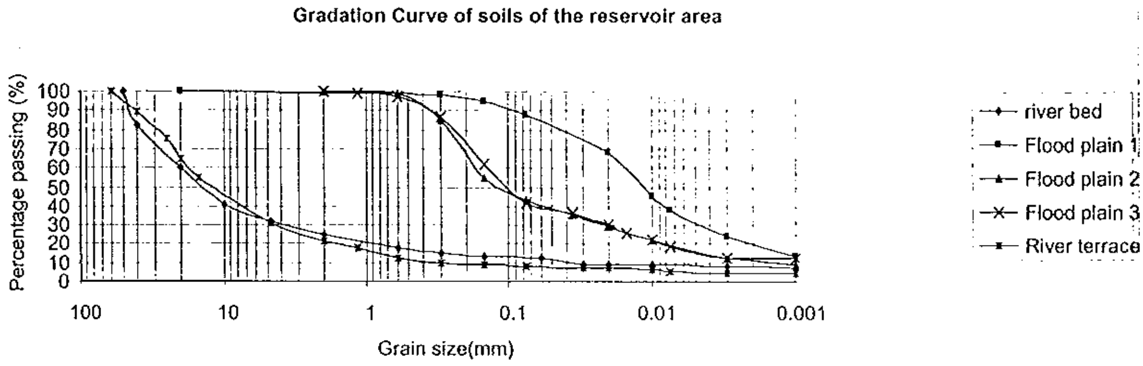
Grain size analysis

It is determined by the gradation analysis of soils and is presented in the form of a cumulative grain size curve in which particle sizes are plotted to a logarithmic scale with respect to percentage by weight of the total sample plotted to a linear scale. The sizes at 10%, 30% and 60% (D_{10}, D_{30}, D_{60}) are used in defining the gradation characteristics of the soil. Other sizes D_{15}, D_{50} and D_{85} are also used for filter design. The permeability and the angle of internal friction of soils is mostly proportional to the grain size of the soil. However, utilizing gradation in determining engineering properties is limited to coarse-grained soils (USBR, 1987).

For the present study, based on ASTM standard, Grain size analysis has been carried out for representative samples, taken from the soils of recent alluvial deposit, borrow area, dam site and reservoir area. The results thus, obtained are presented as an average gradation curve for the study area Fig. 5.1.

Atterberg Limits

Water, as a component of a soil, plays an important role in molding its physical behavior. Specially, properties of fine-grained soils are affected by the amount of moisture present in them. A given clay suspension can pass from a liquid state to that of a plastic material and to a semisolid state as the amount of moisture content in it is reduced. Through a series of tests proposed by Atterberg in 1991, it is possible to demarcate the boundaries of the liquid, plastic and semi solid state of the soils. The demarcation is in the form of numerical limits on the moisture contents present in the soil in each state when remolded. These numerical limits are not same for all the soils, but vary from soil to soil depending upon their physical properties, in the remolded state. So, determination of these numerical limits on moisture contents furnishes data to predict the physical behavior of a soil. Therefore, these limits are important index properties of a soil.



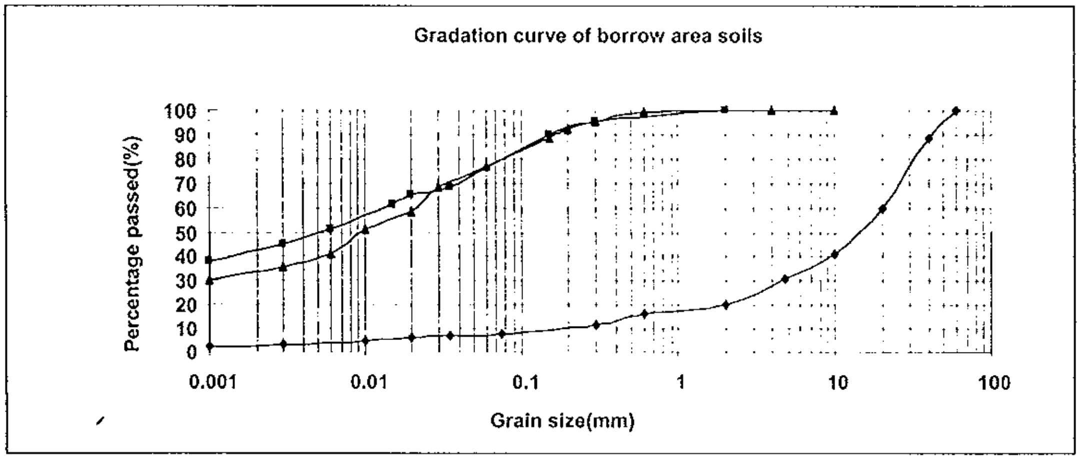


Fig 5.1 Gradation curve for reservoir and borrow area soils

In order to determine the Atterberg limits of the fine grained soils of the study area disturbed samples has been collected from the borrow area, dam site and reservoir area. During the present study 4 samples were collected and tested at Department of Civil Engineering Laboratory, Bahir Dar University. In addition to these, test data for Atterberg limits for 13 samples, were procured from Ministry of Water Resources. The results of Atterberg limits tests of the representative samples is presented in the table 5.3. **Fig5.2** shows the test pits locations from where the sample for Atterberg limit tests were collected.

Swelling Potential (S_P)

The swelling potential of an expansive soil is defined as the percentage swell of a laterally confined soil sample when tested in a consolidometer test, when soaked under a surcharge load of 7 kN/m^2, after being compacted to maximum dry density at optimum moisture content according to AASHO compaction test. The swelling characteristic of a soil, besides being exhibited by its swelling potential, can also be indicated by the plasticity index and shrinkage limit (Ranjan, 1991). Table 5.4 shows the Swelling potential of soils determined from plasticity index.

Table 5.3 Atterberg limit test results of the soils of the study area.

Sample No	Location	Depth (m)	Atterberg limits			Unified soil classification
			LL	PL	IP	
DTPR2-S1	Dam site	1.4 -1.6	67	31.29	35.71	CH
DTPR2-S2	Dam site	2.9 -3.0	64	34.14	29.86	CH
DTPR2-S3	Dam site	5.4 -5.6	63.4	31.8	31.6	CH
DTPR3-S1	Dam site	0.9 -1.0	81.3	32.39	48.91	CH
DTPR3-S2	Dam site	4.5 -4.6	41.4	30	11.4	CI
DTPR3-S3	Dam site	7.3 -7.4	66.8	21.46	45.34	CH
MTPR2-S1	Borrow Area	08 -1.0	34.0	22.96	11.04	CL
MTPR3-S1	Borrow Area	3.3 -3.6	49.3	17.9	31.4	CI
MTPR4-S1	Borrow Area	5.0 -5.3	47.5	26.17	21.33	CI
MTPR4-S2	Borrow Area	0.0 -1.0	57.6	26.6	31.	CH
FTPR1-S1	Reservoir Area	1.0 -1.2	49	32.8	16.2	CI
FTPR1-S2	Reservoir Area	2.0 -2.2	25.96	9.54	2.53	CL
FTPR2-S1	Reservoir Area	0.9 -1.0	71.3	32.34	38.96	CH
TPR1-S1	Reservoir Area	1.0 -1.5	62.1	32.02	30.08	CH
TPR2-S2	Reservoir Area	4.8 -5.0	87.9	33.18	54.72	CH

LL – Liquid limit , PL – Plastic limit , IP-Plasticity index , CI-Clay of intermediate plasticity
CH-Clay of high plasticity ,CL- Clay of low plasticity

Based on the results Plasticity chart (USC) has been prepared for dam site and borrow area soils which are presented as Fig. 5.2 and Fig.5.3, respectively.

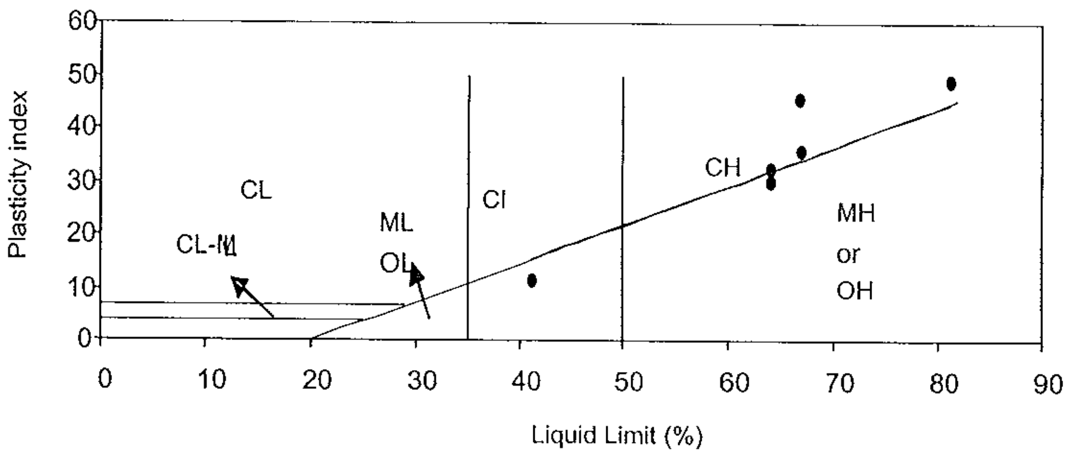


Fig. 5.2 Plasticity Chart for soils at Dam Site

Seed, Woodweard and Lundren (1962) has empirically connected the swelling potential (Sp) with the plasticity index (Ip) by the equation:

$$Sp = 60\ K\ Ip^{2.44}$$

Where; K is constant which is given as 3.6×10^{-5} for soils having clay content between 8 and 65%.

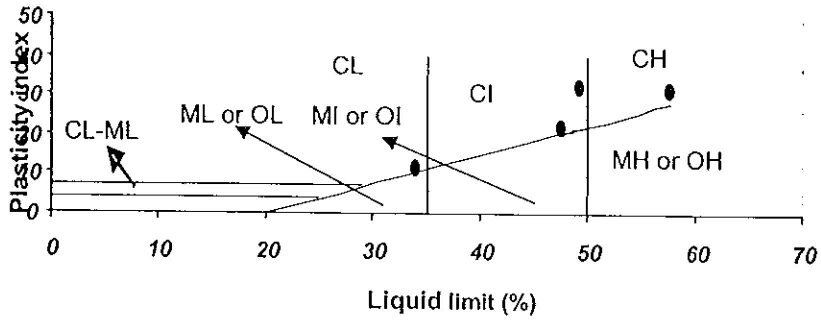


Fig. 5.3 Plasticity Chart for soils at Borrow Area

Table 5.4 Swelling potential of soils determined from plasticity index.

Soil sample	Location	Ip	Sp	Expansivity
DTPR2-S1	Dam site	35.71	13.28	High
DTPR2-S2	Dam site	29.86	8.58	High
DTPR2-S3	Dam site	31.6	9.85	High
DTPR3-S1	Dam site	48.91	28.61	Very high
DTPR3-S2	Dam site	11.4	0.819	Low
DTPR3-S3	Dam site	45.34	23.78	High
MTPR2-S1	Borrow Area	11.04	0.757	Low
MTPR3-S1	Borrow Area	31.4	9.704	High
MTPR4-S1	Borrow Area	21.33	3.777	Medium
MTPR4-S2	Borrow Area	31.	9.405	High
FTPR1-S1	Reservoir Area	16.2	1.930	Medium
FTPR1-S2	Reservoir Area	2.53	0.020	Low
FTPR2-S1	Reservoir Area	38.96	16.42	High
TPR1-S1	Reservoir Area	30.08	8.7387972	High
TPR2-S2	Reservoir Area	54.72	37.6296525	Very high

SP	*Expansivity*	*SP*	*Expansivity*
<1.5	- Low	5 – 25	- High
1.5 – 5	- Medium	>25	- Very High

Activity of the soils

It has been observed that the behavior of clays is influenced by the amount of clay fraction and the plasticity index. Skempton (1953) introduced the concept of activity (Ranjan, 1991). Activity is defined as the ratio of the plasticity index to the percentage by weight of soil particles of diameter smaller than two microns present in the soil. Clays can be grouped into three qualitative categories depending upon their activity (Table 5.5).

Table 5.5 Activity classification of soils.

Activity	Classification
<0.75	Inactive
0.75-1.25	Normal
>1.25	Active

For the present study soil samples were collected from the test pits located at the dam site, borrow area and reservoir area. In total 13 soil samples were collected and analysed for plasticity index and grain size distribution. Further, based on the plasticity index and clay proportions the activity of the soils were calculated . The results are presented in table 5.6.

$$Ac = \frac{\text{Plasticity index (Ip)}}{\text{\% finer than 2micron}} \quad \ldots\ldots\ldots\ldots\ldots\ldots 5.1$$

Table 5.6 Soil classification according to activity number 'Ac' (eq.5.1).

Soil type	Clay Proportion (%)	Sampling Location	Location (UTM)		Ip	Ac	Activity classification
			Easting	Northing			
Clayey sandy gravel	4	Borrow area	390250	1333100	11.04	2.76	Active
Sandy silty clay	43	Borrow area	390330	1333180	31.4	0.730233	In active
Sandy clayey silt	34	Borrow area	390430	1333210	21.33	0.627353	Inactive
Clayey silty sand	22	Borrow area	390510	1333290	31.0	1.409091	active

The activity of clay signifies the swelling characteristics of clays. A reasonable correlation exists between the type of mineral and its activity. It is obvious that the swelling behavior of a soil would depend largely on the type of clay minerals that are present in these soils and the proportion in which they are present. Table 5.7 shows the activity with respect to minerals contained in soil.

Table 5.7 Activity of minerals.

Minerals	Activity
Na- montimorillonite	4 -7
Ca- montimorillonite	1-5
Illite	0.5-1.5
Kaolinite	0.3-0.5
Halloysite	0.1
Calcite	0.2
Quartze	0

5.2.2 ENGINEERING PROPERTIES

. The principal distinction between index and engineering properties is that the determination of index property is simple and may be accomplished by personal with comparatively little training. Whereas, the determination of engineering properties requires considerable knowledge and skill. On the other hand considerably more special knowledge and skills are required to interpret and utilize index information than are required in using information from tests obtained for engineering properties (USBR.1987). The engineering properties of soils are compressibility, shear strength and permeability.

In the evaluation of a soil the transition is gradual between those properties which serve only as a broad quide to the character of the material and the quantitative properties, which define specific performance characteristics.

Compressibility

Construction often involves the use of soil to make a structure or the placement of structure made of other materials on a soil foundation. In either case, the compressibility of the soil used is an important consideration. Compressibility is the decrease in volume of a soil mass as a consequence of either natural or artificial means. This volume change is primarily due to change in volume of voids. To a lesser extent, it can result from a change in the volume of soils. Consolidation is the form of compressibility that occurs under a static load. Consolidation is basically the process of deriving water form the voids in a soil mass. Compaction is artificial densification of soil. It is a consideration when soil is used as a construction material. Both, consolidation and compaction are the forms of compressibility. (Jonson, 1988).

Consolidation

Consolidation occurs when a soil is placed under a static load. It is a consideration whenever a structure is founded on a soil. Two factors are of prime concern in addressing consolidation; the first is the total consolidation expected which is the amount of vertical displacement or settlement that occurs between the start and finish of consolidation and the other is the amount of time required for the total settlement to occur. Since consolidation data is required only for foundations therefore, for the present study, consolidation test has been conducted on the foundation soil sampled on representative soil stratum and expected to have maximum compressibility property. The sample was taken on the silty clay river terrace deposit at a depth of 7.30-7.6m. The test is conducted by using one-dimensional odometer devised by Terzaghi according to ASTM standard procedures. The main objective of this test is to obtain necessary information on the compressibility properties and rate of settlement of the dam foundation soil mass. The analysis result of the laboratory data is presented in table 5.8 and Fig. 5.4.

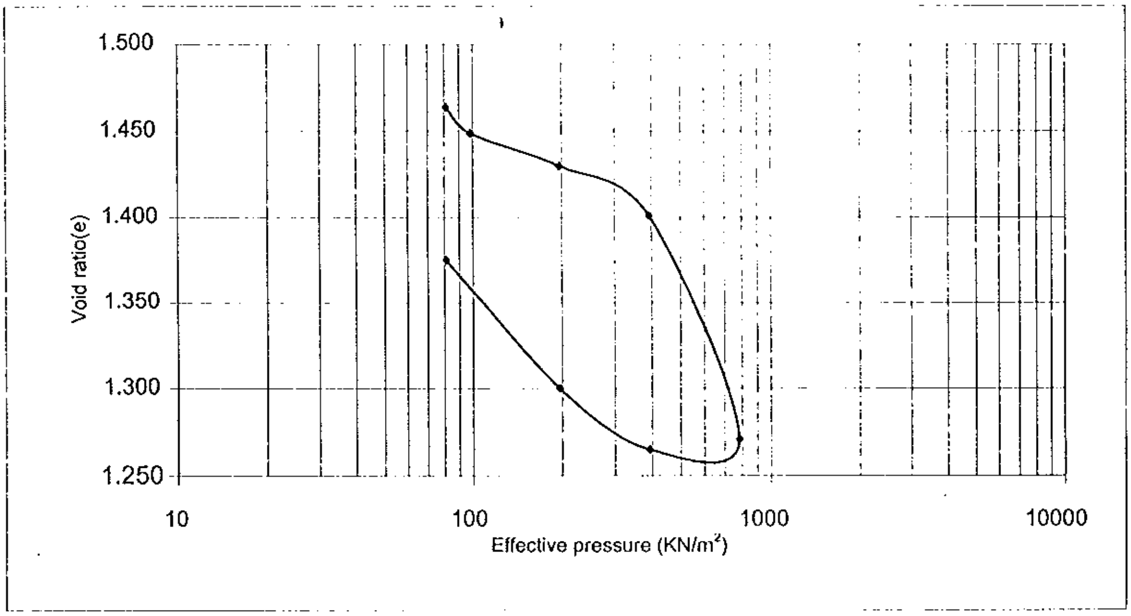


Fig. 5.4 Graph showing consolidation for foundation soil

Table 5.8 Compressibility properties of the soil.

Pressure range (δ) kn/m^2	e	Mv (m2/MN)	Cc	Cv (m2/year)	Av (m2/KN)
81	1.434	-	-	-	-
81-98	1.449	0.363	0.181	10.31	0.00088
98-196	1.430	0.080	0.063	5.72	0.00019
196-392	1.401	0.062	0.094	3.59	0.00015
392-787	1.271	0.143	0.429	3.10	0.00033

Where; Av is coefficient of compressibility defined by;

$$Av = \frac{-\Delta e}{\Delta \delta} \quad \text{.......... 5.2}$$

Mv is coefficient of volume change defined by;

$$Mv = \frac{A_v}{1+e_o} \quad \text{.............5.3}$$

Cc is compression index defined by the slope of the linear portion of the void ratio versus log δ plot defined by

$$Cc = \frac{-\Delta e}{\log_{10}(\delta_f/\delta_o)} \quad \text{........... 5.4}$$

Cv is coefficient of consolidation calculated from;

$$Cv = \frac{K}{M_v \gamma_w} \text{ or } \frac{K}{Cc\,\gamma_w} \quad \text{............5.5}$$

Where: K is coefficients of permeability, γ_w is unit Wight of water

Total consolidation settlement (S):

$$S = Mv\ \Delta\delta H \quad \text{........... 5.6}$$

Where; H is thickness of the stratum, $\Delta\delta$ is the additional pressure brought about by the structure.

