ADDIS ABABA UNIVERSITY

School of Graduate Studies

Faculty of Science

Engineeing Geological Studies for suitability of construction Material and Foundation Condition evaluation - with special emphasis on seepage studies, Tendaho Dam , Afar Region,Ethiopia

A Thesis

Submitted to

The School of Graduate Studies

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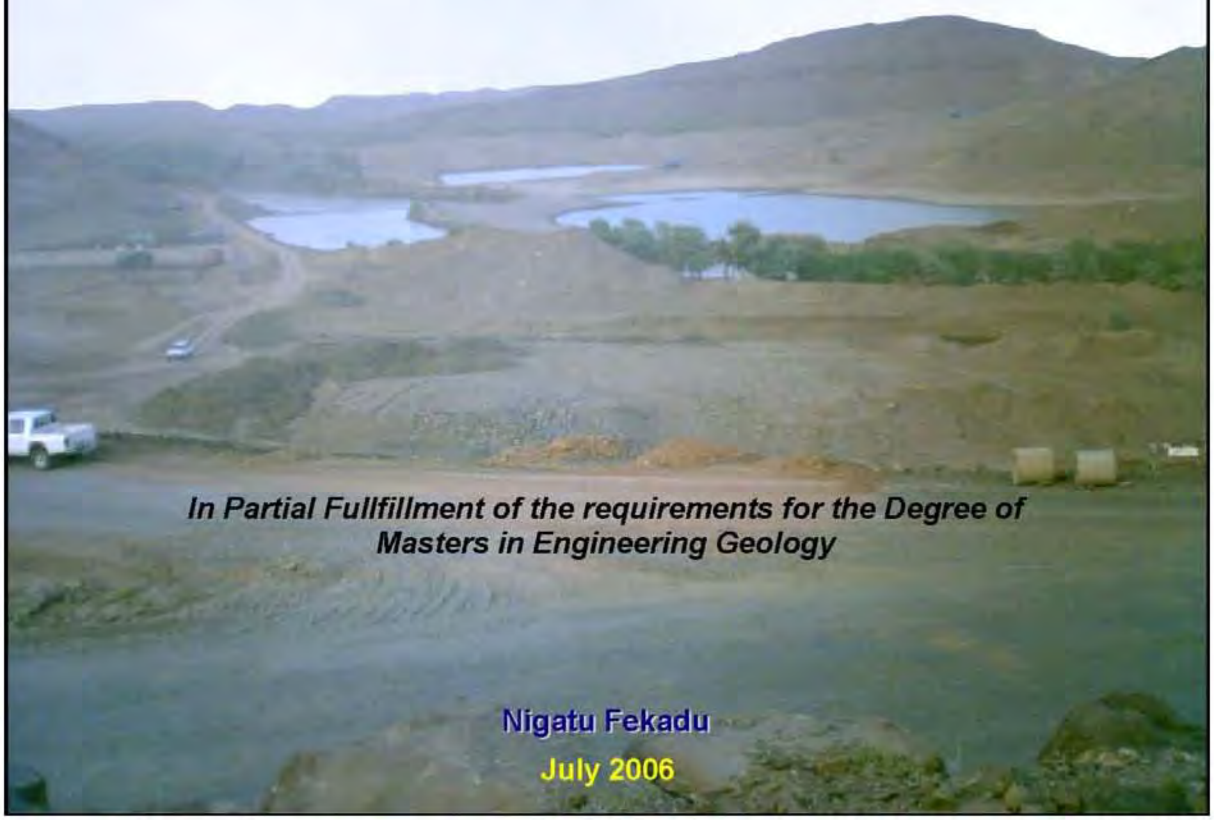

In Partial Fullfillment of the requirements for the Degree of Masters in Engineering Geology

Nigatu Fekadu

July 2006

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DECLARATION

I, the undersigned, declare that this thesis is my original work and has not been presented for a degree on any other university.

All sources of materials used for the thesis have duly acknowledged.

NIGATU FEKADU

signature__________________________

Place and data of submission: school of graduate studies, Addis Ababa University, July, 2006

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ABSTRACT

Tendaho Dam Project envisages a construction of a 42 m high embankment dam across Awash River. The site is located at coordinates 41^o 00’ East and 11^o 07’ North about 570 km from Addis Ababa in Afar Regional state. The main purpose of this dam project is to provide irrigation to sugar cane plantation covering a total area of 60,000 hectares. The success of this project will entirely depend on its safe functioning and the irrigation needs of the agricultural lands. Therefore, it becomes essential that the dam foundation must be stable against the loads imposed by the dam and the impounded water. Besides, to meet out the primary purposes of providing irrigation to the project site, it is necessary that the dam foundation is free from any excessive seepage. Further, the construction material used to form the various zones of the embankment must serve as a water barrier and should be resistant to the piping failure. Therefore, in the light of these important points the main objective of the present study were framed as; (i) to determine the suitability of the dam foundation and the embankment against seepage and (ii) to study the suitability of the construction material, to be utilized in the embankment. Thus, in order to achieve these objectives systematic surface and subsurface explorations were made. Representative samples were collected from the foundation and borrow areas, and laboratory tests were conducted to know index and engineering properties of the soils. Besides, secondary data has also been procured from the project Authorities. Later, an integrated analysis was made to know the foundation seepage conditions and to work out the suitability of the construction material for the embankment dam. Based on the findings a detailed evaluation of the seepage condition in the foundation area and through the embankment was carried out.

Chapter I INTRODUCTION

1.0 General

Stability of the foundation and suitability of construction material have an important consideration in the safe functioning of dam. Nearly up to the first half of the 20th century, dams were designed without proper investigation and very less emphasis was given to the geological environment on which the dam and its appurtenant structures is planned to be constructed. This deficiency in investigation has resulted into the failure of many dams. The main cause of failure of dams has been foundation problems and improper construction materials. In the last few years such failures serve as a pedagogic goal to carry out proper engineering geological exploration.

The study on dams requires a thorough engineering geological investigation to ensure that the structures meet the primary purpose of storage and its stability against the horizontal forces exerted by the impoundment. The engineering geological investigations at dam site are aimed at finding the suitability of the foundation against seepage, sliding and deformation in response to dynamic and static loads. Besides, as a part of engineering geological investigations the suitability of the construction material is also studied for the quality and the quantity required for the dam construction. A thorough investigation in the initial stages of planning of dam project not only ensures the safe functioning of any dam project but it also helps in the identification of adverse foundation problems. Thus, the identification of such problems may help to provide suitable remedial measures. The field investigations must be continued even during the construction stage. It is essential that the prediction of the ground conditions which constitutes the basic design assumption, are checked as construction proceeds and designs should be modified accordingly if conditions are revealed which differ from those predicted. (Bell, 1980).

In light of the above discussion the proposed research study is planned to investigate the engineering geological appraisal of Tendaho dam foundation and to assess the suitability of the proposed construction material.

1.1 The Study Area

The Tendaho Dam site is located in Afar Regional state and the location coordinates of the dam site are 41^{o} 00’ East and 11^{o} 07$^{'}$ North. The Tandaho Dam site falls within the Ethiopian

Mapping Agency topo sheets No 1140 B4. The Tendaho Dam site is located at about 570 km from Addis Ababa and is proposed at lower Awash basin. The project site is accessible through Addis Ababa – Samara asphalt road, which crosses the left abutment of the proposed dam (Fig.1.1). Topographically the study area is mainly characterized by elevated ridges and low-lying flood plains. The elevated areas consist of volcanic (mainly basaltic) rocks and the low-lying plains are characterized by alluvium and fan deposit. The elevation ranges from 360 to 412 m.a.s.l and the dam site is located on the Southern Western part of Afar Depression.