The sample for the consolidation test was taken from a depth of 6.5 to 8.2 m from the soil strata at dam axis, near right valley bank. Hence, the value of H used in the computation of total settlement is 1.7m. The results of the total consolidation settlement analysis for each pressure change is presented in the table 5.9

Table5.9 Total consolidation settlements from one dimensional consolidation test.

Pressure change ($\Delta\delta$) kn/m^2	Mv (m^2/Mn)	H (m)	Settlement (cm)
17	0.363	1.7	1.05

98	0.080	1.7	1.33
196	0.062	1.7	2.07
392	0.146	1.7	9.73

Compaction

Compaction is achieved as soil particles become reoriented to a configuration that contains fewer voids. This reduction in void may also include fracturing of grains and bending or distortion of individual particles. The principal purposes for compaction are to increase the strength of the soil and to reduce permeability. Therefore, it is consideration whenever soil is used as a construction material. Moisture content and compaction effort affects the compaction of a soil mass. Besides, method of compaction and type of soil also affect the compaction. For a particular compaction effort, there is only one moisture content equivalent to its maximum density. The moisture content higher than the optimum moisture content creates excess pore pressures in the water filling the voids which facilitate shearing of the soil mass. In this study the compaction requirement is limited to the soils of the borrow area as these soils would be utilized for the dam embankment construction materials.

The compaction properties of these soils are determined in the laboratory by standard proctor test and modified proctor tests by procedures of ASTM standards. The modified compaction test was conducted at the Department of Civil Engineering, Bahir Dar University whereas, the standard proctor test was carried out in the laboratory of Building Design Enterprise, Addis Ababa. The results of the tests are presented in table 5.10.

Table 5.10 Compaction test results.

Soil type	Test type	M.D.D (kg/cc)	O.M.C (%)
Sandy, silty clay	Procter compaction	20.2	14
Sandy clayey silt	Procter compaction	15.4	23
Silty clayey sand	Procter compaction	15.5	21
Clayey silty sand	Procter compaction	14.9	17
Silty clay	Modified compaction	1.58	26.73
Sandy silty clay	Modified compaction	1.82	16.18

Shear Strength

Shear strengths of soil is the capacity of the soil to resist shearing stress. It can be defined as the maximum value of shear stress on any plane or surface, a point failure will occur in the soil because of movement of a portion of the soil mass along that particular plane or surface. Shear strength or shearing resistance of a soil is the property of the soil that enables the soil mass to keep its equilibrium when its surface is not level under any loading situation, which produces shearing stresses.

The shear strength is the most important engineering property for the problems related to foundations of structures, stability of earthwork engineering etc. However, the determination of shear strength is not simple. The shear strength of a soil is related not merely to the material it self but, It is also a function of the magnitude as well as the manner of application of the loads. Besides, the previous stress history of natural soil deposit is also very significant factor in influencing its shear strength.

The shear strength parameters can be measured in the undisturbed or disturbed remolded state by any of the following tests.

i) Direct shear test
ii) Un confined compression test
iii) Triaxial compression test
iv) Vane shear test

The shear test must be conducted under appropriate drainage conditions that simulate the actual field problem if economy and material facility allows. Depending upon the drainage conditions three types of tests are explained below.

- Unconsolidated -undrained condition
- Consolidated -undrained condition
- Consolidated -drained condition

The shear strength parameters (C and ϕ) depend upon a number of factors, such as the water content, drainage conditions and conditions of testing. Terzaghi established that the normal stresses, which control the shear strength of a soil, are the effective stresses and not the total stresses. In terms of effective stresses the shear strengths (S) of a soil at a point on a particular plane is written as:

$$S = C' + \delta' \tan\phi' \quad \text{................ 5.7}$$

Where;

C' and ϕ' are cohesion intercept and angle of shearing resistance in terms of effective stresses.

Even though there are different types of tests and drainage conditions each method of testing has merits and demerits to measure the shear strength of soils. For the present study, the direct shear test has been utilized. By the use of direct shear test the shear strength parameters were determined for soils on borrow area in the disturbed remolded state. For the dam foundation area undisturbed sample is required to determine shear strength parameters. However, testing undisturbed samples, representing field conditions, is not simple since, the test result is affected by moisture loss and disturbance during transportation form the site to the laboratory. For this reason the shear strength parameters are taken from soil classification average value as a preliminary design data (USBR, 1987). Shear strength parameters of soils of the borrow area, as obtained from laboratory direct shear test are presented in table 5.11

Table5.11 Shear strength parameters of soils samples collected from borrow area.

Soil type	M.D.D (kg/cc)	O.M.C (%)	Cohesion (C) KN/m^2	Angle of shearing resistant (ϕ) (degree)
Clayey sandy gravel	20.2	14	4	39
Sandy silty clay	15.4	23	22	31
Sandy clayey silt	15.5	21	38	22
Clayey silty sand	14.9	17	36	25
Silty clay	1.58	26.73	33	9
Sandy silty clay	1.82	16.18	10	30
M.D.D. – Maximum Dry Density, O.M.C – Optimum Moisture Content				

The shear strength parameter of the dam foundation soil as determined from the unified soil classification is presented in the table 5.12

Table5.12 Shear strength parameters of dam foundation soil

Unified soil classification	PARAMETRES	
	Cohesion (C) KN/m2	**(ϕ) (degree)**
CI	**4-38**	**9-39**
CH	**71-79.3**	**17-25**

Permeability

The voids in a soil mass provides not only the mechanism for compressibility but also passage by which water may move through the soil mass. Such passages are variable in size and the passes of flow are tortuous and interconnected. If sufficiently large number of such paths are considered as acting together an average rate of flow for the soil mass can be determined under controlled conditions that will be representative of larger mass of the same soil under similar conditions, the measure of state of water movement is permeability (USBR, 1987)

In this study permeability of soils is considered for the soils of the reservoir and dam site at insitu condition and for the soils of borrow area at maximum proctor compaction density.

Permeability of Soils of Reservoir Area

As it has already been discussed in the geology of the reservoir area (Chapter 2), the reservoir basin is predominantly covered by river terrace deposit consisting of silty clay soil with variable thickness ranging from 6.0 m to 8.2 m. In order to determine the hydraulic nature of the reservoir basin insitu tests has been conducted on auger holes by using gravity permeability test methods.

Four auger holes has been sunk on representative soils of the reservoir area above the water table and the permeability test was conducted on each auger hole. The method of testing is adopted from the U.S Department of the interior water and Resources Service. The method uses the relation:

$$K = \frac{Q}{C_u RH} \qquad \text{...........} \ 5.9$$

The required data for computing the permeability includes:

Radius of auger hole(R), Depth of hole (D), Depth of water in the hole (H), Length of test section (A), Percent of unsaturated stratum(X), Distance from water surface in the hole to water table (Tu), Condactivity coefficient (Cu) which is obtained from the standard graph and Rate of flow in to the auger hole (Q).

The auger hole permeability tests results obtained from each auger hole are presented in the table 5.13

Table 5.13 Permeability results as obtained from different auger hole test.

Parameters	Auger holes			
	AH-1	AH-2	AH-3	AH-4
Q(cm3)	150	70	120	90
R(cm)	7.5	7.5	7.5	7.5
H(cm)	75	75	75	75
A(cm)	75	75	75	75
X (%)	<15%	<15%	<15%	<15%
K(cm/sec)	2.3×10^{-6}	1.1×10^{-6}	1.9×10^{-6}	1.4×10^{-6}
Cu	32	32	32	32

The average coefficient of permeability calculated from table 5.14 comes out to be 6.7×10^{-6} cm/sec, which implies that the reservoir is blanketed with impermeable soil deposit with the exception in the river bed.

5.3 ENGINEERING GEOLOGICAL CHARACTERIZATION OF ROCKS OF THE STUDY AREA.

An assessment of engineering geological features of the foundation soil and rocks under the dam, reservoir, spillway, intake and bottom outlet structures may be helpful for safe design parameters and criteria to design appropriate type and size of structure. This may also help in adopting proper remedial measures to overcome the adverse characteristics of the foundation. The engineering geological appraisal of Rib dam project is carried out based on existing pre feasibility level data and by generating primary data from insitu and laboratory tests.

5.3.1 Weathering

The atmosphere confronts exposed rocks with forces and reagents that tend to break them apart and transform them into soils, this process referred to as weathering. Most civil construction works intersects rock at relatively shallow depths beneath the surface within the weathering limits in which the rock properties have been considerably altered. Materials under foundation and footings and in shallow cuts for roads and pipelines all confronts weathered rocks. Further, the creation of new rock exposures in the cuts for major structures, like spillway and abutments initiates the weathering of these rocks and their consequent deterioration, immeasurably slow for most hard rocks but rapid for some. Thus, the processes and effects of weathering are vitally relevant to engineering work.

The foundation rocks of the present dam site as a whole are affected by various degrees of weathering. Part of these rocks are slightly to moderately weathered whereas, the rest are highly weathered and disintegrated.

As observed on exposed rock out crops, which are relatively very few in number, and in the boreholes, weathering does not follow uniform trend and it varies vertically and horizontally. The exposed rock on the abutments is the aphanetic basalt. As indicated from the bore hole data amygdaloidal basalt is present below the aphanetic basalt. Table 5.14 shows the degree of weathering encountered in each bore hole log.

Table 5.15 weathering variation as observed in bore holes

Depth	Degree of weathering	Rock type
Bore Hole BH-1		
6.8-15.07	Slightly weathered	aphanetic basalt
15.07-21.07	Moderately weathered	" "
21.07-26.67	Fresh to slightly weathered	Gray aphanetic basalt
26.07-30.87	Moderately weathered	Reddish aphanetic basalt
30.87-32.57	Highly weathered	Basic tuff
32.57-41.22	Slightly weathered	aphanetic basalt
41.22-61.17	Moderately weathered	Amygdaloidal basalt
61.17-63.17	Moderately weathered	Basic tuff
63-70	Moderately weathered	Amygdaloidal basalt
Bore Hole BH-3		
4.93-6.3	Moderately weathered	aphanetic basalt
6.3-7.55	Slightly weathered	aphanetic basalt
7.55-16.69	Moderately weathered	aphanetic basalt
16.69-17.77	Paleo soil	-----
17.77-25.22	Highly weathered	aphanetic basalt
25.22-26.57	Highly weathered	Basaltic tuff
26.57-27.83	Moderately weathered	Amygdaloidal basalt

5.3.2 Discontinuities

A discontinuity represents a plane of weakness with in a rock mass across which the rock material is structurally discontinuous. Although discontinuities are not necessarily planes of separation, most in fact are not and they posses little or no tensile strength. Discontinuities vary in size from small fissures to huge faults. The most common discontinuities are joints and bedding planes. Assessment of joint characteristics is extremely important in evaluation of dam site projects because of the following reasons.

- The system of joints controls the permeability of the rock mass.
- The system of joint promotes weathering significantly and creates significant weakness and fluid conductivity.

- The orientation of joint influence rock slope stability.
- The spacing of joints affects overall rock mass strength or quality.
- The surface characteristics of joints influence the shear strength of the rock mass.
- The separation and filling of joint may have profound influence on the strength and permeability of the rock mass.

In the present study area the only observed discontinuities on rock out crops and drilled cores are joints. Three major sets of joints were identified on the left abutment and their characteristics are given in table 5.15

Table 5.15 Characteristic of joint sets as observed on left abutment.

Joint characteristics	Joint sets		
	J1	J2	J3
Preferred orientation Dip direction/ dip amount	$N35^0w/30^0$	$N47^0/65^0$	$S72^0/75^0$
Dip amount	20^0-30^0	60^0-85^0	80^0-90^0
Dip direction	200-300	120-160	15-45
Spacing	10-20cm	20-40cm	20-30
Continuity	Discontinuous	Discontinuous	Discontinuous
Surface characteristics	Smooth	Smooth	Smooth
Separation	0.5-1cm	0.5-1cm	0.5-1cm
Filling	Clay	Clay	Clay

5.3.3 Rock Mass Classification System

Rock mass classification is a widely used, economical and extremely useful basis for determining engineering properties. The classification methodology should be critically examined to optimize the usefulness of the measured data.

Rock mass classification is the basis to make a definitive decision on engineering projects involving rock, since rock mass classification can represent the actual site conditions than intact rock classifications do. The stability and deformability of the structure is dependent on the strength and deformability of the rock mass. Discontinuities such as bedding surfaces, faults, joints and foliations are the most important characteristics of the rock mass. The presences of discontinuities reduce the Strength of the rock mass. Besides, discontinuities primarily control the mass strength and deformability of the rock mass. Thus, the use of rock, whether as foundation material in excavations and tunnels or in maintaining stable slopes involves determinations of properties of rock mass in which the presence of discontinuities govern the engineering character.

developed by Barton, Lien, and Lunde (1974), Bieniawiski (1974, 1984, 1989), and Wickham, Tiedemann, and Skinner (1974) (Goodman, 1980).

Bieniawiski's classification system, Geomechanics Classification or Rock Mass Rating System (RMR) is based on the assumption that strength of rock mass largely depends on the density, nature and extent of the fracture within it. The weathering and ground water conditions are also related to the rock mass strength. Bieniawiski's classification provides a general rock mass rating (RMR) increasing with rock quality from 0 to 100 based on six parameters. The RMR classification system considers six parameters, i) Unconfined compressive strength (UCS), ii) Rock Quality Designation (RQD (%)), iii) Mean fracture spacing, iv) Discontinuity condition, v) Ground water conditions and vi) Orientation of discontinuities. The engineering evaluation of an area, considers measurement of discontinuity characteristics. Discontinuity characteristics mainly depends on, i) Orientation , ii) Spacing, iii) Continuity, iv) Surface characteristics and v) Separation and physical properties of the in filled material. The conditions of discontinuities, spacing and orientation of discontinuities and ground water conditions are determined in the field. The spacing of the discontinuities has been measured at each locality and the condition of discontinuities and the groundwater conditions were visually assessed.

For the present study, data pertaining to RMR has been collected from 7 locations in the limited out crops. Schmidit hammer rebound value is used to determine the strength of the rock (UCS). Whereas, RQD was determined using Palmstrom's volumetric count method (eq. No. 5.18)

Based upon the measurements/observations at each locality each of the parameters was assigned ratings as per the standard tables of the RMR system. The sum total of the ratings for each parameter provides the RMR value which is presented in tables 5.16. According to the RMR classification (Table 5.16) the aphenitic basaltic outcrops exposed at or around dam site, are of poor quality (class III).

5.3.4 Rock mass Deformation

The rock mass deformation results primarily from the closure of discontinuities and the elastic and plastic deformation of the intact rock that comprises the rock mass. In rock mass the modulus of deformation, E_d is used as a measure of deformation. The modulus of deformation is defined as the sum of deformation that occurs with closure of joints in the rock mass under deformation (plastic) and the deformation that occurs with continued stress application after crack closure (elastic) (Jonson, 1988).

Table 5.16 RMR classification of the Aphanatic basalt

Location		UCS	RQD	Spacing	Ground water	Discontinuity condition	RMR	Class	Rock Quality
Left Abutment	Value	21.36	75	23	Damp	Smooth, moderately weathered	43	III	Poor
	Rating	2	17	10	10	4			
Right Abutment	Value	19.07	72	20	Damp	Smooth, moderately weathered	37	III	Poor
	Rating	2	13	10	10	2			

Agarwell et al proposed an emperical relation to workout modulus of deformation 'E_d' from the rock mass rating (RMR) value. The values of modulus of deformation (E_d) obtained by Agarwall et al (1991) relation are the closest to the actual observed data .According to the relation:

$$E_d = 10^{(RMR-10)/50} \qquad \text{............ 5.10}$$

Serafim and Perera (1983) have also proposed emperical relation based on the case historys of mostly dam foundations by back analyses of deformation modulus from the measured deformation. This relation works good for the rockmass for which RMR lies between 10-50.

According to the relation:

$$E_d = 10^{(RMR-10)/40} \qquad \text{............ 5.11}$$

Based on the above relations the deformation of modulus of rock outcrops at the abutments is presented in table 5.17

5.3.5 Rock mass shear strength

The strength of jointed rock mass is notoriously difficult to assess. Laboratory tests on rock samples are not representative of rock mass of significantly larger volume. On the other hand, insitu strength testing of the rock mass is seldom practical or economically feasible.

Table 5.17 Modulus of deformation 'E_d' of the rocks exposed in and around dam site

Location	Empirical relation used (UNITS)	
	Agarwal et al (1991)	Serafim and Periera (1983)
Left abutment	1.82	6.70
Right abutment	1.38	4.73

Back analysis of observed failures can provide representative values for larger scale rock mass strength but obviously this is possible for cases in which rock mass failure has occurred. The more general problem of forward strength prediction for larger scale rock mass remains as one of the great challenge in rock mechanics.

Currently more or less accepted approach to this problem is the Hoek and Brown failure criteria and estimates the required parameters with the help of rock mass classification. This method is relatively well established but not with out its difficulties. Hoek and Brawn (1980) proposed a method for estimating the strength of jointed rock mass based on an assessment of the interlocking of rock blocks and the condition of the surface between these blocks. This method was modified over the years to solve problems related to parameters that are not considered when the original criterion was developed. The application of this method to very poorly quality rock masses required further changes and development of a new classification called the Geological Strength Index (GSI) (Hoek 1995). The generalized Hoek and Brown failure criterion for jointed rock mass is given by the equation.

$$\delta_1 = \delta_3 + \delta_c \left(m_b \frac{\delta_3 + S}{\delta_c}\right)^a \quad \text{.............................. 5.12}$$

Where:

δ_1 and δ_3 are the major and minor principal stress, respectively.

δ_c is unconfined compressive strength of a intact rock sample.

m_b,a & S are material constant, which depends upon the physical properties of the rock mass.

The material constant 'mb', 'S' and 'a' used in the equation 5.12 can be determined from the Geological Strength Index (GSI) developed by Hoek and Brown (1989). The GSI for RMR > 23 is calculated by the relation given by eq.5.13.

$$GSI = RMR-5 \quad \text{......... 5.13}$$

For GSI > 25 (undisturbed rock mass) the material constants can be calculated by the relation:

$$mb = mi * \exp \frac{(GSI-100)}{28} \quad \text{......... 5.14}$$

$S \quad = \quad \exp \frac{(GSI-100)}{9}$ 5.15

$a \quad = \quad 0.5$ 5.16

For the present study an attempt has been made to workout the shear strength parameters of the rocks, exposed at the dam abutments, by Hoek and Brown failure criteria. The RMR values obtained on the left and right abutments are 43 and 37, respectively. The determination of RMR is already discussed in detail in section 5.3.3.