1.2 Previous Works

Five important studies have been carried out for assessing the water resources of the Awash basin. The first study was done by Sogreah –FAO (1965) the report entitled as “survey of the awash basin” in which key hydrometerological stations were installed and measurements of the elements were taken during the period running from 1962 to 1965. The report also indicated that some 200,000 ha of suitable land could be made available for Irrigation. Moreover, there was a conclusion that the maximum development potential of the basin would be about 163,000 ha, a figure less than current estimates. In the report, storage dam locations were considered to be Koka, Kesem and Tendaho. In addition the study identified various potential irrigation development areas as well as potential reservoir storage sites. The second study was made by Gibbs (1975) for the Feasibility Study of the lower Awash Valley, which basically concentrated on the Design of the Tendaho Dam and Irrigation project, in this study the location of drilling bore holes were located and drilled. Besides, detailed information on the foundation of the dam site has also been collected from the surface and subsurface geology. Moreover, location of the construction materials has also been proposed. The third study was done by UZBEK (1985), the study paid attention to the design of the Tendaho Dam and irrigation Project. Under this study emphasis was given to the geological structures. The study utilised the subsurface and surface geological investigation and listed the following components of the proposed development;

(i) UZBEK proposed a multipurpose dam near Tandaho on river Awash. The proposed height of the dam was considered as 38m and will be having a storage capacity of 720 Mm^3. The dam may be used for irrigation as well as for power generation of 6MW capacity.

(ii) Development of some 36,900 ha of new lands under gravity level basin irrigation.

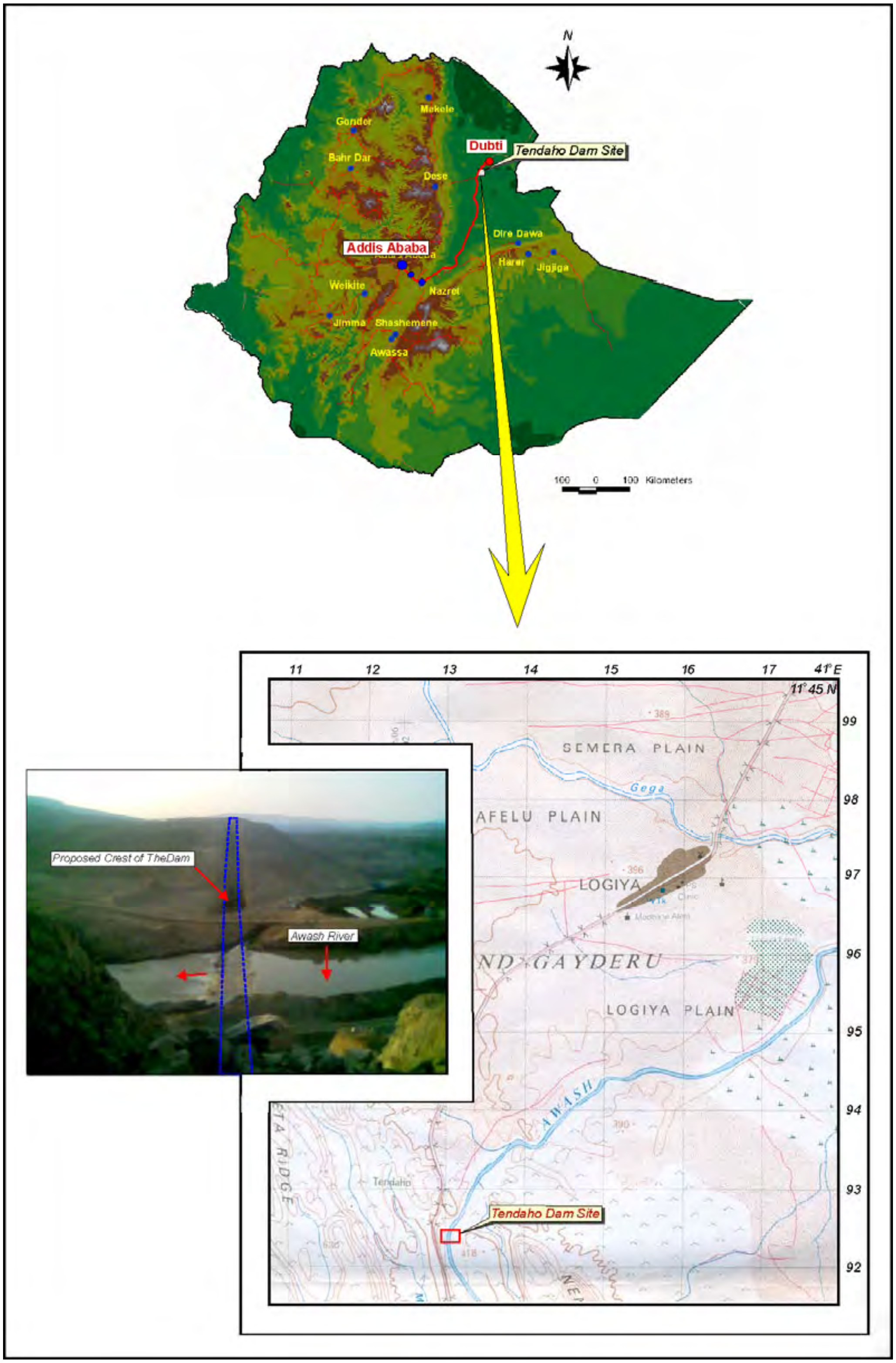


Fig. 1.1 Location of the study area

(iii) Rehabilitation of some 24,00 ha existing Irrigation land in Dubti, Asayita and Dit Bahri.

(iv) Construction of 6 intake structures on the Awash River and the provision of flood dyke protection for all development areas.

The fourth study was done by Halcrow (1989) which conducted an extensive study on the surface water resources in the Awash Basin while preparing the Master plane for Development of surface water Resources in the Awash Basin. The last and the recent study is by 'Water Works Design & Supervision Enterprise' in association with Water & Power Consultancy Service India Ltd, (2005). Under this study a detailed extensive study on the dam foundation was carried out. The study suggested an excavation of the top alluvial deposit in the core section while for the shell foundation excavation up to the depth where insitu relative density of greater than 55% is available. This is to guard against liquefaction of this soil in the event of major earthquake. For the Abutments the permeability result on the boreholes indicate the higher permeability of the rock mass, therefore to improve the permeability condition of the abutments curtain grouting is suggested.

1.3 Objective of The Present Research Study

The "Tendaho Dam" is a very important dam, as it forms a part of "Tendaho Dam and Irrigation Project", which aims to harness the inflow of river Awash and provide irrigation to sugar cane plantation covering a total area of 60,000 hectares. The irrigation through Tendaho Dam project will facilitate a high yield of sugar cane in the area and a total production of about 500,000 tone of sugar per annum (Main Report, TDIP, 2005). The success of this project will entirely depend on its safe functioning and to meet out the irrigation needs of the command area. Therefore, it becomes essential that the dam foundation must be watertight and it must provide a stable foundation to withstand the loads imposed by the dam. Besides, the construction material used to form the various zones of the embankment must serve as a water barrier and should be resistant to the piping failure. Keeping all these facts in mind following objects for the present study were framed;

General Objectives

(i) Engineering Geological appraisal of the dam foundation

(ii) Investigation of the suitability of the construction materials for the embankment dam.

Specific Objectives

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

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

(iii) Engineering Geological characterization of the construction material and its suitability for various zones in the embankment.

1.4 Importance of The Study

Engineering geological investigations for a dam site involves exploring the ground conditions at and below the surface in the foundation area. It is the prerequisite for the successful and economic design of a dam project. Engineering geological investigation plays a very important role in proper site selection, designing and construction planning of a dam project. A thorough investigation ensures the long life of the project. Identification of possible adverse and unfavorable conditions in the dam site area in the initial stages of investigation helps in adopting proper remedial measures. Insufficient or inadequate information with respect to the character of the ground can lead to the production of an unsatisfactory design, which may subsequently result in serious damage, or even failure of the structure. The investigations become more important for an embankment dam. Embankment dam are the earth or rock fill dams, which comprises various zones with different characteristics of the materials. An embankment dam must be sufficiently impermeable to retain the reservoir. It must be stable enough to withstand the forces to which it will be subjected and it should be capable of resisting internal erosion or piping. Hence, it becomes important to select the appropriate construction material for the requirements most efficiently and economically. The most likely engineering geological problems associated with the embankment dams are the seepage problems from the dam foundations and through the main dam body. The seepage problems are resulted mainly because of the inadequate investigations of the foundation area and poor selection of the construction material.

The proposed “Tendaho Dam” is an embankment dam, which will be constructed across the Awash river at Tendaho. The main purpose of this dam project is to provide irrigation to sugar cane plantation covering a total area of 60,000 hectares. The success of this project will entirely depends on its safe functioning and to meet out the irrigation needs of the command area. Therefore, it becomes essential that the dam foundation must be stable against the loads imposed by the dam and the impounded water. Besides, to meet out the primary purposes of providing irrigation to the command area, it is necessary that the dam foundation is free from any excessive seepage. Further, the construction material used to form the various zones of the embankment must serve as a water barrier and should be resistant to the piping failure.

The present research study was aimed to determine the suitability of the dam foundation conditions against seepage, sliding and deformation of the foundation material in response to the loads imposed by the dam. Besides, it was also intended to study the suitability of the construction material, to be utilised as the barrier against seepage in the embankment. Thus, through the present study an attempt has been made to identify the adverse foundation conditions, which may pose problems during or after construction stage. Thus, based on the foundation conditions certain remedial measures have been worked out.

1.5 Methodology

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

i) Preparation of the base map of the study area from existing topographical maps.

ii) Literature review to have an overview of geological, geomorphologic, hydro-geological and engineering geological condition of the dam site and the surrounding areas.

iii) Geological Mapping of the dam site and the borrow areas.

iv) Sub-surface exploration through Borehole logs.

v) Collection of engineering rock mass classification data to workout the strength and deformability characteristic of the rock mass present at the dam foundation area.

vi) Collection of soil and rock samples from dam foundation area, borrow areas and query sites for laboratory testing and analysis to determine various index properties.

vii) Assessment of permeability of the dam foundation material through existing water pressure test data and qualitative assessment through surface and sub-surface geological conditions.

viii) Assessment of stability conditions of the abutments. Collection of necessary data and samples for laboratory testing.

ix) Determination of the suitability of the construction material for shell, core, filters and transition zones. For this, the data generated from the laboratory testing and the existing data on gradation, permeability, compressibility, flexibility and erosion resistance has been utilized. Various criteria for the selection of appropriate construction material have been applied.

x) Based on the above methodology suitable remedial measures to improve the foundation condition has been suggested. Besides, recommendations on the quality of the construction material have also been made.

1.6 Application of the result

The results and the findings of the present study may be utilized by the Project Authorities or by any other individual or organization. The data / information generated through this study may highlight certain aspects of the dam foundation and the suitability of the construction materials. The data generated through this study may also be utilized by the later researchers intending to work on the same subject or in the same study area.

All efforts are being made to carry out the present study in a very systematic and organized way, well supported by the actual field data, laboratory tests and the secondary data procured from various sources. However, these efforts were made under time, resources and the financial constraints and further studies might be necessary on some aspects of the research findings.

Chapter II GENERAL OVER VIEW OF THE STUDY AREA

2.0 Introduction

The Awash River is one of the potential rivers for irrigation and hydropower as far as suitability and accessibility of surface water potential is considered. Awash River originates from the highlands around Addis Ababa and drains the rift valley and terminates into the low land areas of Afar Depression. The catchment area of the Awash river basin is very large and Gibb & Hunting (1975) classify the basin into five-sub basins. The first sub basin is upto Koka Dam, this is characterized by high rainfall, the second sub basin is from Koka Dam to Awash station, which is characterized by two major tributaries (Keleta and Arba). Third sub basin is from Awash station to Hertale station. The major tributaries for third sub basin are Kesem and Kebena. The fourth sub basin is from Hertale to Tendaho station, the major tributaries of this sub basin are rivers coming from Wollo mountains such as; Mille, Borkena, Ataye and Cheleta. Finally, the fifth sub basin is from Tendaho to the Alluvial fans in the Afar area where it finally terminates into Gamari lake.

The proposed Tendaho Dam falls in the fourth sub basin of the Awash catchment. This sub basin is mainly recharged from streams flowing from Wollo mountains such as; Mille, Borkena, Ataye and Cheleta.

2.1 Topography and Drainage of the Basin

The Awash drains the northerly part of the rift valley from approximately 8.5^0 N to 12^0 N with total drainage area of 1,12,211 Km2 (Hal Crow ,1989). The source of the Awash lies at an altitude of around 2500 m in plateau to the west of Addis Ababa. It first flows east, draining the Becho plains and is joined by several small tributaries before entering Koka Reservoir. After being released through Koka Dam, which came into operation in 1960, it descends into the Rift valley. The fall of the river in this reach is used for hydropower generation at Koka and at series of run-of river schemes designated as Awash II and Awash III. The river then turns gradually northwards, flowing at a reduced gradient along the base of the western highlands. Fig.2.1 shows the Awash basin catchment and its major sub basins.

In the reach between Koka and Awash Station, small tributaries join the Awash River like

Keleta, Wererso and Arba draining the highland, which define the catchments boundaries to the east, Beyond Awash Station, the Awash Basin expends in to the eastern plains as the Rift valley widen. Although the eastern plains account for some 40% of the area of the basin, its