Thus, the GSI values as computed by eq. 5.13 comes out to be 38 and 32 for left and right abutments, respectively. The value of the constant "mi " is taken from the standard table prepared for rock groups, for basalts it is 17 (Hoek and Brown, 1989). Thus, by using eq. 5.14, 5.15 and 5.16 the material constants for both abutments have been computed and are presented in table 5.18

Table 5.18 Input data sheet for computation of shear strength

Input Parameters	Value	
	Left abutment	Right abutment
Mi	17	17
GSI	38	32
S	1.02×10^{-3}	5.23×10^{-4}
Mb	1.86	1.50
δ_c	21.36	19.07
a	0.5	0.5

The values of normal stress (σ_n) and shear stress (τ) are obtained from Mohr circle drown from the values of minor principal stress (σ_3) and major principal stress (σ_1). The maximum value of the minor principal stress (σ_3) can be taken upto a maximum of one fourth of the unconfined compressive strength and the corresponding value of major principal stress (σ_1) is calculated from equation eq. 5.12. The results thus obtained are presented in table 5.19. Further, based on the major and minor principal stresses Mohr diagram was prepared to obtain the shear strength parameters for rocks exposed at left and right abutment (Fig. 5.5).

Table 5.19 Major principal stress for different minor principal stress

Location	Stress	
	Minor Principal stress δ_3	Major Principal stress δ_1
Left abutment	5.34	19.92
	3	13.94
	2	10.94
Right abutment	4.77	16.46
	2.5	10.97
	1.5	8.06

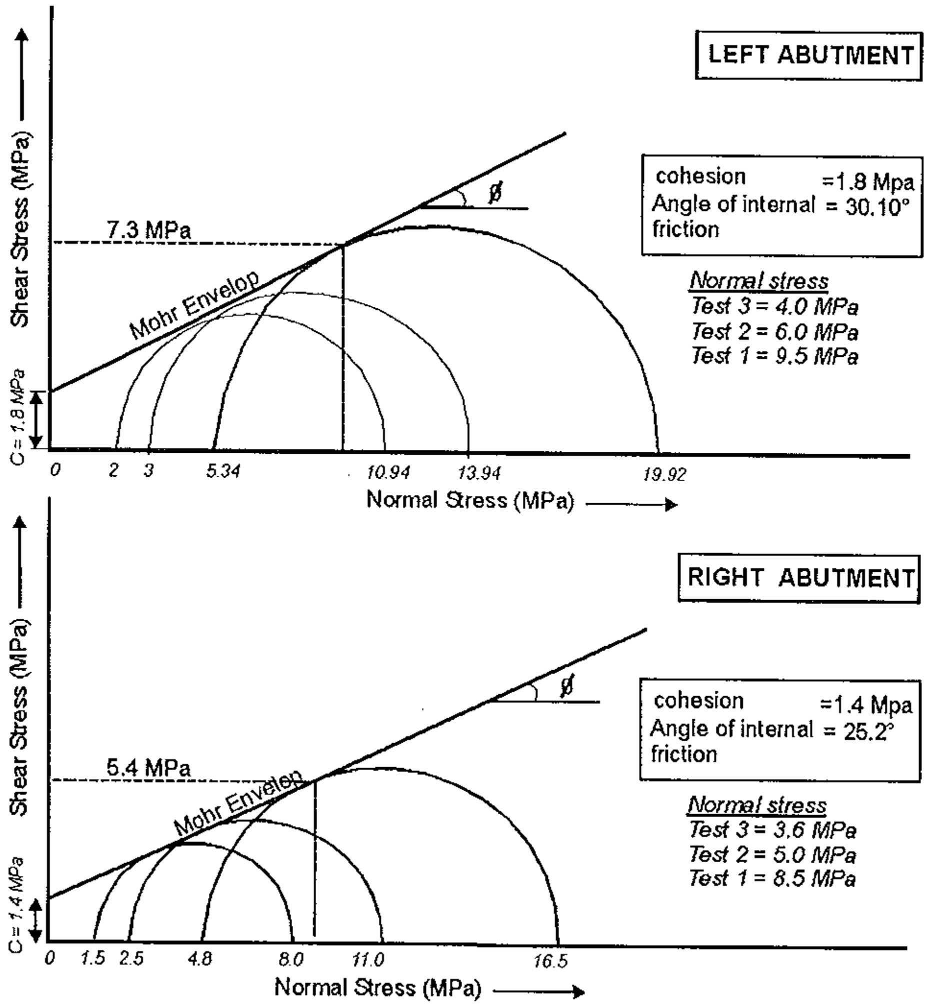


Fig. 5.5 Mohr circle diagram drown for left and right abutment.

The results thus obtained from Figure 5.5, for cohesion and angle of friction for both abutments are summarized in table 5.20

Table 5.20 Shear Strength parameters as obtained from Mohr Diagram

Left abutment	Right abutment
• Cohesion( ) = 1.8Mpa	• Cohesion( ) = 1.4Mpa
• Angle of internal friction( ) = 30.1^0	• Angle of internal friction( ) = 25.2^0
Normal stress ( ) : Test 1 = 9.5 Mpa Test 2 = 6 Mpa and Test 3 = 4 Mpa	Normal stress ( ) : Test 1 = 8.5 Mpa Test 2 = 5 Mpa and Test 3 = 3.6 Mpa

5.3.6 Unconfined Compressive Strength (UCS)

The most common method to determine the UCS of an intact rock is by UCS testing machine. The standard shape for the UCS testing is core cylinder of 54 mm diameter with standard diameter to length ratio in the range of 2.5 to 3.0. The variation in size of the core may influence the results.

For the present study the core sample of prescribed dimension were not available. Hence, the UCS has been determined by using Schmidt hammer. The UCS from the Schmidt hammer value has been determined by using Barton and Choubey (1977) emperical relation. This relation is given by eq. 5.17;

$$\log_{10} \sigma_C = 0.00088\gamma R + 1.01 \qquad \ldots\ldots\ldots\ldots 5.17$$

Where; σ_C is the un-confined compressive strength in Mpa

γ = dry rock density in KN/ m^3

R = rebound number of Schmidt hammer

The average Schmidt hammer rebound number obtained form the rock out crop exposed in trench excavations on the left and right abutments were 13 and 11, respectively. The average UCS value as obtained from eq. 5.17 are given in table 5.21

Table 5.21 Average un-confined compressive strength of rocks on both abutments

Location	Schmidt Rebound Number	Dry rock density kN/m^3	UCS (MPa)
left abutment	13	27.93 kM/m^3	21.26
Right abutment	11	27.98	19.07

5.3.7 Rock Quality Designation (RQD)

The RQD was proposed by Deere (1963) as an index of rock fracturing. In 1967 a modified core recovery procedure was developed to provide a rock quality designation (RQD) for a given cored interval of NX core (Deere al, 1967). The RQD value is the percentage obtained by dividing the summed length of all core pieces equal to or greater that 10 cm in length by the cored interval length. Smaller pieces and/or core loss, are assumed to result form closely spaced discontinuities, shearing, faulting or weathering, all of which decrease rock mass quality. RQD values of 100% does not imply an un jointed rock mass but pieces greater than 10 cm (Bell, 1992).

The RQD percentage is directly proportional to the various measures of rock mass quality such as fracture frequency, velocity ratio and insitu modulus of deformation. It also provides some measure of rock loosening that may occur. Most of the methods advanced for engineering classification of rocks use the RQD as one of the parameter. The RQD in boreholes is direction dependent and it's value may change significantly depending upon orientation. Therefore, for the present study empirical relation proposed by Palmstorm (1982) has been utilized,

$$RQD = 115 - 3.3\ Jv \quad \text{......... 5.18}$$

Where;

Jv is the sum of the number of joint in 1m x 1m exposed rock face.

The average RQD values as obtained by eq. 5.18 are given in table 5.22

Table 5.22 Average RQD values for Rock mass exposed on left and right abutments

Location	Volumetric Count (Jv)	RQD (%)	Description
Left Abutment	12	75	Fair rock
Right Abutment	13	72	Fair rock

5.3.8 Permeability of The Rock Mass

The permeability of rock abutments and dam foundations is determined mostly by the joint and crack patterns. Undetected joints or strata of higher permeability in the foundation or abutments of dam creates serious leakage and uplift problems (Cendergren, 1977)

In order to obtain test results as reliable as possible drive holes and water pressure tests must be distributed to determine the permeability of all rock types that occur in the bottom and flanks of the dam site at representative locations. The permeability of rocks is assessed on the basis of criteria established by various authors like Lugeon, Jahde, Terzaghi, Holsby etc, However Lugeons criteria is most commonly used as it given reliable value for the first orientation. According to Lugeon, in dams higher than 30m the water loss in water pressure tests should not exceed 1liter in 1minute per 1meter of the hole at 10 atm pressure which act at least 10 minute. The permeability results are normally described in terms of Lugeon units, one Lugeon is equal to a flow of 1 liter/m/min at a pressure of 1 MPa and a Lugeon unit is approximately equal to a coefficient of permeability of 10^{-7}m/sec. According to Lugeon (1933) rock absorbing less than one lugeon unit can be considered water tight.

For the Rib Dam site permeability tests using packer tests were conducted by the, Project Authorities, on three boreholes located along the dam axis. These tests were conducted in boreholes BH-1, BH-2, and BH-3 which are located on left abutment, riverbed, and right abutments, respectively.

The permeability results are presented in Table 5.23. A perusal of table indicates that permeability does not follow a regular pattern with depth. However, in all the bore holes the permeability of rock mass varies from 10^{-5} to 10^{-8}. From the test results of the bore holes it can be concluded that the permeability of the rock mass at the dam axis alignment is at the boundary of permeable to semi permeable zones (Bell, 1992).

Table 5.23 Results of permeability tests

BH N0	Test interval (m)	Permeability value	Description
BH-1	0-15	$3.45x10^{-5}$m/sec	Permeable
	15-21	$5.745x10^{-8}$m/sec	Semi permeable
	21-22.8	$2.9645x10^{-6}$m/sec	Boundary of semi permeable
	22.8-33.3	$2.745x10^{-7}$m/sec	semi permeable
	33.3-39.6	$5.145x10^{-8}$m/sec	semi permeable
	39.6-45.0	$3.445x10^{-7}$m/sec	semi permeable
	45.0-66.0	$4.6845x10^{-8}$m/sec	semi permeable
	66.0-70.0	$1.3945x10^{-6}$m/sec	
BH-2	27.18-60	$6.16845x10^{-8}$m/sec	semi permeable
BH-3	0-15.0	$4.86\ x10^{-6}$m/sec	Boundary of semi permeable
	15.0-21.6	$4.12\ x10^{-8}$m/sec	semi permeable
	21.6-25.2	$4.62\ x10^{-6}$m/sec	Boundary of semi permeable
	25.2-34.5	$4.46\ x10^{-7}$m/sec	semi permeable
	34.3-60.9	$4.01\ x10^{-8}$m/sec	semi permeable
	60.9-64.5	$4.34\ x10^{-7}$m/sec	semi permeable

5.4 ENGINEERING GEOLOGICAL MAPPING

The general objective of a site investigation is to assess the suitability of a site for the proposed structure. Engineering geological maps may serve as a special purpose maps which provide information on specific aspect of engineering geology such as grade of weathering, jointing patterns, mass permeability or foundation conditions. Whereas, multipurpose engraining geological maps cover various aspects of engineering geology and provide information for planning or engineering purposes (Bell, 1992).

The engineering geological maps indicate the distribution of units, defined in terms of engineering properties. The unit boundaries are drawn for changes in the engineering properties. However, mostly the boundaries of the units coincide with the stratigraphical boundaries.

For the present study, Engineering geological mapping of the dam site and reservoirs area has been carried out based on the strength parameters and mass permeability classes, respectively. The aerial extent of the engineering geological mapping covers the dam and the reservoir area. The dam site and the reservoir area are mapped at the scale of 1:2000 and 1:10000, respectively.

5.4.1 Engineering Geological Mapping of The Dam Site

Engineering properties of rocks and soil units has been derived after analysing the primary and secondary data. The index property tests are used for classifying soils and rocks. Further, based on the classification of rocks and soils the engineering properties of these units were derived. Later, based on the engineering properties of rocks and soils engineering map for the dam site has been prepared. There is a limitation of presenting all the engineering properties on a map, therefore only selected engineering properties namely, compressibility and shear strength has been presented. For the dam site the soils and rocks have been classified into following 5 units, based on the parameters variation and an engineering geological map has been prepared which is shown in Fig. 5.6 . The parameter variation at dam site coincides with stratigraphic boundaries.

1. <u>Al1</u> - Recent alluvial deposit consisting of sand, gravel, cobble, and boulders (GW) having good bearing value for foundation and excellent shear strength.
2. <u>Al2</u> - Flood plain deposit consisting of inorganic clay of low to medium compressibility(CI) having good to poor bearing value for foundation and fair shear strength value, average cohesion C=71kn/m2 and angle of internal friction $\phi=25.1^0$,Fair to poor shear strength.
3. <u>Al3</u> - River terrace deposit consisting of inorganic clays of high compressibility (CH), Fair to poor bearing value for foundation and poor shear strength, average cohesion C = 79.3kn/m2 and angle of internal friction $\phi=16.8^0$, Fair to poor shear strength.
4. <u>SCO</u> - Colluvial and residual soils consisting of loose clayey sand (SC) having good to poor bearing value and good to poor shear strength (average cohesion(C) =34.5kn/m2 and angle of internal friction (ϕ) =33.9^0), Fair to poor shear strength.
5. <u>Ba</u> - Moderately to highly weathered, moderately strong basalt (UCS=19.07-21.36 Mpa)

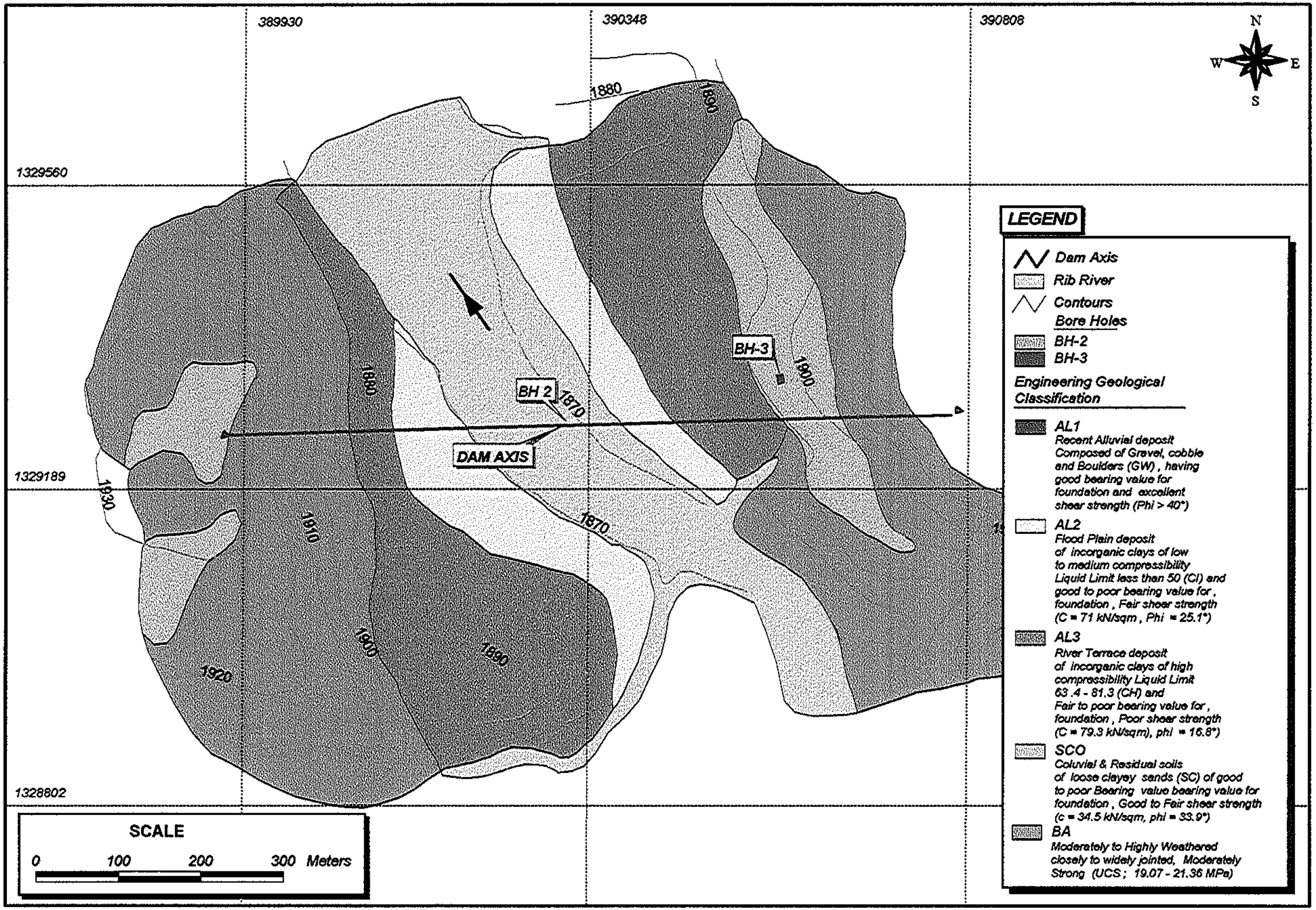


Fig. 5.6 Engineering Geological Map of the Dam site

5.4.2 Engineering Geological Mapping of The Reservoir Area

The water tightness of the reservoir area is an important engineering geological parameter in evaluating the reservoir feasibility. In order to determine the water tightness of the reservoir area, tests has been carried out, these tests include soil classification and insitu permeability. Based on these test results, the hydraulic conductivity of the soils and rocks of the reservoir area has been determined. In total four hydraulic conductivity parameter variations has been identified and based on these units the engineering geological map has been prepared for the reservoir area and is presented as Fig. 5.7.

Flood plain clayey silt	- $K=3.8x10^{-6}$ to$7x10^{-6}$ cm/sec	**Terrace silty clay deposit**	- $K=1.68x10^{-6}$ to $6.0\ x10^{-7}$ cm/sec
Recent river deposit	- $K=10^{-3}$ to 10cm/sec	**Weathered basalt**	- $K=4.86x10^{-6}$ to $3.45x10^{-5}$ cm/sec

5.10 SELECTION OF THE SPILLWAY ALIGNMENT

Among the factors which affect the spillway design and its capacity are the geological and topographical conditions of the site such as steepness of the terrain, possibility of scour down stream, stability of slopes etc.