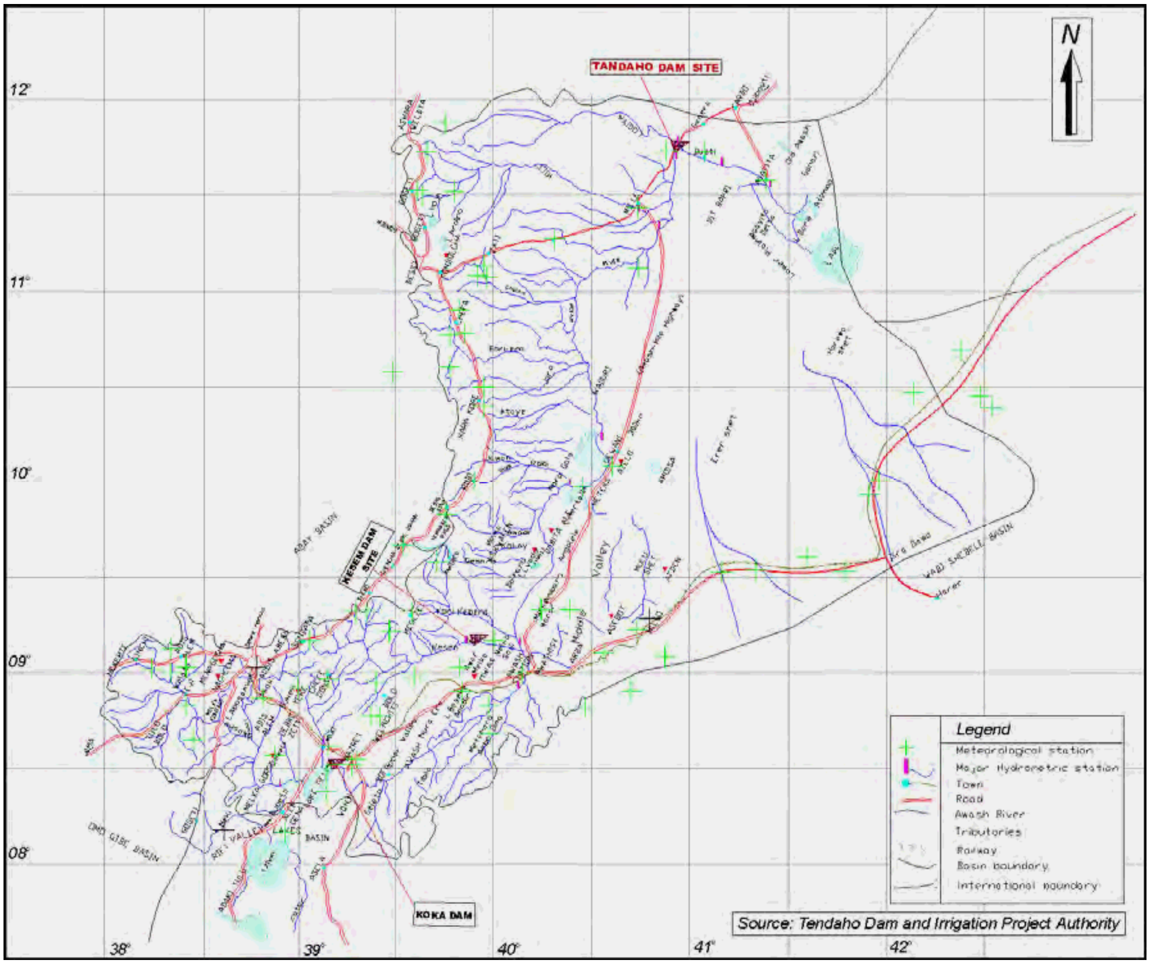


Fig. 2.1 Awash basin catchment and its major sub basins

drainage channel terminates before reaching the Awash and also receives low rainfall (less than 600 mm per year). Between the Awash Station and Gedebessa swamp (Harteal Station) major tributaries are Kesem and the Kebena enter the Awash originated from western highlands after passing through deep gorges. From the eastern sides the Herdini river joins the Awash. From the Hertale Station to the Tendaho Station the Ataye, the Borkena, the Chelela and Mille river originated from Wollo highlands joins the Awash river and all these streams contributes about 900 Mm^3 of water annually (Final Report of Hydrology, TDIP, 2005).

2.2 Land Suitability Evaluation of the Command Area

Land Evaluation is the process by which the suitability of land is checked for the required purpose, such as, irrigation, agriculture, waste disposal and construction activities. Proper utilization of land and water resources in the development of irrigation could lead to substantial increase in food production. As a result the objective of land evaluation for Tandaho project is to select suitable lands that are phyically and economically viable for the use of irrigation agriculture. The major soil characteristics which affect the sugarcane cultivation may inclued soil texture, particularly pore-size, geometery, soil depth, slope, water retention (available water capacity), oxygen requirement, infiltration rate, permeability, salinity / sodility, flooding, soico-economic and other relevant aspect (Tendaho Dam and Irrigation Report, 2005).

As per the Project Authorities land suitability classification for sugarcan production of the commaned area is as follows;

i) The total irrigation area suitable for irrigation of sugarcane through construction of Tendaho Dam is 85,617 ha.
ii) Potential sites for irrigation are 64,738 ha.
iii) Additional area for available expansion of irrigation is 20,879ha.
iv) Area that is marginally not suitable due to sodicity, wetness and other characteristics is 49,943 ha.
v) Area reserved for rangeland is 23,545 ha, out of which 11,041 ha land are suitable for future expansion for cultivation of sugar cane.

Table 2.1 presents the command area land use.

2.3 Agricultural Soil Description in the Project

The soil classification based on certain properties is essential in preparing a proper irrigation planning and irrigation schedule. This is important because irrigation is not always useful in increasing the fertility of the soil. Some soils after they are subjected to applied irrigation for longer duration may decrease their fertility as most of the essential ingredient of soil like potassium, magnesium and nitrogen is filtered down with the water. Soil classification of the command area, based on geomorphology and parent material, as per the Project Authorities is classified as under;

i) Lacustrine and old Alluvia
ii) Recent alluvial –Assayita Delta
iii) Young Riverine Alluvial –Hydromorphic flood plains.

Table 2.1 Command Area land use (sources tendaho dam and irrigation Report, 2005)

No.	Location	Command Area (ha)		Already developed under cultivation and /or fallow (ha)	To be developed (ha)			
					Under bushes		Under forest	
		Gross	Net		Gross	Net	Gross	Net
1	Dubti	12126	9367	6000	6126	3367	-	-
2	Galifage	3119	2409	-	3119	2409		
3	Boyale	6092	4706	-			6092	4706
4	Dit Bahari	12205	9428	5000	7205	4428		
5	Interable – Bokaitti	4303	3324	-	-	-	4303	3324
6	Assayita	26893	20773	15000	2550	2057	43605	3716
Total		64738	50007	-	19000	12261	15000	11746

Lacustrine and old Alluvial;

This soil type is found covering most of the command area, the north part of Dubti plain, northeastern part of bare land along with Semara –Assayita link road extending up to Assayita town and western part of the command area including Dit Baheri and adjoining rangeland area. The soils in Dit Baheri plain consist of saline and calcareous alluvium and colluvium material. The clay particles surface were predominately occupied by exchangeable sodium which result in dispersed clay sealing.

Recent alluvial - Assayita Delta;

This soil unit covers the Assiyita and extends to lakes Gamari, Afembo and Barrio. Due to frequent flooding by Awash River of this area this is not suitable area for irrigated agriculture. The alluvial consists of sand, silt and clay soils.

Young Riverine Alluvia-Hydromorphic flood Plaines;

These soils are present along the river course and in the low-lying areas, where water stays for long time. These soils are leached and characteristically have low electronic conductivity (EC) and exchangeable sodium absorption (ESP), which is attributed to leaching of salt to deeper horizons. The soils are black with heavy texture ranging from heavy clay to silty clay loam. These soils are relatively rich in organic matter.

Table 2.2 presents the general FAO Classification of the soil. The soils of the project area based on FAO classification are summarized in Table 2.3 and Fig. 2.2.

2.4 Reservoir Siltation

Siltation is the process of accumulation of the fluvial materials into the reservoir. These fluvial materials may be soil or rocks eroded from the upstream of the dam and transported by inflow water to the reservoir. Therefore, the data on ‘suspended sediment transport rate’ are useful in determining the effective and productive life of any Dam project.