For the Rib dam site, based on the topographical suitability, the dam site has two alternative spillway arrangements. Each of these alternative is located on either abutments. However, the alternative on the right abutment doesn't full fill the geological criteria. The possible spillway arrangement site on the right flank is dominantly covered by deep alluvial deposit mainly accumulated by the small tributary stream which joins the Rib River downstream of the dam axis. Since, this spillway arrangement option has failed to satisfy stability and scouring resistance criteria. The only possible option would be on the left abutment. This spillway alignment crosses borehole BH-1. Based on the information obtained from BH1- log, this site is covered with 6.8m thick highly weathered and fragmented basalt, overlying slightly weathered aphanetic basalt. The rock quality designation (RQD) value obtained from the bore hole BH1 below 6.8m depth, indicates that the basalts are slightly weathered (RQD = 80%), which implies rock is of very good quality. Hence based on the degree of weathering and rock quality the spillway site can be designed on the left flank of the Rib river. As it was seen in the stability analysis of the abutments (discussed later in Chapter 6) having steep slope than the spillway site, instability of slope was not found at any degree of saturation however, for the cut slope retaining walls may be required.

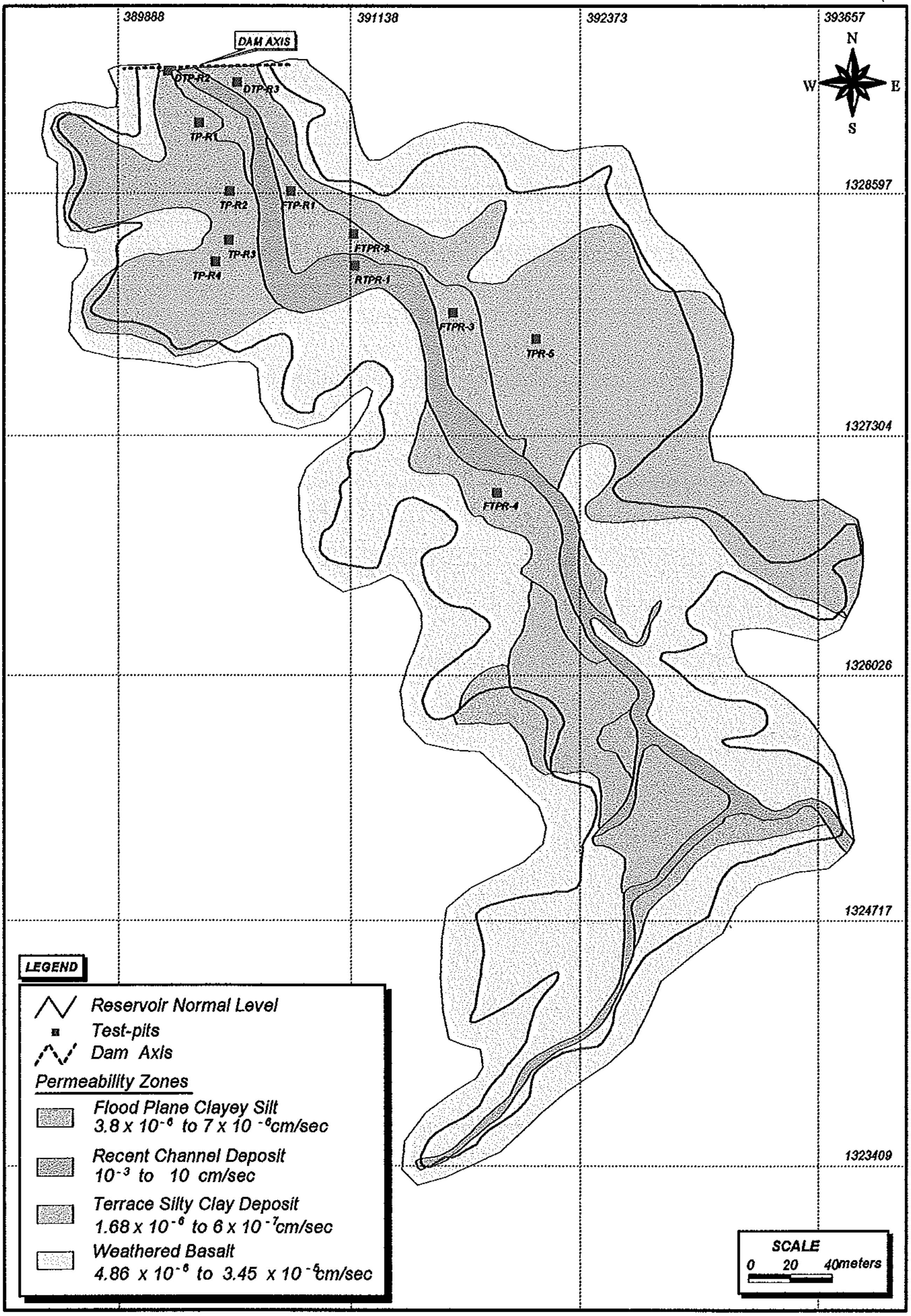


Fig. 5.7 Engineering Geological Map (Based on Permeability) of the Reservoir Area

CHAPTER VI

SLOPE STABILITY STUDIES

6.1 GENERAL

Design of stable cut slope is a prerequisite for the safe construction and functioning of a dam project. Potential instability of slopes in the reservoir rim area or the abutment slopes may pose problems during construction and operational stages. Therefore, it is essential to recognize the potential slope stability problems in the initial stages of investigations, so that proper remedial measures can be adopted for the slope stabilization.

For the present study stability studies has been carried out for both abutments and the slopes in the reservoir rim area. In order to carry out the stability studies for the abutments, cross sections of both abutment slopes were prepared to workout the slope geometry and the distribution of the geologic units present on the abutments. Further, discontinuity data were collected from the abutment slopes, from natural outcrops and from the excavated trenches, and was stereographically analysed to get the preferred orientation (Fig.6.1) In addition to this data pertaining to Geomechanical classification system or Rock Mass Rating (RMR) system has been collected from 7 localities and was utilized to workout the shear strength parameters.

6.2 STABILITY STUDY OF LEFT ABUTMENT SLOPE

Based on topography the left abutment can be categorized into two slope segments, the first slope segment is flat and extends from the river channel (El 1870m) upto the left flank (El 1890m)and the second slope segment is relatively steeper and extends from El 1890m to the top of the hill (El 1930 m). Figure 6.2 shows the slope geometry for left and right abutment. Predominantly the left abutment is composed of two geological units, these units are, aphanetic basalt and the alluvial terrace deposit. The aphanetic basalt unit is moderately weathered having closely to widely spaced joints and it covers relatively steeper slope segment of the left abutment.

The river terrace deposit is composed of predominantly silty clay and the grain size and thickness of this deposit increases as it is traced towards the river course (Fig. 8.1, Chapter 8)

The shear strength parameters of the aphanetic basalt is determined from the empirical relations derived from the rock mass rating value (RMR), (discussed in Chapter 5).

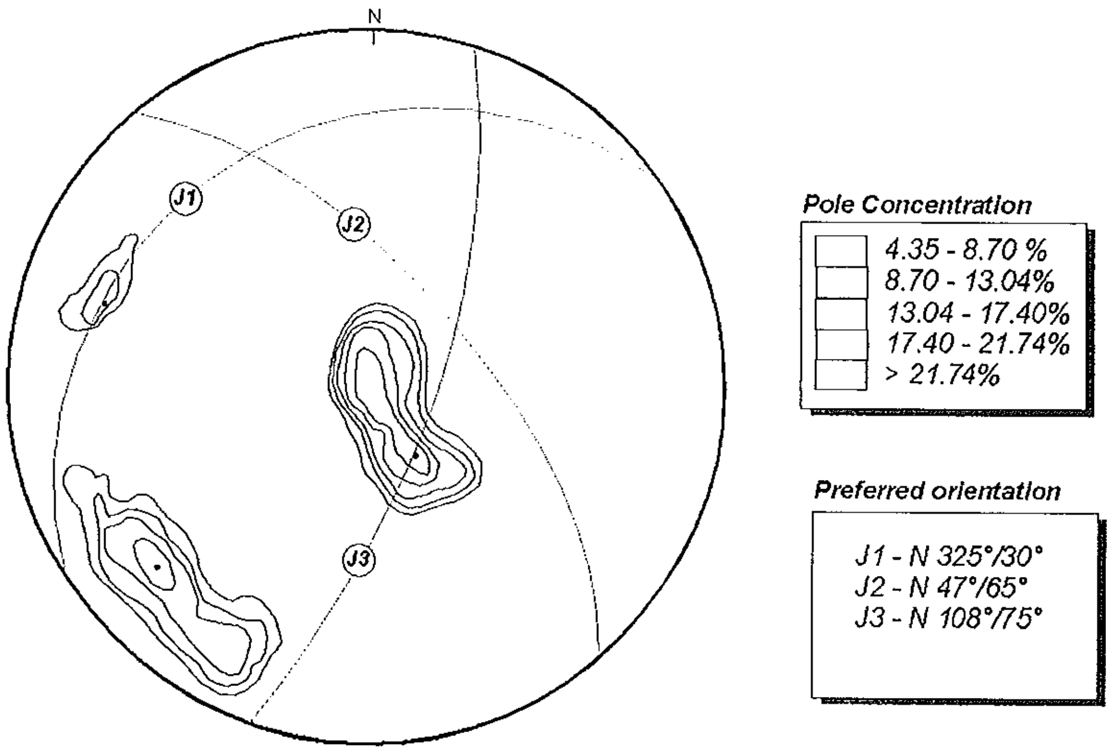


Fig. 6.1 **Preferred orientation of discontinuity planes as observed on the left abutment.**

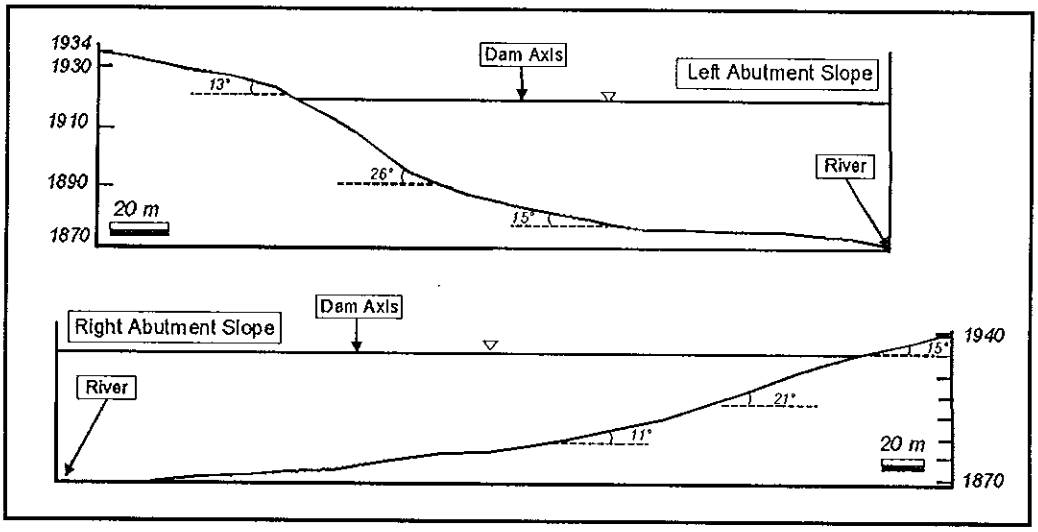


Fig. 6.2 **Slope Geometry of Abutments.**

The calculated RMR value of the the aphtaneic basalt is 43, from this value the shear strength parameters are calculated by using Bieniawiski's relation;

$$C=0.05xRMR\ (kg/cm^2) \qquad \ldots\ldots\ldots\ 6.1$$

$$\phi=0.5\ RMR+5\ (deg) \qquad \ldots\ldots\ldots\ 6.2$$

Thus, by utilizing eqs. 6.1 and 6.2 shear strength parameters have been derived. The value of cohesion for aphenatic basalt is 2.15 kg/cm^2 whereas, the angle of friction comes out to be 26.5^0.

6.2.1 Possible Mode of Failure

In order to work out the possible mode of failure on the left abutment Markland test has been applied. For this test the slope face inclination, the failure plane inclination, the plunge of the line of intersection of wedge forming plane and the angle of internal friction has been considered and later they were plotted on a stereoplot. According to Markland test the Kinematic conditions are;

For plane failure .... $\alpha_f > \alpha_p > \phi$ 6.3

For Wedge failure .... $\alpha_f > \alpha_I > \phi$ 6.4

Where: αf is angle of slope face, α_p is the dip of potential failure plane, α_I is the plunge of the line of intersection of the two wedges forming plane and ϕ is the angle of internal friction

The plots of the great circles (Fig. 6.3) indicate that the planes do not satisfy the Markland's kinematic condition of failure; hence the left abutment slope is potentially stable.

6.3 STABILITY STUDIES ON THE RIGHTABUTMENT

The right abutment slope is principally composed of highly weathered aphenatic basalt overlain by shallow colluvial soil deposit. The stability analysis has been carried out for circular mode of failure, since the rock material is intensively weathered and behaves as soil rather than rock. For the determination of shear strength parameters of this rock mass the empirical relation proposed by Bieniawiski (eq. 6.1 and 6.2) has been utilized. The rock mass rating for rock mass exposed on right abutment is of poor quality and shows RMR value equal to 37 (Table 5.16, Chapter 5). The shear strength parameters as determined from RMR give the following values;

C=0.05*RMR (kg/cm^2) = 1.85 kg/cm^2

Φ= 0.5*RMR+5(deg) = 23.50

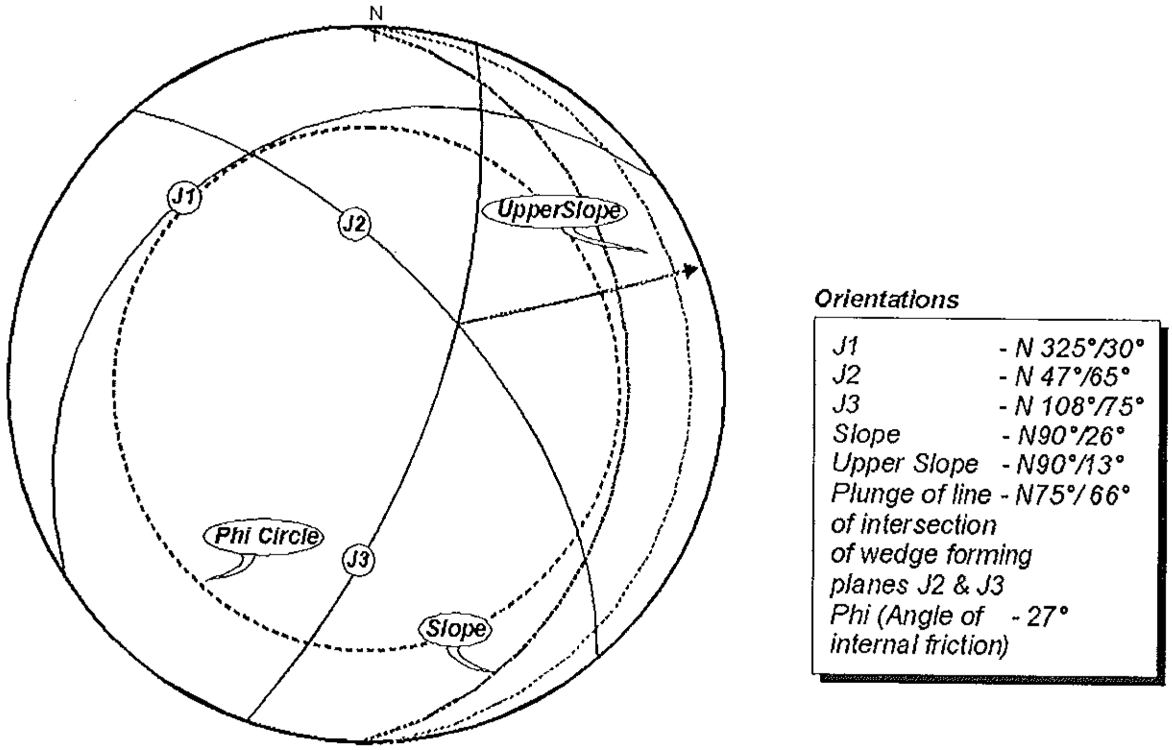


Fig. 6.3 Markland test for left abutment

The right abutment slope is 50 m high (1890m to 1940 m) and has a gradient of 0.36 (height vs. width). The slope geometry of right abutment slope is shown in Fig. 6.2.

In order to carry out the stability analysis 'Limit Equilibrium Method' has been adopted and the Factor of Safety (FOS) has been calculated for static conditions under varied water saturation situations.

The FOS has been determined by using SARC computer program which utilizes the Bishop's equation for various slip surfaces. The software analysis the FOS for all possible failure slip surfaces and finally, provide FOS for the slip surface for which it is minimum. A brief description about SARC is given at Annexure 1.

To perform the stability analysis SARC software is utilized and for the input data 14 profile coordinates were taken from the slope profile and a series of calculations were carried out assigning values of cohesion zero and different water saturation conditions. And the analysis is done under the static and dynamic conditions since the area is located in the earth quake intensity zone of IV the vertical and horizontal accelerations are taken based on this intensity (already discussed in Chapter 4). The input data sheet for stability analysis of right abutment is presented at Table 6.1

Table 6.1 Input data used for the stability analysis of the right abutment

Paramters	Values
N	14
X (I), Z (I), I=1 up to N	(0.0, 1890.0), (20, 1894.0), (28, 1896.0), (46, 1900.0), (56, 1903.0), (68, 1906.0), (82, 1910.0), (90, 1916.0), (112, 1920.0), (120, 1924.0), (130, 1926.0), (146, 1930.0), (158, 1934.0), (182, 1940.0)
Rock, RWL, XS, WI, ZC, ZWR	0, 0.0, 146, 0.0, 0.0, 0.0
C, PHI, GAMA, GAMAW, BBAR, AH, AVR, EQM	0, 23.5, 2.79, 1.0, 0.2, 0.0, 0.0, 0.0 BBAR will have values 0.2, 0.1 and 0.0 for saturated, moderately saturated and dry conditions respectively. And the values of AH, AVR and EQM will be respectively 0.01, 0.05 and 4 for dynamic conditions and zero for static conditions.
NENP, <ENTX<I>, ENTY, I=1 TO NENP>	1, 0.0, 1890
NEP, NOPT	0, 0
XEXITI, XEXITL, GAP	68, 182, 114

Where:

N- Number of profile coordinates, X (I), Z (I)- Coordinates of profile points,
ROCK-Reduced level of hard strata with respect to origin
RWL= Reduced level of GWT/reservoir water with respect to origin
XS= X-coordinate of point from where surcharge starts
WI= Uniform surcharge intensity
ZC= Depth of tension crack
ZWR= Depth of water in tension crack (ZC)
C= Cohesion of soil/rock
PHI= Angle of internal friction of soil/rock
GAMA= Unit weight of soil/rock
GAMAW= Unit weight of pore water
BBAR= Pore water pressure/ (GAMA* Average height of slices)
AH=Horizontal component of earth quake acceleration near crest of slope
AVR= Vertical component of earth quake acceleration
EQM= Corresponding earth quake magnitude on Richter scale
NENP= Number of entry points of slip circles
ENTX= X-coordinate of entry point of circle
ENTY= Y-coordinate of entry point of circle
NOPT=0, When only minimum factor of safety is required
=1, when all factor of safety corresponding to all exit points are also required
NEP= Number of exit points (NEP=0, When no individual point is given)
XEXITI= X-coordinate of first exit point of circle
XEXITL= X-coordinate of last exit point of circle
GAP= Horizontal distance between consecutive exit points
XEXIT= X-coordinate of exit point of circle

The existing conditions at the time of field visit, was static dry. However, the analysis has also been carried out for both static and dynamic conditions represented by all water saturation conditions.