Table 2.2 FAO Soil Classification

Soil Mapping Unit	Land form/Major soil group	FAO soil unit	Area (ha)
RA-1	Recent alluvium	Calcaric fluvisols	11132
RA-2	Recent alluvium	Eutric fluvisols	10099
RA-3	Recent alluvium	Glegic fluvisols	233
RA-4	Recent alluvium	Eutric fluvisols	5758
RA-5	Recent alluvium	Pellic fluvisols	8274
RA-6	Recent alluvium	Salic vertisols	942
LS-1	Lacustrine sediments	Sodic Solonchaks Solonetz	10457
LS-2	Lacustrine sediments	Sodic solonchaks-Solonetz	2343
LS-3	Lacustrine sediments	Pellic vertisols	1234
LS-4	Lacustrine sediments	Calcaric vertisols	10484
LS-5	Lacustrine sediments	Natric vertisols	2865
LS-6	Lacustrine sediments	Eutric fluvisols	6699
LS-7	Lacustrine sediments	Eutric fluvisols	12090
LS-8	Lacustrine sediments	Pellic vertisols, Indandic phase	7983
LS-9	Lacustrine sediments	Eutric vertisols,gilgai phase	2266
LS-10	Lacustrine sediments	Pellic vertisols	869
LS-11	Lacustrine sediments	Eutric vertisols	4699
LS-12	Lacustrine sediments	Arenic Regosols	3410
LS-13	Lacustrine sediments	Salic Solonetz	11996
YA-1	Young riverine alluvium	Calcaric fluvisols,Inandic phase	14182

(Source: Tendaho Dam &Irrigation project Report, 2005)

Table 2.3 Soil units of the Command Area –based on FAO Classification (in ha.)

Potential sites	RA-1	RA-2	RA-5	RA-6	LS-2	LS-3	LS-4	LS-5	LS-6	LS-7	LS-8	LS-9	YA-1	TOTAL (ha)
Dubti	-	-	-	-	-	1086	3644	1708	5888	-	-	-	-	12126
Gallifage	-	-	-	-	474	-	483	-	-	-	2386	250	-	3593
Boyale	-	-	-	-	-	-	3015	-	-	-	3077	-	-	6092
Ditbahari	-	-	-	-	-	-		562	845	9678	1120	-	-	12205
Lower Ditbaheri	2175.12	-	529	-	-	-	1598	-	-	-	-	-	217	4519
Assayita	8956.42	10099	7745	93	-	-	-	-	-	-	-	-	4190	31083
Total	11131.54	10099	8274	93	474	1086	8740	2270	6533	9678	6583	250	4407	69618

(Source: Tendaho Dam &Irrigation project Report, 2005)

Over one third of the storage volume of Koka reservoir has been lost by sedimentation (Halcrow, 1989). This indicates high silt concentration of the Awash river and its tributaries. The productive design period of Tehdaho Dam is estimated to be 50 years. The summery of different studies on sedimentation estimations of Tendaho Dam reservoirs are summarized in the following paragraphs;

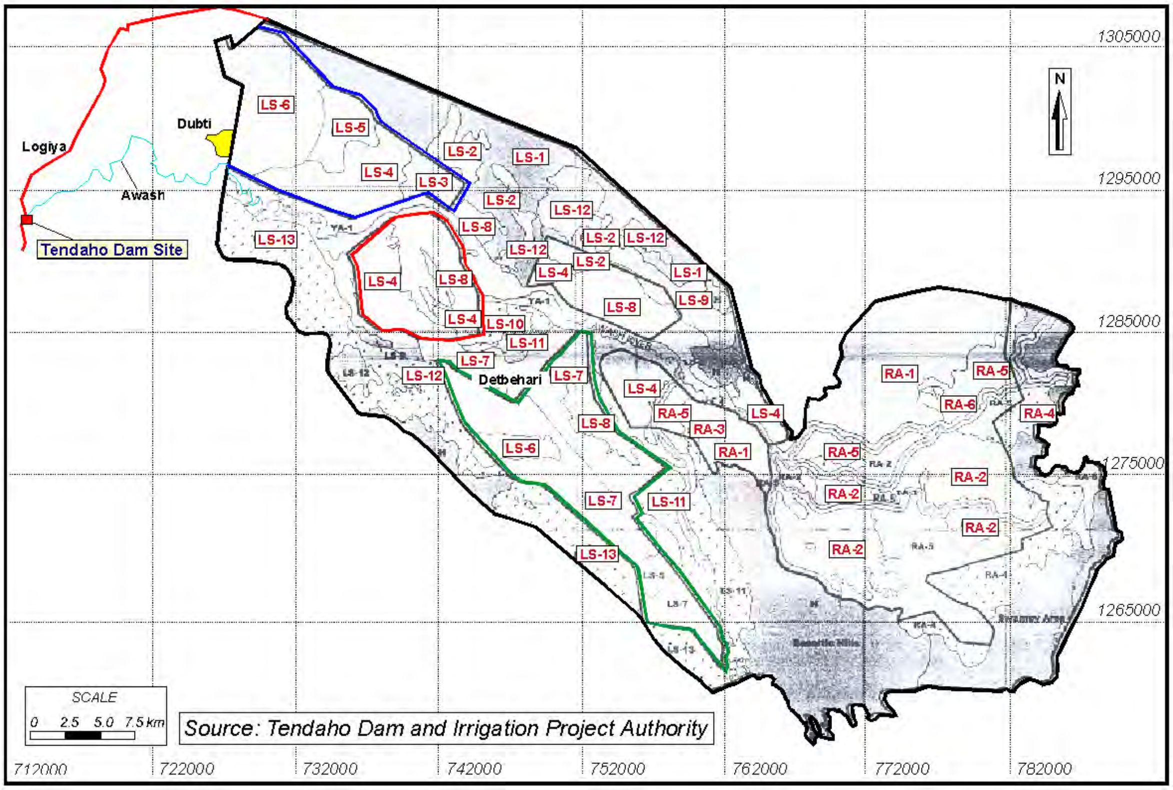


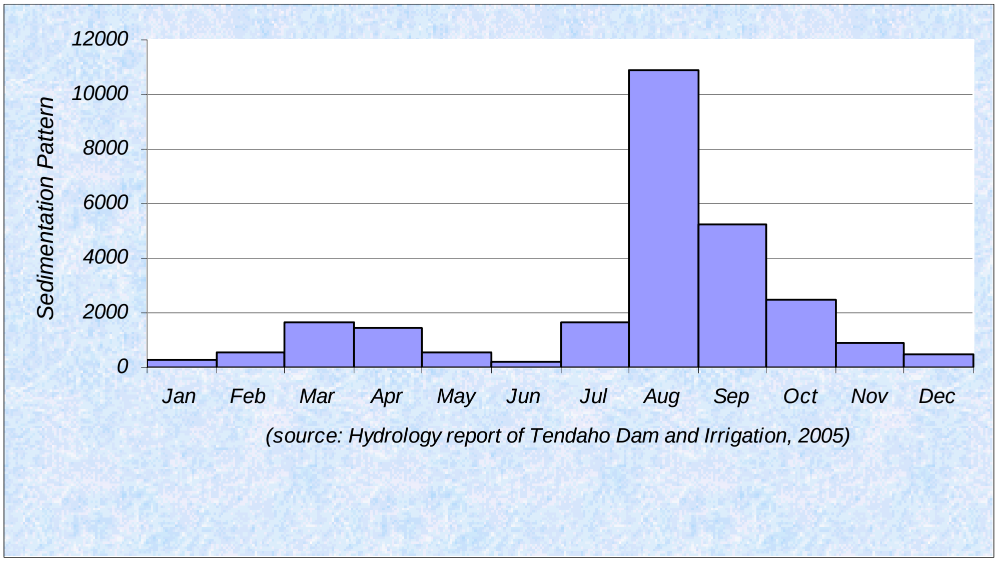
Fig. 2.2 Soils of the Command area based on FAO classification

The productive life of Tendaho Dam project as estimated by Gibb (1975) is 50 year, with a total storage of 970 Mm^3 and live storage of 400 Mm^3. Gibbs estimated the bed load (density of 1.4 gm/cc) materials to be 4 Mm^3/year, suspended material as 8 Mm^3/year and thus, the total sediment deposit as 12 Mm^3/year. In the case when maximum run-off or maximum annual sediment load from the reservoir is not draw down during the months of July and August, additional 11 Mm^3/year sediment deposits may accumulate. This may result into a total sediment deposition of 23 Mm^3/year. Thus, the reservoirs may be filled with the sediments before the expected productive life of the reservoir.