The worst possible condition could be represented by moderately saturated slope with cohesion equal to zero. Thus, the calculated FOS for different conditions is presented through Table. 6.2

Table 6.2 Stability condition of right abutment slope for existing and possible worst conditions.

Conditions		FOS	Dynamic displacement	Weight (Tones)	Failure slip circle		
					Coordinates		Radius (m)
					Center	Exit point	
Static	Dry	1.47	0	9.4×10^2	(-226.77,3071.68)	(182.0, 1940.0)	1203.24
	M/Sat.	1.31	0	9.4×10^2	(-226.77,3071.68)	(182.0, 1940.0)	1203.24
	F/Sat.	1.15	0	9.4×10^2	(-226.77,3071.68)	(182.0, 1940.0)	1203.24
Dynamic	Dry	1.42	0	9.4×10^2	(-226.77,3071.68)	(182.0, 1940.0)	1203.24
	M/Sat.	1.26	0	9.4×10^2	(-226.77,3071.68)	(182.0, 1940.0)	1203.24
	F/Sat.	1.11	0	9.4×10^2	(-226.77,3071.68)	(182.0, 1940.0)	1203.24

A perusal of table 6.2 indicates that the right abutment slope is stable for the existing conditions, i.e for static and dynamic dry condition, as the FOS value is 1.47 and 1.42 respectively. The slope will remain stable even for possible worst conditions i.e when the slope will be moderately saturated by water (FOS= 1.31and 1.26). Even for the worst conditions, represented by fully saturation conditions the slope will be critically stable. Although, fully saturated conditions may only occur when the slope is subjected to heavy rains which continues for days. Occurrence of such conditions is very rare. Therefore, it may be concluded that the right abutment slope is stable and may not pose any problem during the construction period.

Stability of the reservoir slopes

Stability of the reservoir slopes has an important consideration in the safe functioning of the dam. Major slides in the reservoir can significantly reduce the storage capacity of the reservoir and if larger magnitude of slope failure occurs close to the dam, the dam may fail due to overtopping. In order to identify the potential unstable slopes in the reservoir rim area, field reconnaissance study has been carried out. The collection of the field data was done following the principle of " whole to part" where the entire area was surveyed initially and the potentially unstable zones were identified based on there field manifestation of instability. However, the reconnaissance study has revealed that the slopes in the reservoir area are flat to gentle and the area is covered with cohesive soils. No traces for instability were recognized in the reservoir area. Thus, it may be concluded that the slopes in the reservoir rim are relatively gentle and may not pose any problem of instability during the operation of the reservoir.

CHAPTER VII

SELECTION OF DAM TYPE AND CONSTRUCTION MATERIALS

7.1 General.

Embankment dams are basically of two types, one Earth fill dams and other Rock fill dams. An earth fill dam is an embankment dam constructed primarily of compacted earth whereas, rock fill dam is one in which the principal structural material is rock either dumped in lifts or compacted in layers with sealing provided by an impervious element. Earth fill dams can be subdivided into: homogeneous dams, zoned dams and the rock fill dams. Rock fill dams can further be divided into two types; earth core and upstream facing rock fill dam.

7.2 EARTH FILL DAMS

7.2.1 Homogeneous dams

As the name implies homogeneous dams are built entirely of one type of material. The requirements of basic material properties are that the material should be sufficiently impervious to provide an adequate water barrier and the soil should develop adequate shear strength on compaction and maintain most of it on saturation after filling of the reservoir. A rock toe is provided on the down stream to improve stability against seepage.

7.2.2 Zoned dams

A zoned dam is one in which the dam section consist of central impervious core as the impervious element and the outer shells of pervious materials to provide stability. The up stream pervious zone affords stability against rapid draw down and the down stream pervious zone act as a drain to control the line of seepage.

7.3 ROCK FILLS DAMS

7.3.1 Earth core type

These are basically zoned sections with an impervious earth core for water tightness and rock fill material in the shells for strength. Earth - core rock fill dams require wide filter zones to take care of piping of core material as a result of excessive hydraulic gradients.

7.3.2 Upstream- facing type rock fill dam

In this type of dam the impervious element consist of an upstream facing which is usually concrete. Asphalt concrete facings are also used for dams of low to moderate heights.

7.4 SELECTION OF TYPE OF DAM

Based on the stability and availability of materials for construction zoned type earth fill embankment dam is the most suitable option for Rib dam site (Fig. 7.1). Therefore, this option is selected for this study.

The main objective of zoning the dam is to take advantage of the most economical combination of the readily available materials at the dam site and in the surrounding areas. Zoned type earth embankment dam facilitate the maximum utilization of material available from the excavation of appurtenant works and foundations. In addition the wider span of Rib river bed material comprising sand, gravel and cobble deposits which can be utilized for the outer shells. This may provide zoning that properly defends against excessive seepage pressures, erosion, and piping.

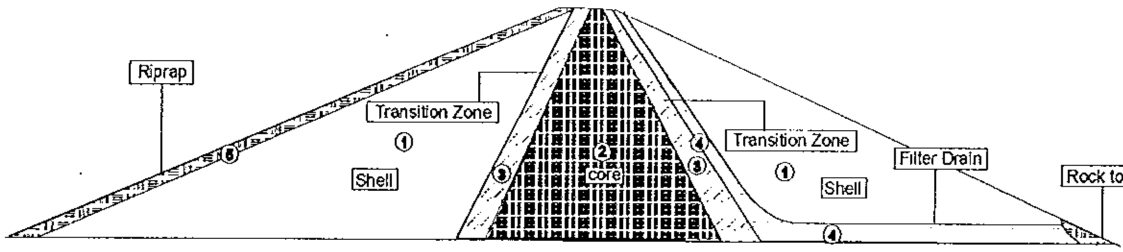


Fig. 7.1 Proposed zones for Rib Dam.

7.5 Construction Materials

The availability of construction materials for the dam embankment is a very important consideration in selecting the dam type. Moreover, this may influence the performance of the dam. The available materials and its excavation costs dictate the type of dam. Sufficient quantities of both pervious and impervious materials leads to the choice of zoned dam. The thickness of the impervious core will depend mainly on the quantity of materials available and the relative cost of materials available. The relative cost of the core and shell material will also be influenced by the thickness of the impervious core. Since, the shear strength of the core material is generally much less than that of the available shall materials therefore, for the purpose of stability a core thickness of about two times the reservoir head is usually provided, where, the upper limit implies maximum slope of about 1 : 1 on both faces of the core (Sharma, 1991)

7.6 DAM EMBANKMENT TYPE AND ZONING

Zones of dams must be developed in conjunction with foundation seepage control requirement because, their performance heavily depends on these structures functioning as a complete embankment-foundation systems.(Cendergren, 1977). Therefore, it is proposed to construct Rib dam with 1) central impermeable core, 2) outer shell of pervious material, 3) longitudinal drains and blankets, 4) internal chimney drains, 5) upstream riprap 6) and a rock toe just at the downstream toe of the dam. Figure 7.1 explains the various zones proposed for the Rib dam.

7.6.1 Clay core

Location and accessibility

Borrow area for the embankment construction material is located about 3 km, down stream of the dam site, on way to Zeha village . The borrow area is located on the eastern side of the Rib river and is defined by the co-ordinates 12° 03' 55" N and 37° 59' 00" E. At present no motor able road is present to the borrow area site.

Present land use in the borrow area

At present the proposed borrow area is used as cultivation and grazing land. For construction use of the soil in the borrow area priority should be given for the grazing land since, the soils in this area reveals more appropriate property for the intended use. In addition, the cultivated land should be used for production of construction material, if and only if, the amount of the required soil from the grazing land is not sufficient.

Quantity of expected borrow area soil

The quantity of embankment soil to be produced from the proposed borrow area both from the grazing land and the cultivated land is sufficiently enough for this dam construction as estimated from the soil thickness and area coverage. The test pits dug during the present investigations reveals, the average depth of the excavated soils is 6m having three soil horizons, namely dark brown silty clay of average thickness 1m, followed by yellowish brown sandy silt having an average thickness of 2m and brown silty clay of average thickness 3m. The soil horizons are observed from nine test pits and three parallel aligned deep gullies. From the test pits and gully cuts a total of four samples were collected for insitu dry density determination. Further, this data was used to estimate the quantity of soil to be excavated. The field density has been determined on moisture content higher than compaction moisture obtained in the laboratory. At the time of field testing the soils were saturated due to occasional rains, therefore the density of the soil as determined is for saturated conditions. Thus, the estimated quantity of soils available from the borrow site for construction purpose will be less.

__Quality of the core material__

The gradation analysis results of the clay which are used to designate the various components of the soil by grain sizes show an appreciable amount of sand and silt sizes. On the other hand the plasticity index in combination with liquid limit classified the soil to be CI and CH under Unified Soil Classification systems (Fig. 7.2).

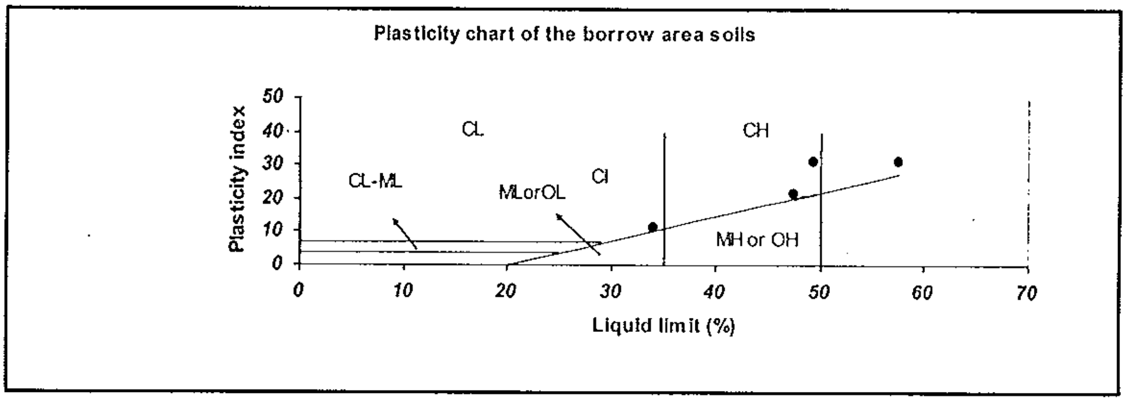


Fig.7.2 Classification of the borrow area soils (USCS)

Activity number (ratio of plasticity index to clay fraction) of more than 1.4 are indicator of swelling characteristics of clay minerals but the ratio in the borrow area lies 0.63 and 1.41 except, for one sample out of four, which might be a testing error. However, due to the limitation of time re-sampling and re-testing could not be conducted. Hence, from the present test data the soil of the borrow area are classified as inactive and normal.

The swelling potential value reveals that the soil deposit has variable swelling characteristics (Table 5.4 Chapter 5); out of four tests conducted on borrow area soils, the result shows low medium and high swelling potential.

According to the Abay Basin Master Plan (1996), the chemical analysis of the soils of the area shows that the average exchangeable sodium percentage (ESP) is 3.6 for the sub soil and the maximum and minimum values being 8.4 and 0.4, respectively. Soils with an ESP of 7 - 10 are moderately dispersive and may suffer piping when the reservoir water has low content of dissolved salts and soils with an ESP of > 15 are at serious risks of piping (Bharat Singh, 1995).

Based on the average values of ESP the soils of the borrow area are slightly dispersive. High ESP values and piping potential are generally found in montimorilite clay fractions (Bharat Singh and Varshney, 1995). The activity of the montimorillonite clay range is between 1 and 7 whereas, the activity of the borrow area soils in the present study area is in the range of 0.63 to1.4. Thus, high ESP is not expected in this soil deposit of the borrow area.

The other requirements of core materials are that it should provide the necessary degree of imperviousness and it should with stand differential settlements with out cracking and should be non erodible in event of cracking.

The ideal clay core material is well graded and moderately plastic clay. As this type of clay is impervious and has high shear strength. In addition, it exhibits low compressibility and may be compacted to high density (Sharma, 1992). A total of 6 soil samples has been taken from the clay core borrow area for testing. The test has been carried out for grain size proportion, plasticity characteristics and shear strength parameters. The result thus obtained, are presented in the Table 7.1

Table 7.1 Test results for Borrow area Soils

Soil type	Atterberg Limits			USCS	γd	O.M.C	Shear strength	
	LL	PI	Ip				C (N/m2)	Φ (degre)
Clayey silty sand	34.0	22.96	11.04	CL	2020	14	4	39
Sandy silty clay	49.3	17.9	31.4	CI	1540	23	22	31
Sandy clayey silt	47.5	26.17	21.33	CI	1550	21	38	22
Clayey silty sand	57.6	26.6	31.0	CH	1490	17	36	25
Sandy silty clay	-	-	-	-	1820	16.18	10	30
Silty clay	-	-	-	-	1.58	26.73	33	9

According to Sherard et al (1967) classification of core materials on the basis of capabilities of restriction to concentrated leaks, the soils of the proposed borrow area can be classified as good material since the soil deposit has well graded coarse mixture of sand gravel and fines with plastic index greater than 12 (Table 7.1).

From the gradation curve of the borrow area soils (Figure 5.1, Chapter 5) it may be seen that the core material grain size with corresponding percentage for the filter design are D15 <0.001mm, D50 = 0.06 mm and D85= 0.5 mm.

7.6.2 FILTER AND DRAIN

Filters and drains can provide permanent security against the damaging section of seepage and ground water however, certain fundamental requirements must be strictly enforced. If filters and drains are required to serve their intended purpose, the materials used in their construction must have the correct gradation and they must be handled and placed with care to avoid contamination and segregation (Cendergren, 1977);

Piping requirement

The pore spaces in drains and filter that are in contact with erodible soils and rocks must be small enough to prevent particles from being washed through them.

Permeability requirements

The pore spaces in drains and filters must be large enough to impart sufficient permeability to permit seepage to escape freely and thus, provide a high degree of control over seepage forces and hydrostatic pressures. When small quantities of seepage are to be removed a single layer of well-graded, moderately permeable material meeting both requirements may some times serve the dual roles of filter and drain, but when large quantities of seepage are to be removed a filter layer is usually needed for the prevention of piping and a coarse layer, for the removal of water (graded filters).

Location and property of filter materials

The source of filter and drains is proposed from the Rib river alluvial deposit, located down stream of the dam site about 1.5 km from the site. In order to evaluate the filter material requirements representative samples has been collected from the sand deposits of the river. Laboratory tests has been carried out on grain size analysis (Fig. 7.3) ,bulk density and specific gravity. According to the gradation analysis test result the important grain sizes which influence the suitability of the filter materials are derived, these grain sizes are D15 and D50 and the grain sizes with corresponding percentage are 0.3mm and 0.6mm, respectively.

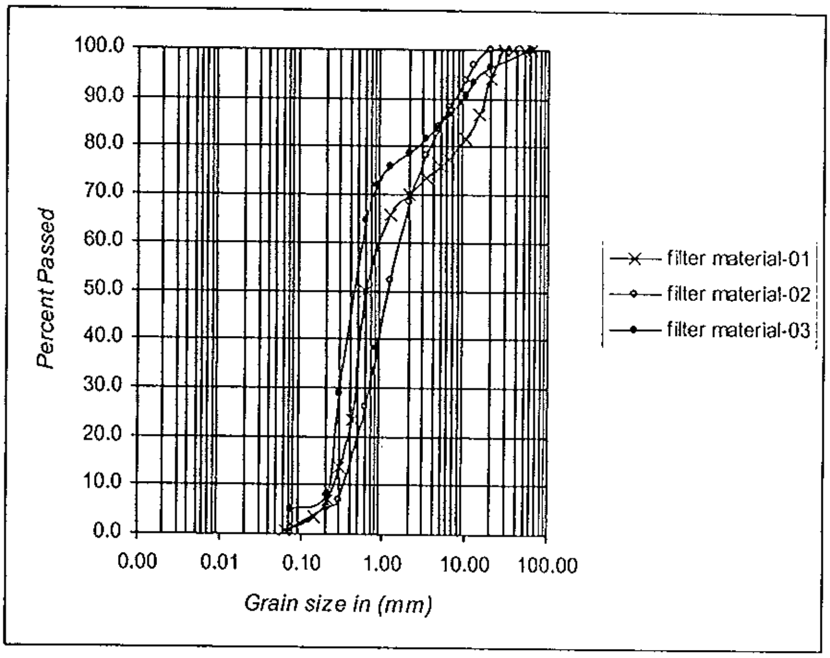


Fig.7.3 Gradation curve of the filter and drain material

Criteria For Filter Selection

For the present study different filter criteria have been tested for the available filter materials in the Rib river alluvial deposits. These filter criteria are discussed in the following paragraphs;

Terzaghi filter criteria

For the design of filters Terzaghi (1930) proposed the following criteria;

$$\frac{D_{15}(f)}{D_{85}(soil)} < 4 < \frac{D_{15}(f)}{D_{15}(soil)} \quad \ldots\ldots\ldots\ldots \quad (7.1)$$

The left half of the equation (7.1) may be defined as - The 15% size (D_{15}) of filter material should be at least four or five times the 85% size(D85) of protected soil (Criteria-1). The intent of this criteria is to prevent movement of erodible soils through filters. Hence, the pore spaces between the filter particles should be small enough to hold the protected materials in place.

The right half of equation (7.1) may be defined as - The 15% size (D15) of filter material should be at least four or five times the 15% size (D15) of protected soil (Criteria-2).

The intent of this criteria is to guarantee sufficient permeability to prevent the build up of large seepage forces and hydrostatic pressure in filters and drains.