UZBEK (1985) estimated 25 Mm^3/year sedimentation of the Tendaho reservoir with no explanation on the derivations.

For reservoir operation Gibb (1975) and UZBEK (1985) indicated that density current removes the sediment fraction less than 0.015 mm, which contributes about 60% of sediment deposition. By this about 21% of the suspended sediment could be removed by releasing sediment-laden water during heavy rainfall through bottom outlet. As a result sediment deposition in the reservoirs will be reduced and the life of the Dam may be 50 year.

Fig 2.3 shows the monthly sediment load in the Tedaho Dam based on generated monthly sediment load of the Awash River at the Tendaho Dam site (1962-2002). The dead storage level is present in Table 2.4.



(source: Hydrology report of Tendaho Dam and Irrigation, 2005)

Fig. 2.3 Monthly mean sediment pattern of Awash River at Tendaho Dam Site.
(Based on the 1962-2002 data)

Table 2.4 Estimated dead Storage level of Tendaho Reservoir after 25 and 50 years of reservoir service for different full reservoir levels

Year	Full reservoir Level	408m(Elevation)	409m	410m
Sediment Volume (Mm^3) after 25 years of service	Storage capacity	1859 MMC	2017 MMC	2180 MMC
	Sediment Volume	517.9	517.9	517.9
	Dead storage level (m)	384.0	384.0	383.0
	Live Storage (Mm3)	1329.6	1448.2	1643.7
Sediment Volume (Mm^3) after 50 years of service	Storage Capacity	1859 MMC	2017 MMC	2180 MMC
	Sediment Volume	1035.8	1035.8	1035.8
	Dead storage level (m)	392.0	392.0	391.5
	Live Storage (Mm3)	647.9	710.4	1031.7

(Source: Hydrology report of Tendaho Dam and Irrigation Project, 2005).

As per the Hydrological Study (2005) of the Awash River at Tendaho dam site the estimation of the productive life of the reservoir is 50 years. This study suggests that the largest sediment load is received in August (41% of the annual sediment load), with the second maximum in September (20%). About 21% of the annual flow of $400Mm^3$ contributed by August. Therefore, they recommend to plan maximum sugar cane Irrigation during the months of August and September. With this, it may be possible to flush out the maximum sediments and in a process; the siltation of the reservoir will be reduced. For this purpose density current removal of incoming sediment during the month of August and September can be used.

Chapter III CLIMATE AND HYDROLOGY

3.0 Preamble

Hydrology deals with the waters of the earth, their occurrence, circulation, and distribution, their chemical and physical prosperities and their reaction with the environment including their relation to living things. Hydrological study includes the important parameters such as precipitation, evaporation and run-off data use for design of engineering project, site selection and the land use planning. Hydrology is used in engineering mainly in connection with the design and operation of hydraulic structures, what flood flows can be expected over a spillway, what reservoirs capacity is required to assure adequate water for irrigation or municipal water supply during droughts (Fetter.1987)

The geology, topography, climate and land cover determines hydrology of a given basin. The Climatic data of the basin such as precipitation, temperature, and wind speed, evaporation, and relative humidity and sunshine duration are important parameters in the analysis of hydrology of the basin. Occurrence of ground water may affect engineering works therefore during site selection emphasis must be given to evaluate the presence and the effect of the ground water on the proposed structure.

3.1 Climate

The inter tropical convergence zone (ITCZ) influence the rainfall formation over the Awash basin. Two rainy seasons has been experienced on the most part of the Awash watershed. The climate of the Tendaho dam site is hot and arid with very low rainfall. The climatic elements such as temperature, relative humidity, wind speed and sunshine hours are primarily required in estimating the potential evaporation (ETo) and reservoir evaporation. In the Tendaho Reservoir and Irrigation, commend area there are three meteorological stations, these are; Dubti, Dit bahari, and Assayita. However, the long period climatic records are only available with Dubti station. The mean maximum and mean minimum monthly rainfall, as determined from the data obtained from Dubti station, is 43.9 mm and 2 mm in the months of August and November (1986-2003), respectively. In the study area there are two rainy seasons one from March to April and other from July to August, with mean annual rainfall of 221mm.

The mean monthly temperature ranges from 17^{o}C to $40\,^{o}$C. The monthly mean maximum temperature varies from 33.6 ^{o}C in February to 43.2 ^{o}C in June. Whereas, monthly mean minimum temperature variees from 17.8 ^{o}C in January to 25.1 ^{o}C in August. The relative humidity and sunshine hours varies from 50.5 % to 65.7% and 8.3 to 9.8 hour during June and April, respectively. Movement of air and moisture transfer depends on the wind speed (Tenalm & Tamiru, 2001), the maximum wind speed in the project area is 186.9 Km/day in March and Minimum 95.4 Km/day in September (1979-2002).

The mean monthly temperature, wind speed, relative humidity, sun shine duration and rainfall are presented in Table 3.1 and Fig 3.1 & 3.2.

Table 3.1 Summery of the climatic data for the project area as recorded at Dubti meteorological station

Year	Parameter	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1986-2003	Rainfall	3.8	13.3	24.7	35.7	9.4	2.7	43.1	55.6	15.2	8.9	2	6.6	221.8
1979-2002	Max.T^{o}C	32.3	33.6	35.6	38.0	40.1	43.2	40.9	39.2	38.9	36.9	34.4	32.8	
	Min.T^{o}C	17.8	19.3	21.4	22.6	24.1	26.1	25.7	25.1	24.4	20.8	17.9	16.9	
1979-2002	Relative Humidity	67.4	66.4	63.4	61.6	55.5	50.5	55.0	58.7	59.6	63.3	62.5	65.7	
1979-2002	Shin shine hours	8.9	8.3	8.1	9.2	9.9	8.3	7.4	7.8	8	9.5	10	9.8	
1979-2002	Wind speed	167	176.3	186.9	169.9	126.3	126.3	137.2	134.9	95.4	102.3	134.1	149.1	

Evaporation of the study area was estimated using open water evaporation by different groups. Sogreah (1985) estimated an evaporation of 31335 mm/year; UZBEK (1985) estimate evaporation as 2999mm/year and LUN (1987) estimated evaporation as 2700 mm/year (Hydrology report, TDIP, 2005). Table 3.2 presents the estimated evaporation of the study area.

3.1.1 Surface Water Hydrology

Surface run-off will be defined as the portion of precipitation that flow over land surface towards the surface water system such as river, lakes and streams. Part of precipitation that falls on land surface is retained in the vegetation cover and evaporates before reaching the land surface, another part infiltrate in to the soil. If the portion of precipitation reaching land surface is greater than the infiltration capacity of the soil then only the excess water will start forming ponds and eventually it may flow towards the surface water system lakes, streams and rivers. The duration and intensity of the precipitation in general influence the run-off characteristics of a given basin. The various factors which influence the precipitation are;

i) type of the vegetation cover,
ii) infiltration capacity of the soil,

iii) slope and roughness of the drainage basin,
iv) geological formation
v) permeability of rocks and the soil.