The Indian Standard Code

The recommendation for filter materials selection as per the Indian standard code is;

a) $\frac{\text{D15 of filter}}{\text{D85 of base}} < 5$ ---------- 7.2

b) $\frac{\text{D15 of filter}}{\text{D15 of base}} > 4 \text{ and } < 20$ 7.3

c) $\frac{\text{D50 of filter}}{\text{D50 of base}} < 25$ ----------- 7.4

d) The gradation curve of filter material should be nearly parallel to the gradation curve of the borrow material (Fig.7.4).

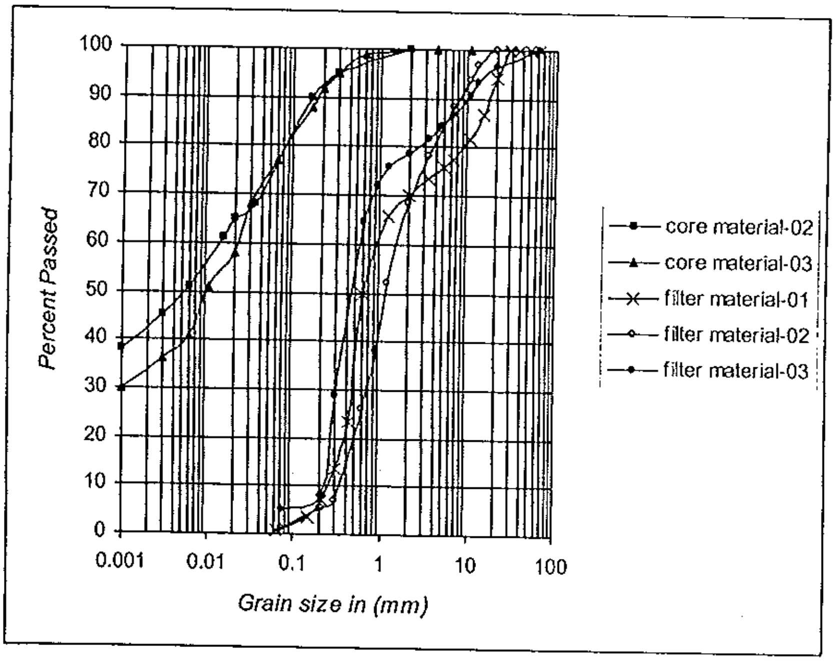


Fig. 7.4 Gradation curve for filter material and the borrow material

U.S.Army Corps of Engineers (1955) Criteria

According to the US.Army corps of engineers recommendation for the filter criteria are;

$$\frac{D15\,(F)}{D85(P)} \leq 5 \qquad \ldots\ldots\ldots\ldots 7.5$$

$$\frac{D50\,(F)}{D50\,(p)} \leq 25 \qquad \ldots\ldots\ldots\ldots 7.6$$

It can be seen that eq. 7.5 is an other expression of the relationship given by eq. 7.1

Table 7.2 Summary of different criteria for filter selection

Criteria used	Requirement	Result	Remark
Terzagi's criteria	$\frac{D15(F)}{D85(P)} < 4 < \frac{D15(F)}{D15(P)}$	0.6< 4<300	Criteria is satisfied
U.S. Army corps of Engineers(1955)	$\frac{D15(F)}{D85(P)} \leq 5$ and	$0.6 \leq 5$	Criteria is satisfied
	$\frac{D50(F)}{D50(p)} \leq 25$	$10 \leq 25$	Criteria is satisfied
I.S.Code 9429-1980[22]	$\frac{D15(F)}{D85(P)} < 5$	0.6 < 5	Criteria is satisfied
	$\frac{D15(F)}{D15(p)} > 4$ and <20	300 >4 and <20	not satisfied
	$\frac{D50(F)}{D50(p)} < 25$	10 <25	Criteria is satisfied
Sherard's criteria	$\frac{D15(F)}{D85(P)} < 9$	0.6 <9	Criteria is satisfied

According to the analysis result described in the tablet he proposed filter material has satisfied the filter selection criteria almost all the criteria mentioned above.

Sherard et.al (1972a) point out that piping failures are likely to occur in certain types of clay which erodes by a process called "dispersion or deflocculating" when the clay mass is in contact with water, individual clay particles are detached from the surface progressively and moves into suspension. If water is flowing the dispersed particles are carried away and erosion channels or pipes can form quite rapidly. Dispersive clays can not be recognized by the identification tests by the above mentioned criterias used in engineering practice. However, in the present study area no such soils are present.

7.6.3 Riprap and Rock Fill Materials for Shell

Rock fragments are required in connection with earth work structures for the protection of earth embankments or exposed excavations from the action of water either as wave, turbulent flow or heavy rain fall. The rock fragment protective work associated with wave action or flowing water is designated as riprap whereas, the rock fill materials which are more massive bodies of fill in the dam embankments consisting of rock fragments which are used primarily to provide structural stability and serve as drainage blankets.

Rock Fill Materials

The basic requirements for rock fill materials used for the construction of dam shell materials are soundness, sufficient permeability and shear strength. When rock fill materials are used for this purpose it should be sound not liable to disintegration by weathering, nor it should split or crush under the loads to which it will be subjected. In general granites, diorites, gneisses basalts, dense sandstones, lime stones, dolomite, quartizites and massive schists are likely to prove satisfactory. In the absence of rock out crops for rock fill quarry sites river deposits comprising sand gravel and boulders can be used as satisfactory dam shell materials (Sharma,1991).

For the construction of Rib dam the source of rock fill material is proposed from compulsory excavations and rock quarry located on the ridges surrounding the abutments . The geology of the reservoir and dam site indicates that rocks in the immediate vicinity of the dam are slightly to moderately weathered aphenetic basalts which can be used for this purpose. Moreover, considerable amount of river bed deposit consisting of sand, gravel, cobble and boulder size rock fragments are also available in the Rib river channel which can also be utilized for the said purpose.

Riprap Materials

Riprap surfaces on earth dams have to withstand very sever action from the waves generated in the reservoir and the destructive forces associated with temperature changes. Securing materials for this purpose is therefore most desirable. No general statement can be made as to whether one type of rock is better than other for slope protection except sedimentary rocks containing clay. In selecting riprap material size of rock fragments is very important, fragments up to one cubic yard in volume are required on dams associated with large reservoirs. In the immediate vicinity of the Rib dam site bed rock exposures containing satisfactory rock that can be developed into quarry is not available because of the prevailing close spaced joint systems which could not produce sufficient size of rock fragments. However, it is possible to secure riprap material by using alluvial rock fragments from the river bed deposit and from scattered field boulders from the upstream left ridges.

CHAPTER VIII

RESULTS AND DISCUSSION

8.1 Dam Foundation Condition

Based on the field investigations and the laboratory testing results on the foundation condition an overall appraisal has been made. In order to overcome the various problems associated with the dam foundation suitable remedial measures have been worked out and are discussed hereunder.

i) The moderately to highly jointed aphanetic basalt out cropped on the right and left abutments of the dam has to be excavated and removed upto the level where comparatively fresh rock with less joint intensity is encountered. In order to work out a stripping limit on the abutments it is recommended to make exploratory drifts or inclined drilling along the dam axis. The exploratory drifting or inclined drilling may be planned as per the plan shown in the cross section (Fig. 8.1).

ii) Further, underlying the aphenetic basalts relatively fresh jointed rock is present which has to be grouted since it may provide possible seepage path through the interconnected joint systems. According to Ewart (1985) relation the depth of curtain grout can be estimated as :

$$D = 1/3*H+C \qquad \text{............ 8.1}$$

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

Based on the foundation condition results obtained from bore holes along the dam axis the foundation condition is in boundary condition. Therefore, taking 16.5 as average value for C, the depth of grout curtain (D) for a height of 50 m dam, is around 34m. The coarse granular river channel deposit has a depth of 27.18 m at the dam axis and extends upstream and downstream with an average width of 50m. This deposit is pervious and can pose leakage problem through the dam foundation. In order to prevent seepage through this zone following remedial measures are suggested;

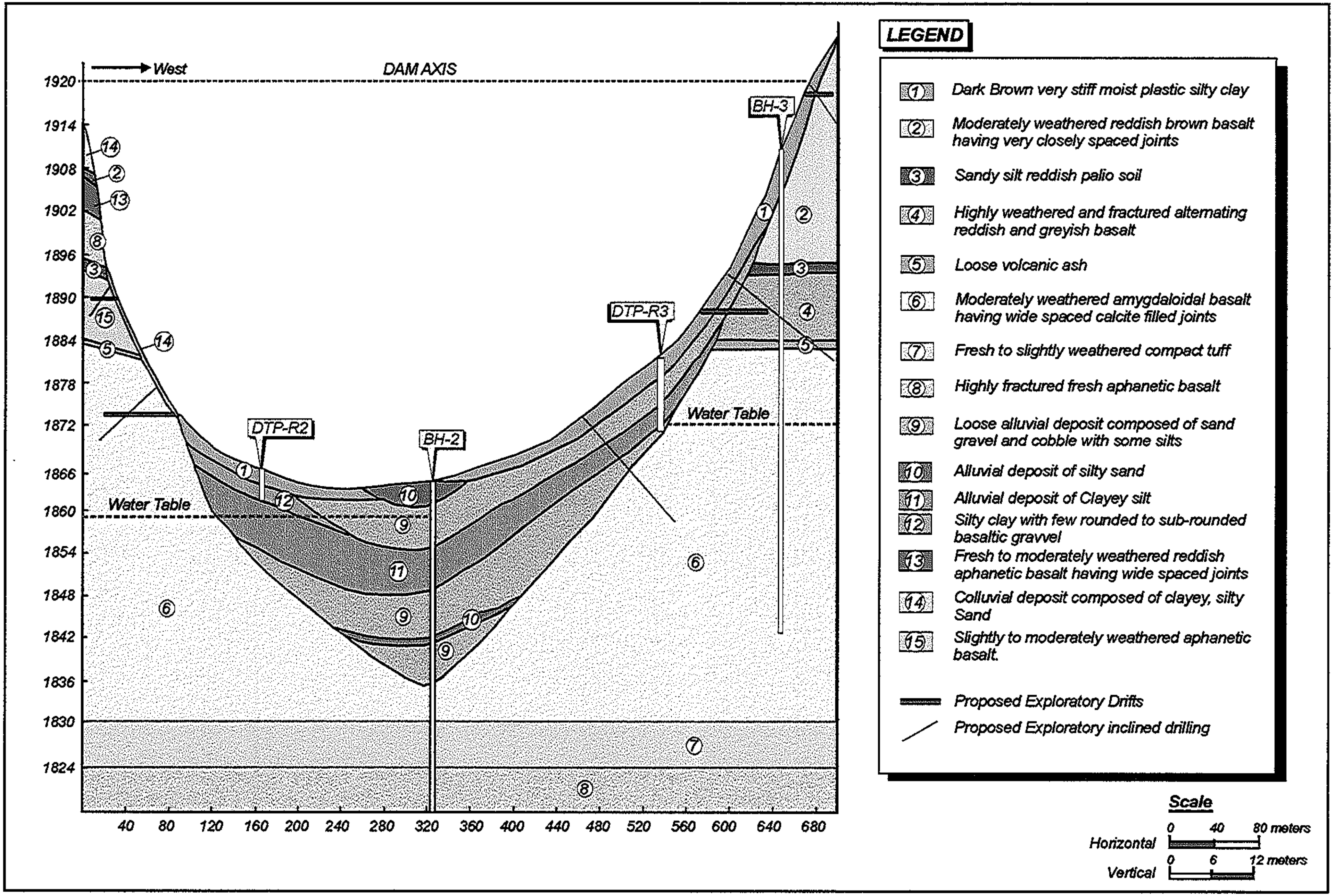


Fig. 8.1 Geological Cross section at Dam Axis

Positive cut off trenches

At the rib dam site it has been found through the borehole data and the permeability tests that the foundation rocks are permeable. Seepage through this pervious foundation may be cut off by a trench extending to the bed rock and can be back filled with impervious material. This is the most positive means of controlling seepage and ensuring that no problems will be encountered either from piping through the foundation or heaving due to excessive seepage pressure at the down stream of the toe. In order to provide a sufficient thickness of impervious material and to ensure adequate contact with the bed rock or other impervious stratum, the bottom width of the cut off trench should increase with the heads, subject to a minimum of that giving a hydraulic gradient of not steeper than 0.5 (H) 1 (V). A minimum width of about 6m may be provided to enable smooth operation of excavation and compacting equipment. A filter should also be provided on the downstream face of the trench to prevent trench migration of fines into the coarse foundation material.

The cut off trench is a preferred foundation treatment where the thickness of the material to be excavated is not more and where reasonable measures can be used to handle ground water problems in the pervious stratum. On the other hand this method has the advantage to carryout construction with conventional equipments and to inspect the foundation directly and to treat as required.

Partial cut off trenches

Studies have shown that partial cut of extending to a depth of 60% of the pervious stratum will reduce the seepage by only 25% and 80% cut off is required to reduce the seepage by 50% . Even with 90% cut-off, the opening would permit about 38% of the seepage to pass through.

A partial cut-off may be effective in stratified foundations by intercepting relatively more pervious layers in the upper portion of the foundation, hence in Rib dam case the conditions do not work for partial cut-off (Fig. 8.2).

Upstream impervious blanked

When a positive cut-off to the underlying impervious stratum is expensive and some leakage is permissible, or when the depth to the underlying impervious stratum is

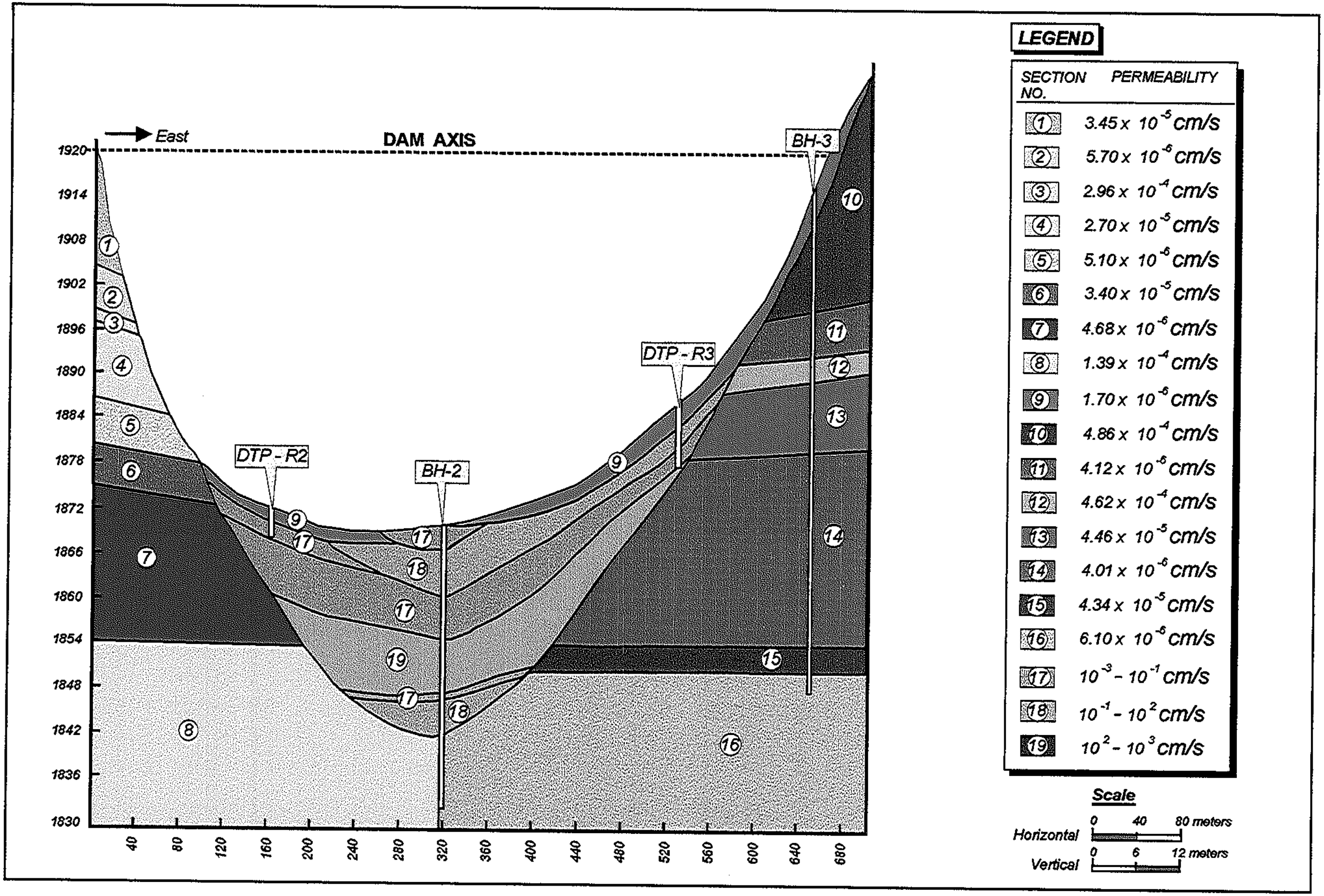


Fig. 8.2 Cross section at Dam Axis showing permeabilty of the foundation material.

more and a positive cut-off is not feasible and the foundation permeability is moderately high, reduction of seepage and over all seepage gradients may be accomplished by construction of an upstream impervious blanket. To minimize the possibility of subsurface erosion, the distance from the upstream end of the blanket to the downstream end of the impervious base of the dam should be at least 15 times the head difference between these two points. To control the exit gradients drains of relief wells and or drainage blanket are provide at the down stream of the dam .

Sheet pilling cut –offs

Sheet pilling is occasionally used in combination with partial cut-off trenches as a comparatively economical means of increasing the depth of the cut- off and under certain conditions it may be used in lieu of cut-off trench. Such piles should be still, because of its high strength sheet cut- off piles are practically limited to in foundations of silt, sand, and fine gravel where cobbles or boulders are present or where the material is highly resistant to penetration driving becomes difficult, hence not recommended in the case of Rib dam. This option does not work because of the presence of boulders, cobles and gravels in the stratum.

Grouted cut off

Grouted cut-offs are produced by filling the sediment voids within the cutoff zone with cement, clay, chemicals or combination of these materials. Grouted cut- off are generally effective when seepage occurs primarily through pockets and zones or layers of coarse material. Impermeability can be achieved with low cost materials, such as mixture of clay, cement and bentonite. Coarse sediments with insitu permeability of 10^{-1} cm/sec can generally be treated effectively with clay- cement grout whereas, Chemical grouts are used for fine grained soils. At Rib dam site the river deposit comprises of mainly gravel to cobble size deposition in a matrix of coarse to fine sand and having a permeability range between 10^{-3} to 10^{2} cm/sec (Fig.8.2). Therefore, the grout cut off may be provided in the river bed section.