The Awash River drains the Northern part of the rift valley from approximately 8.5 ^{o} to 12 ^{o} N with a total drainage area of 112211 km^2.

Table.3.2 Estimated evaporation (mm) by different studies at Tendaho

Month	PAN NMSA	PET est. By USBEK 1985	PET est. By Gibb 1975	PET est. By LUP 1987 Dubti	PET est. By LUP, 1987 Tendaho	Sogreah 1965 Est.at Gewanita (625m)	Reservoir Evaporation Tendaho LUP-1987	Reservoir Evaporation Tendaho UZBEK-1985	Evapotranspiration From Irrigation area In the lower Vally UZBEK_1985
Jan	187	158	155	179	185	196	213	180	158
Feb	194	165	167	181	188	217	216	173	165
Mar	249	182	212	213	222	276	255	252	182
Apr	246	181	223	224	273	306	314	235	181
May	281	200	237	233	256	302	294	255	200
Jun	287	226	241	238	218	314	251	264	226
Jul	312	183	235	219	213	262	245	276	183
Aug	248	180	238	206	209	308	240	242	180
Sep	242	186	208	206	201	316	231	240	186
Oct	227	177	200	203	229	242	263	220	177
Nov	201	164	170	179	210	198	242	280	164
Dec	182	154	150	172	204	198	235	283	154
Annual	**2856**	**2156**	**2436**	**2453**	**2608**	**3135**	**2999**	**2900**	**2156**

(Soure:Hydrology report of Tendaho Dam & Irrigation project,2005).

The estimated mean annual flow of the Awash River at Tendaho Dam site with drainage area of 63,485 Km^2 is 2,334 Mm^3 over the reference period of 1962 to 2003 (Main report of Tendaho Dam & Irrigation project, 2005). During the wet year of 1964 the annual flow was 4003 Mm^3 and during drought year of 1984 the annual flow drops to 932 Mm3. According to Gibb (1975) based on their basin development models, the estimation average inflow at Tendaho reservoirs is 1690 Mm3/year. The model considers the upstream abstraction. In Awash, basin number of gauging station area available to measures the river discharge. The Awash River in lower basin fed major tributaries rivers emerging from Wollo Mountainous namely, Mille, Borkena, Ataye and Cheleta Rivers.The mean flow of Awash river, as observed at the gauging station 600 m downstream of the dam site, is 2506.43Mm^3. The mean monthly discharge of the Awash River at Tendaho is presented through Fig. 3.3.

3.1.2 Reservoir Storage Capacity

The storage capacity of the reservoir mainly depends upon the topography, height of the Dam, and the amount of water inflow. The inflow into the Tendaho reservoir is mainly determined by three factors. The first is runoff characteristics of the Awash watershed upstream of the Tendaho Dam, and the second is irrigation and water supply abstraction from the Awash river, and the third is the magnitude of evaporation and seepage losses in the Gedebessa swamp complex (Hydrology report, TDIP, 2005).

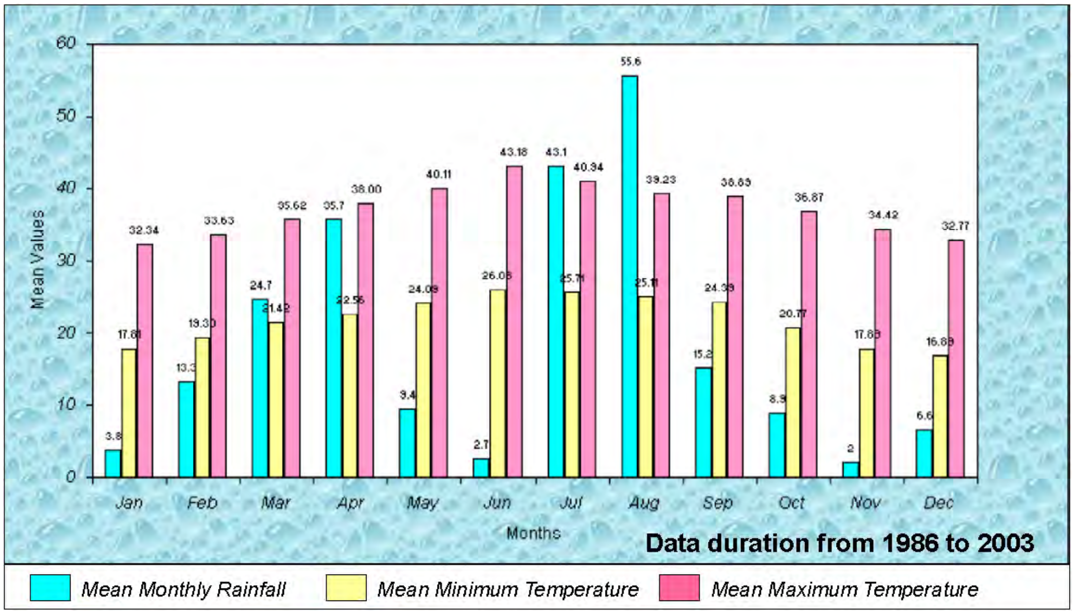


Fig 3.1 **Mean monthly rainfall (in mm), minimum and maximum temperature (^oC)**

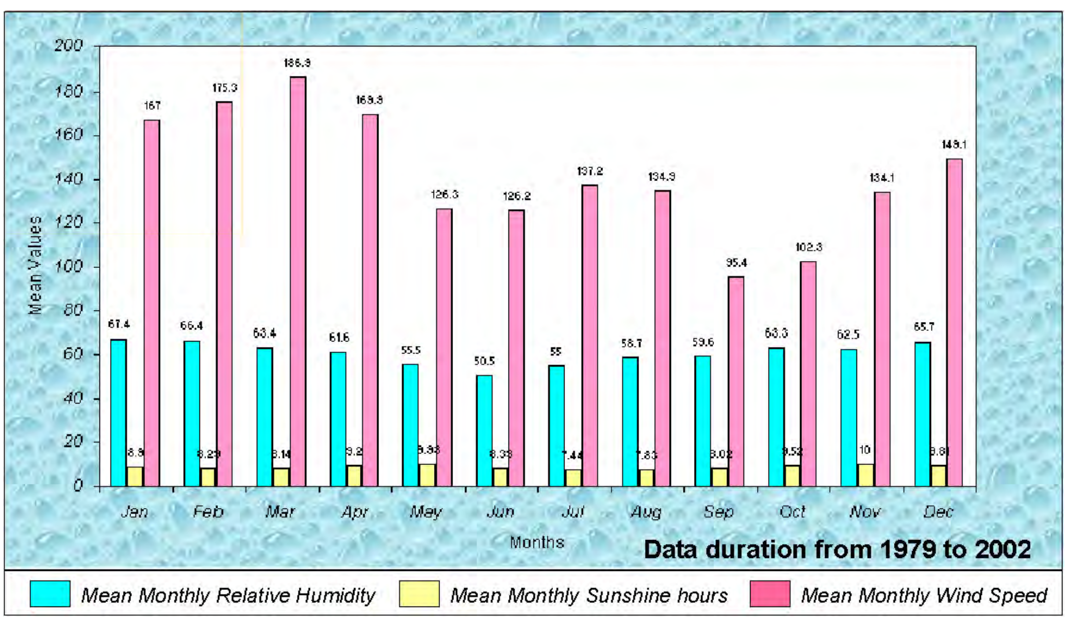


Fig 3.2 **Mean monthly relative humidity (in%), Sunshine hours and wind speed (km/h)**