8.2 Foundation strength and bearing capacity

At Rib dam site, both flanks of the river and the central valley floor is covered with highly compressible black cotton clay. The average thickness of compressible clay layer is about 3.75 m. When the foundation of an earth dam consists of saturated fine grained soils or saturated impervious sands their ability to resist the shearing stress imposed by the weight of the embankment may be determined by their soil group classification(USBR, 1987). Based on the classification test result the soils of the Rib dam site are dominantly highly compressible in organic clays (CH) having liquid limit greater than 50. According to their group classification the soils suitability falls in fair to poor range.

Therefore, in order to reduce the soil compressibility (reduce foundation settlement) or to increase the strength (in order to improve structural stability or bearing capacity) ground improvement is required.

The simplest and the most effective method to reduce the soil compressibility and the settlement of earth dam foundation is to provide shallow compaction. This may increase the density of the foundation soil. For this purpose any type of roller may be employed.

The best mitigation method for the adverse effect of compressible soils is to avoid founding the structure on compressible soils. One of the effective method is to remove entire compressible soil and replacing it with non compressible soils. However, at Rib dam site the thickness of the compressible soil is more, therefore it may not be feasible to remove the compressible soils in the foundation area.

Other alternative may be to reduce settlement of earth dam foundation on compressible soil and to provide broader and flatter slope for the embankment, so that the dam base area is increased and the loads imposed by the dam are distributed over a wider area. However, flatter and broader slopes increase the volume of embankment and which will result in more construction material. Thus, the cost for construction and time will increase. Therefore, this option may be adopted after working out the techno-economic feasibility.

8.3 Reservoir Area water tightness

The two major factors which control the retention of water in reservoirs are the piezometric conditions in, and the natural permeability of the base and flanks of the basin (Kennard and Knill, 1969).

The presence of a reservoir changes the ground water system over an area that is much larger than that occupied by the reservoir it self. If the maximum height of the phreatic surface in the flanks of the reservoir after impounding exceeds the top water level, then there should be a net flow of ground water towards the reservoir. On the other hand if the ground water table in the flanks is depressed with respect to the reservoir top mater level then deep seepage flow will take place from the reservoir to an adjacent catchment.

The present study on Rib dam doesn't cover the ground water condition study of the reservoir area and its flanks. However, the boreholes drilled for dam site investigation reveals that the water table level at the abutments is located below the proposed top water level of the reservoir. Further the permeability of the rock, as obtained from the packer test result, shows that the rock mass is in boundary condition between the semi permeable and permeable. However, the rock mass in the reservoir area is covered with impervious soils except at the top periphery of the reservoir and river channel. Therefore, the impervious soil blanket over the reservoir basin may prevent seepage of the reservoir water to the adjacent catchment by itself.

8.4 Construction materials

The present study on construction material shows that the proposed borrow area soils is suitable for the intended purpose with no physical and chemical properties limitation. Two types of soils has been identified in horizontal layers in the borrow pits , they can be either excavated selectively and placed in different zones of the dam or excavated together and blended in to a single material with intermediate properties. The properties of such blended mixture can be controlled by varying the excavation procedure to obtain different percentage of the two soil.

Further, for construction material two borrow area options are also proposed one from grazing land and the other from existing cultivated land. The soil deposit from cultivated land is predominantly clay. The grazing land soil comprises of well graded sand, silt and clay with high compaction density, with relative lower optimum moisture content. It is also possible to prepare an embankment material of intermediate property by mixing two materials from the two borrow areas, by blending. However, for embankment stability and detail design physical property tests such as shear strength parameters must be determined for the blended sample, separately.

CHAPTER IX

CONCLUSION AND RECOMMENDATIONS

The geological and engineering geological evaluation of Rib dam project has been conducted by characterizing soils and rocks of the study area with much accuracy and in an extensive manner, supported with important design data generated during the present study. However, all these efforts were made under the limitations on resources, time and financial constraints. The present study may serve as an effort to highlight some of the problems related to dam foundation conditions and the problems related to the reservoir area. Therefore, the present study may serve as an first hand information to the planners and developers, based on which detailed investigations may be planned and undertaken for the safe design of the dam.

An attempt has been made to conclude the findings and based on the results some recommendations are being made which are expressed systematically in the following paragraphs.

At dam foundation area, both flanks of the river and the central valley floor is covered with highly compressible black cotton clay. The average thickness of compressible clay layer is about 3.75 m. Based on the classification test result the soils at the foundation area are dominantly highly compressible in organic clays (CH) having liquid limit greater than 50. According to their group classification the soils suitability falls in fair to poor range. Therefore, in order to reduce the soil compressibility or to increase the strength ground improvement is required.

The simplest and the most effective method to reduce the soil compressibility and the settlement of earth dam foundation is to provide shallow compaction. This may increase the density of the foundation soil. For this purpose any type of roller may be employed.

Other alternative to reduce settlement of earth dam foundation on compressible soil is to provide broader and flatter slope for the embankment, so that the dam base area is increased and the loads imposed by the dam are distributed over a wider area. However, flatter and broader slopes increase the volume of embankment and which will result in more construction material. Thus, the cost for construction and time will increase. Therefore, this option may be adopted after working out the techno-economic feasibility.

The deep pervious alluvial deposit encountered at the river course area at dam foundation, can cause concentrated leakage of the reservoir water through the foundation. This may be treated by providing positive cut off wall, which extends up to the bed rock. However, the bed rock is at a depth of around 27 m which is comparatively very deep. Therefore, a better option could be to provide partial cut off with upstream clay blanketing. Further, the clay blanketing, to a thickness of 2.25 m, should be extended all along the river channel section upto an upstream length of 100m.

Further, at the abutment areas the impervious soil blankets are shallow and the underlying rock mass is affected by interconnected joint systems which may cause seepage of reservoir water through the abutment rock mass. Hence, there is a need to improve the foundation conditions in the abutment section. The moderately to highly jointed aphanetic basalt out cropped on the right and left abutments of the dam has to be excavated and removed upto the level where comparatively fresh rock with less joint intensity is encountered. In order to work out a stripping limit on the abutments it is recommended to make exploratory drifts or inclined drilling along the dam axis.

Further, underlying the aphenetic basalts relatively fresh jointed rock is present which has to be grouted since it may provide possible seepage path through the interconnected joint systems. According to the present study a grout curtain upto a depth of 34m will be required to make the foundation rocks free from any seepage.

In the reservoir area, even though, there is a natural blanket of impervious soils of considerable thick ness, however at the river course and periphery areas the natural blanket is either shallow or not present. Therefore, the river course and the periphery of the reservoir area should be blanketed with a minimum thickness equal to 5% of the hydraulic head at that level.

To determine the water tightness of the reservoir, ground water level and flow direction bore holes located at appropriate locations are required but for the present study bore holes are not available. Therefore, the ground water level and flow direction in the reservoir flank is not determined. Hence, it is strongly recommended to carry out detail study on ground water level and flow direction by providing bore holes at appropriate locations within the reservoir area.

For the present study the shear strength parameters of the embankment clay core material are determined by using direct shear apparatus, however this method has many demerits to use for detail design of the embankment. Therefore for detail design purpose triaxial tests has to be conducted for reliable data.

Finally, it may be concluded that Rib dam site is comparatively suitable site for the zoned dam with minimum engineering geological problems.

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ANNEXURE - I

A Brief outline of SARC computer program

The computer program SARC is prepared by Prof. Bhawani Singh, Department of Engineering, Indian Institute of Techonology. The program (x) is written in Fortran 77 and EXE files work in DOS environment. The users' manual is also indexed as IX.NEW for preparation of input data files. Further, typical input data files are also given as IX.DAT beginning with I. The corresponding out put files OX.DAT are added, beginning with O.

The typical computer commands are:
NE IX.DAT- To open input file
NE OX.DAT- To open out put file
X- Name of computer program
IX.DAT- Input file name
OX.DAT- Output file name
2- For execution
1- For help menu
NE OX.DAT- To see the output file OX.DAT

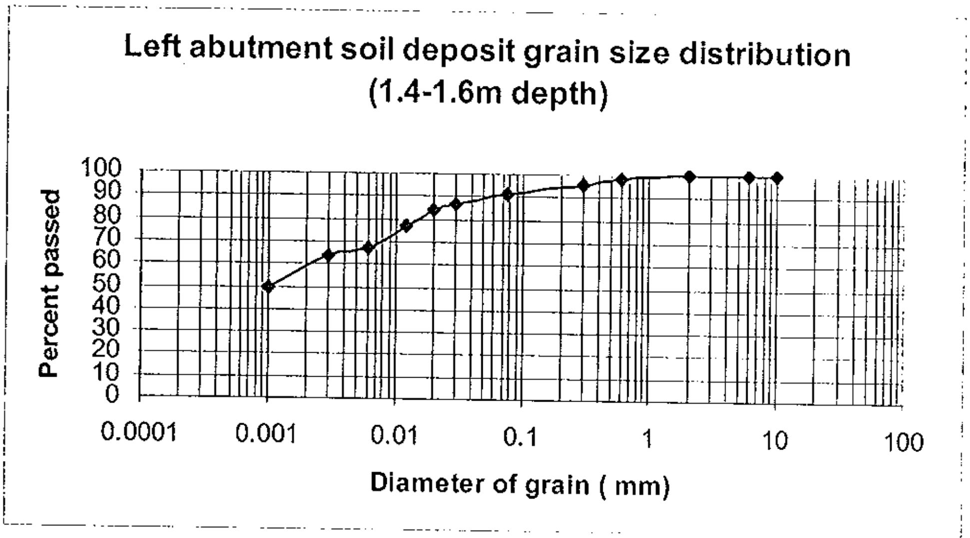
SARC

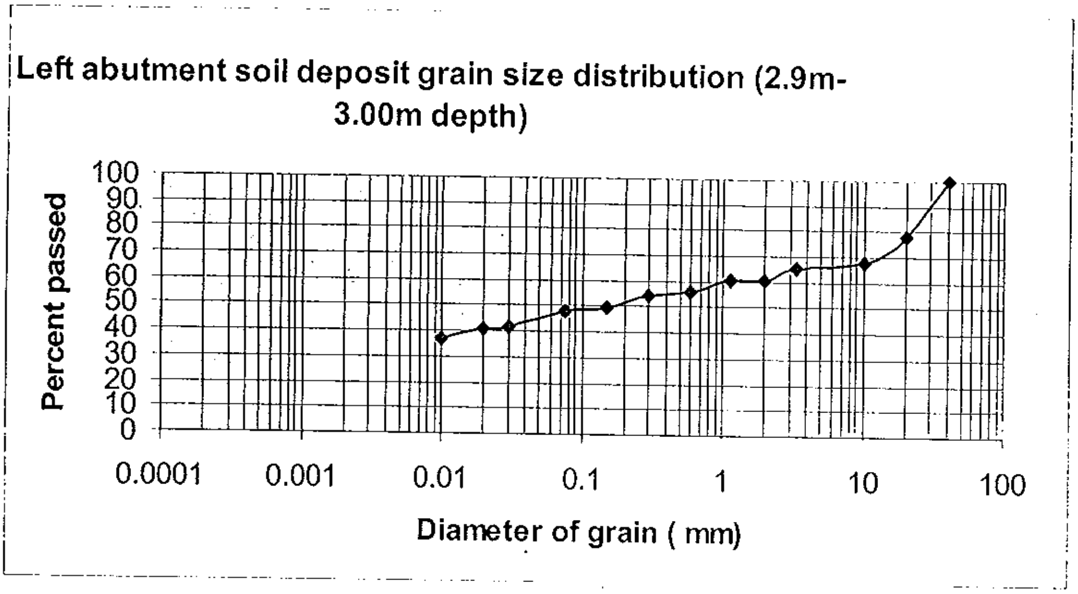
This program facilitates to compute the factor of safety with circular failure surface emerging at the toe. It analyses any general profile of the slope surface and for various forces that is pore water pressure, depth of tension crack at the top of the slope, depth of water in tension crack and earthquake force. In the first step it draws the various slip surfaces along which failure can take place. Then it calculates the radius and center of each slip surface.

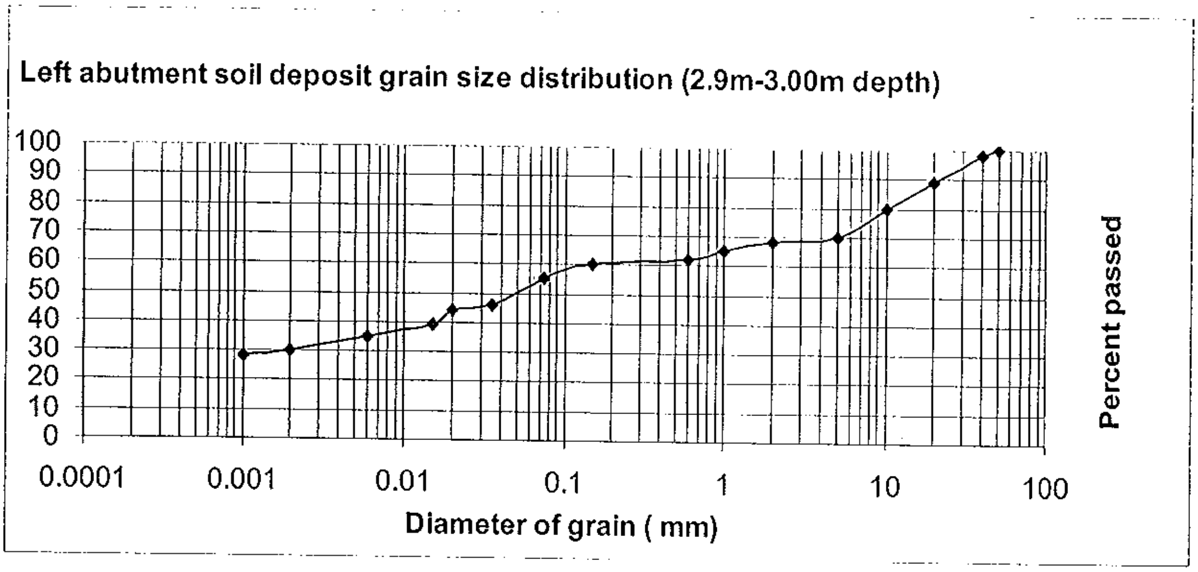
In the next step, the factor of safety is computed using Bishop,s equation for various slip surfaces until a minimum factor of safety is obtained. The analysis evaluates critical acceleration for slopes with factor of safety less than unity and compute dynamic displacement utilizing coorelation developed by Lavania et al. (1987).

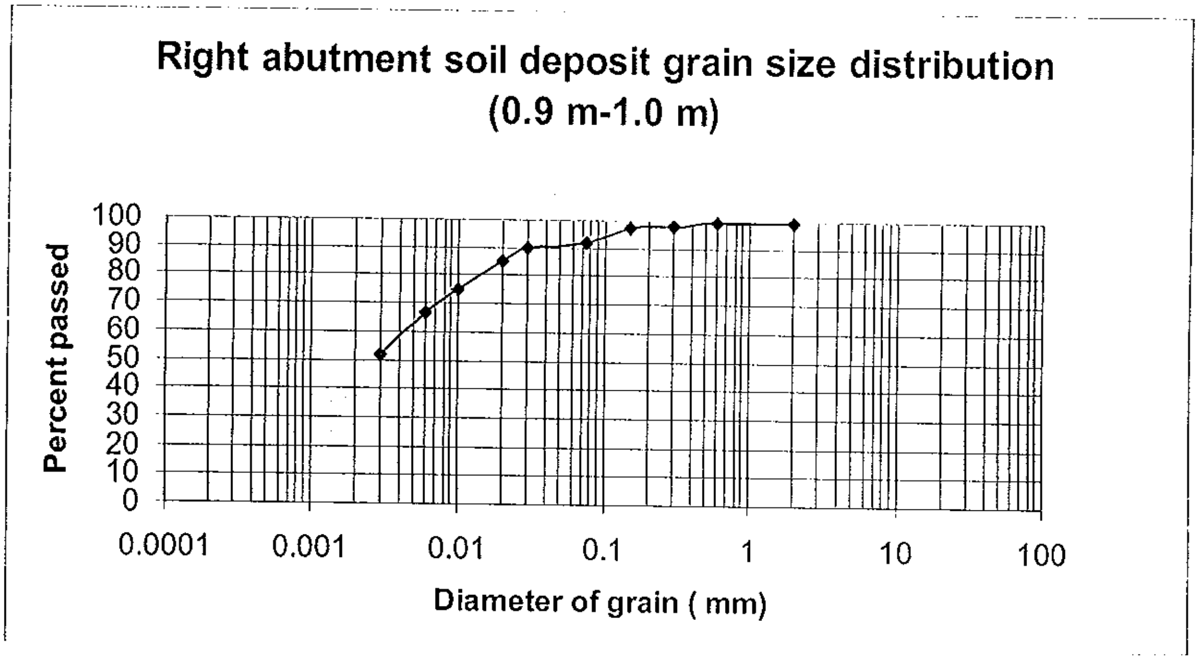
ANNEXURE-II

GRADATION SOILS OF DAM AXIS

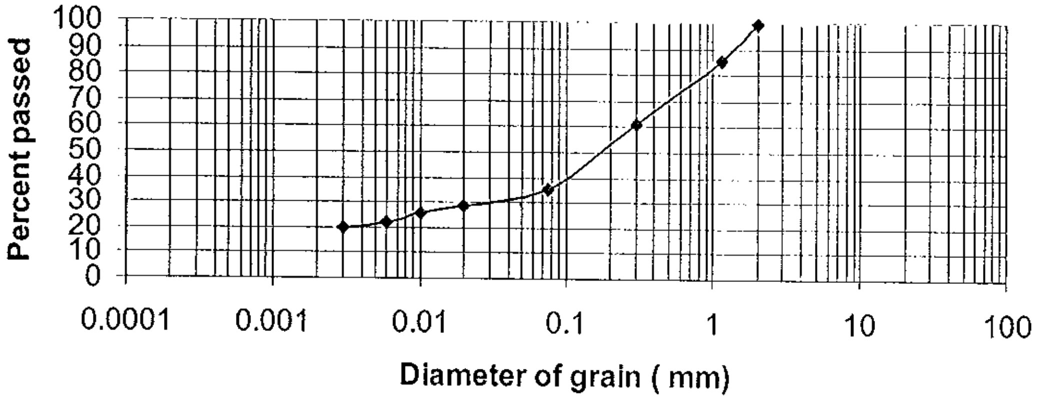




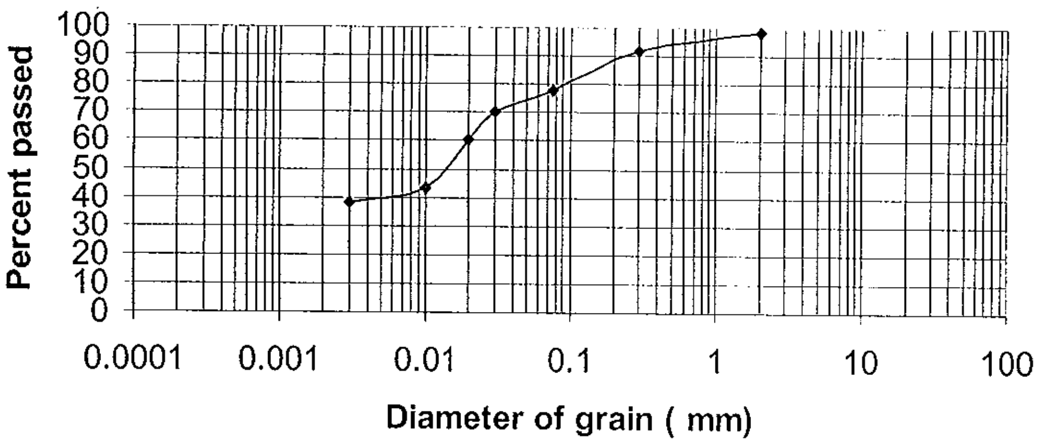

Left abutment soil deposit grain size distribution (2.9m-3.00m depth)
100
90
80
70
60
50
40
30
20
10
0
0.0001
0.001
0.01
0.1
1
10
100
Diameter of grain (mm)
Percent passed

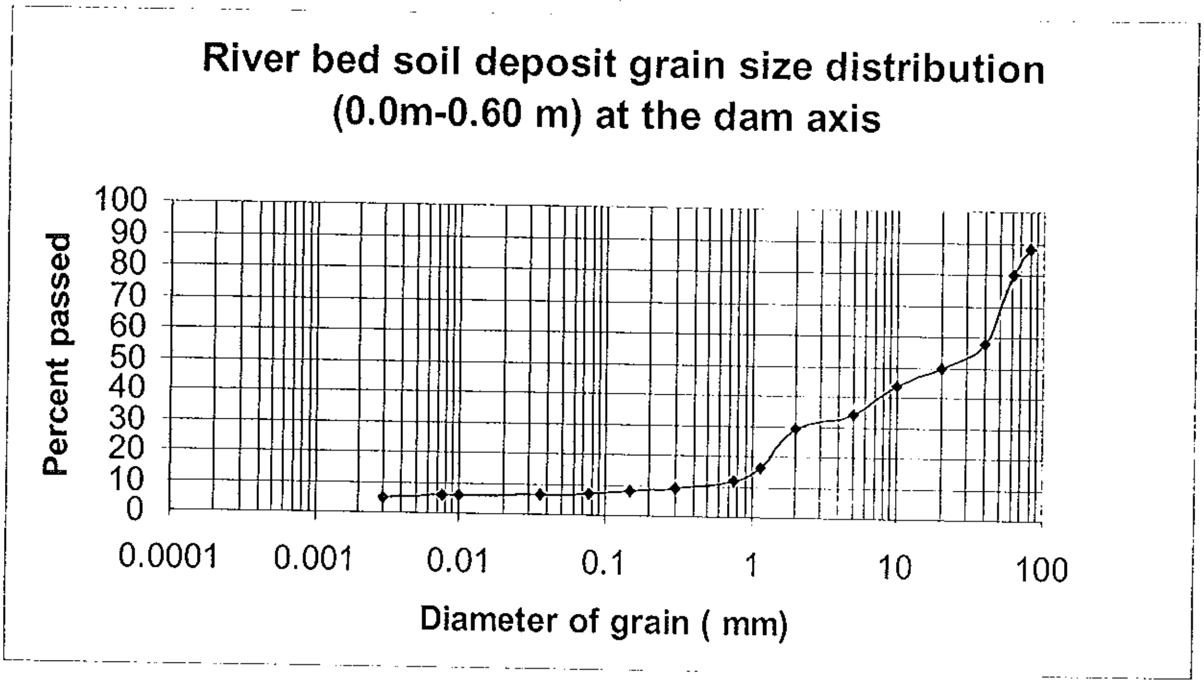

Right abutment soil deposit grain size distribution (0.9 m-1.0 m)
Percent passed
100
90
80
70
60
50
40
30
20
10
0
0.0001
0.001
0.01
0.1
1
10
100
Diameter of grain (mm)

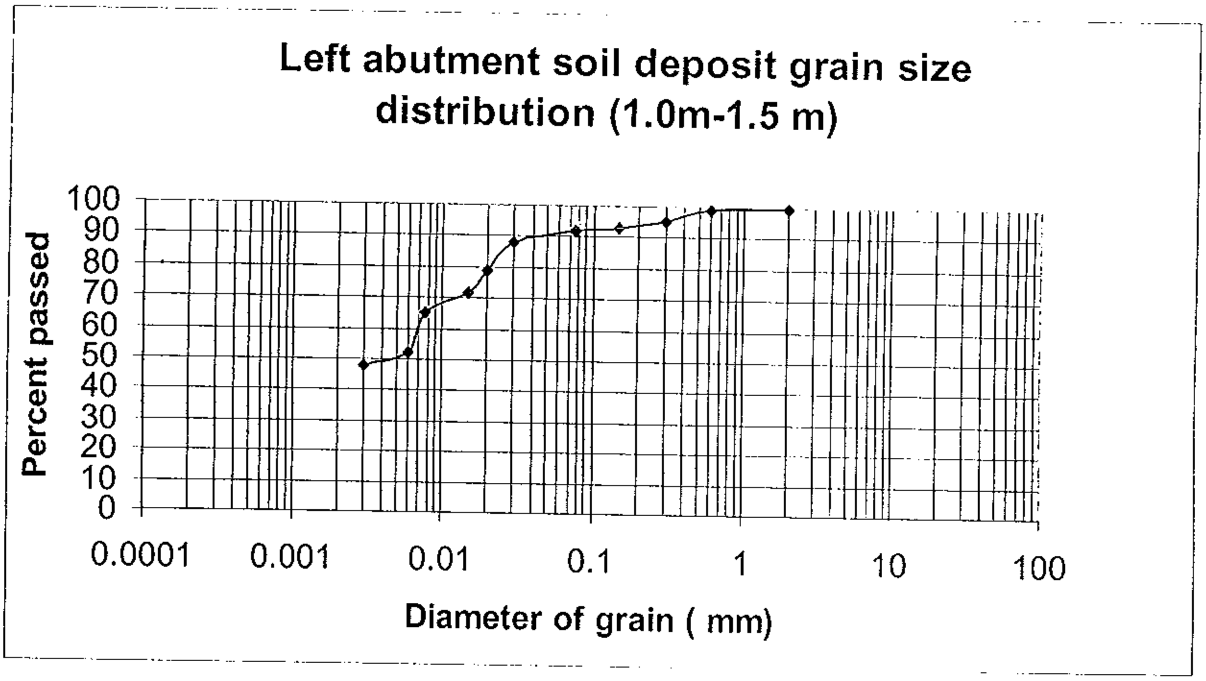
Right abutment soil deposit grain size distribution (4.5 m-4.6m)



Right abutment soil deposit grain size distribution (7.3 m-7.4 m)



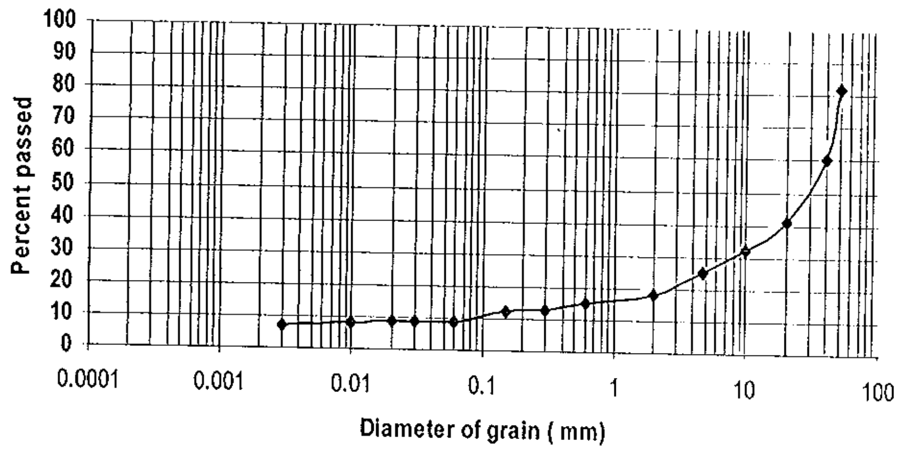

River bed soil deposit grain size distribution (0.0m-0.60 m) at the dam axis
Percent passed
100
90
80
70
60
50
40
30
20
10
0
0.0001
0.001
0.01
0.1
1
10
100
Diameter of grain (mm)


Left abutment soil deposit grain size distribution (1.0m-1.5 m)
Percent passed
100
90
80
70
60
50
40
30
20
10
0
0.0001
0.001
0.01
0.1
1
10
100
Diameter of grain (mm)

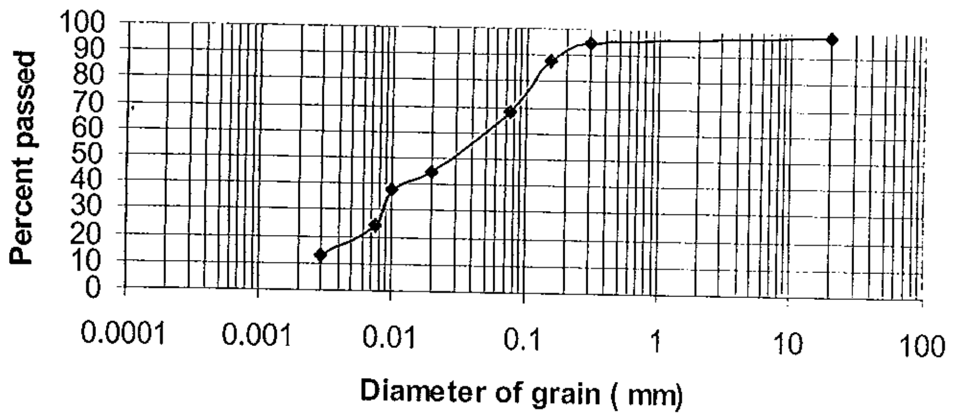
ANNEXURE-III

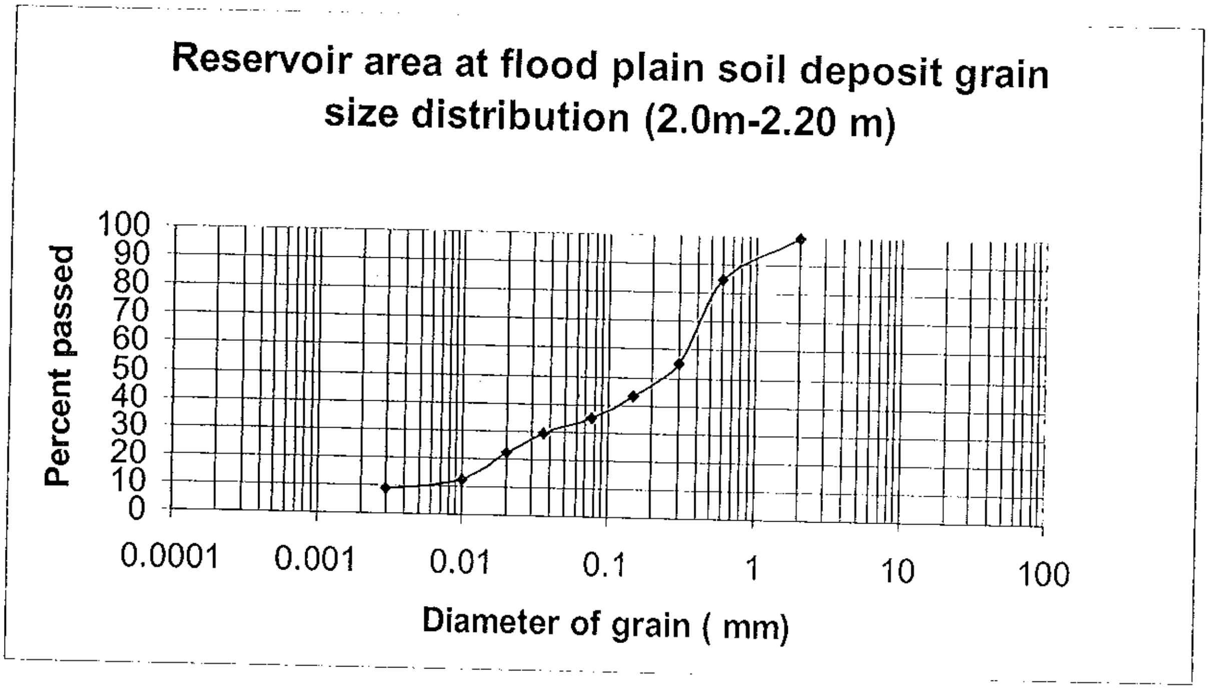
GRADATION OF SOILS OF THE RESRVOIR AREA

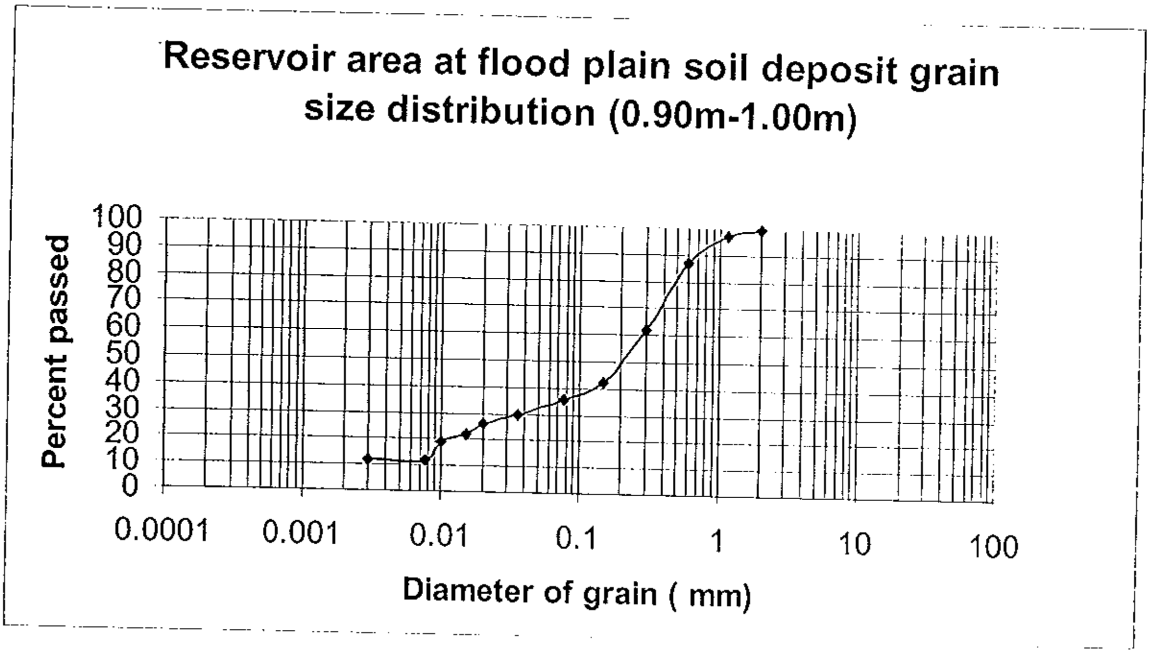
Reservoir area at river bed soil deposit grain size distribution (0.0-1.0 m)

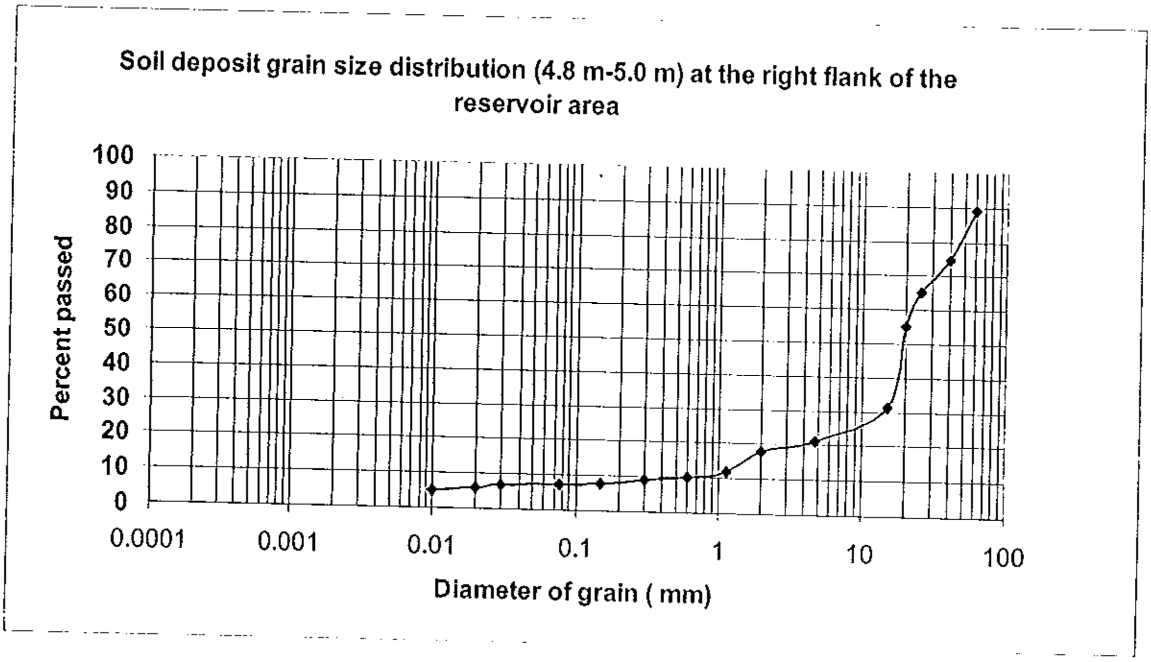


Reservoir area at flood plain soil deposit grain size distribution (1.0 m-1.2 m)




Reservoir area at flood plain soil deposit grain size distribution (2.0m-2.20 m)
Percent passed
100
90
80
70
60
50
40
30
20
10
0
0.0001
0.001
0.01
0.1
1
10
100
Diameter of grain (mm)


Reservoir area at flood plain soil deposit grain size distribution (0.90m-1.00m)
Percent passed
100
90
80
70
60
50
40
30
20
10
0
0.0001
0.001
0.01
0.1
1
10
100
Diameter of grain (mm)


Soil deposit grain size distribution (4.8 m-5.0 m) at the right flank of the reservoir area
100
90
80
70
60
50
40
30
20
10
0
Percent passed
0.0001
0.001
0.01
0.1
1
10
100
Diameter of grain (mm)