The area coverage of Tendaho reservoir is 1700-hectare .The full supply level of the reservoir is at elevation of 408m having maximum fetch length of 38.7 kilometer. The gross storage capacity of the reservoir that was planned during feasibility studies in the past by Alexander Gibb, UZBEK and the current study are present in Table.3.3

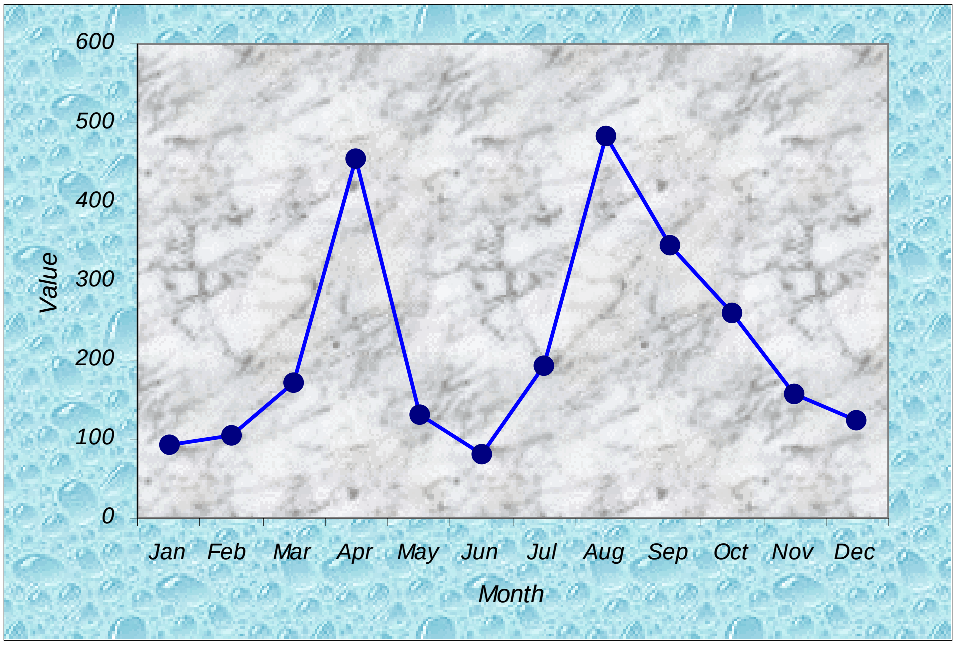


Fig 3.3 Mean monthly discharge of Awash river near Tendaho (1962-2002)

Table.3.3 Storage capacity of the reservoir as proposed by the Different studies

Proposed by	**Dam Height (m)**	**Gross Storage (Mm3)**	**Remarks**
Alexander Gibb Dam crest 407.2 FRL 401.2	36.0	720	As per old topographic Survey of reservoir
UZBEK Dam crest 407.2 FRL 401.2	36.0	720	
Dam crest 407.5 FRL 401.2	42.5	1225	As per current topographic Survey of reservoir
Dam crest 410.0 FRL 406.0	42.5	1561	
Dam crest 412.0 FRL 408.0	42.5	1860	

As per the recent Design of the Tendaho Dam and Irrigation Project maximum reservoir level is assumed to be 408m with maximum reservoir depth of 42.5m in the dam axis. The reservoir capacity of the Tendaho Dam is present in Fig 3.4.

3.1.3 Ground water hydrology

In many Dam relate project the effect of ground water consideration of the site and remedial measures get less emphasis. However if not properly treat the seasonal fluctuation of ground water table cause uplift pressures on structures constructed above it, also the presence of ground water in the discontinuities of rock mass affect shear strength of the rock mass resulting instability of the reservoir slopes and abutment slopes. As far as hydrogeological condition of the study area is considered it is classified into five hydrolithological units; Alluvial fan, Alluival and Lacustrine deposit, Recent basalt, Acidic rocks and older basalts (Hydrogelogical report of TDIP, 2005).

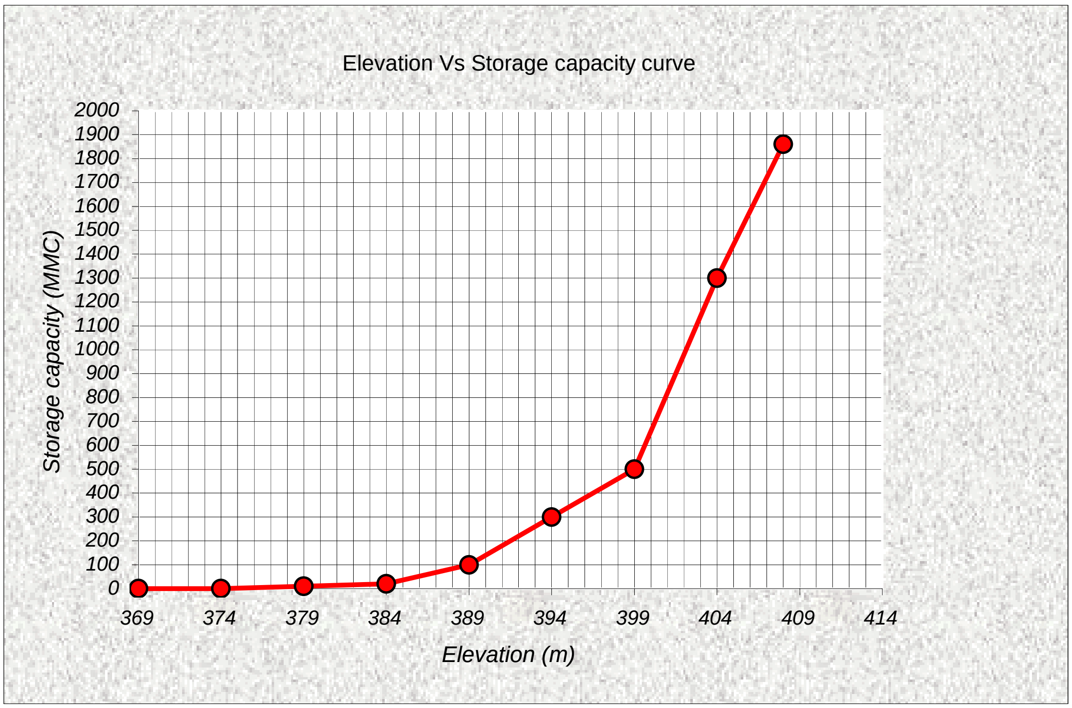


Fig3.4 Elevation vs Storage capacity curve

The major recharge sources of ground water for Tendaho graben are sub-surface seepage from the fractured opening of the volcanic rocks and unconsolidated sediments of the catchments and seepage form Awash River and irrigation (Hydrological report, 2005).

The ground water of the study area varies from place-to-place and classified as regional ground water, irrigation water and river water seepage. The regional ground water depths depends on its location and elevation. The ground water table of Logiya, Dubti and Assaytia, where there is no influence of irrigation and river water seepage, the ground water depth is 17m, 15m and 28 m, respectively. Wells drilled along the Awash River shows shallow water table due to the presence of seepage from the river, not more than 4m, whereas in areas of irrigation practices, like Dubti and Deitbahi, shallow ground water depth of 3m is observed. In the dam site three peizomteric wells were installed to investigate the ground water fluctuation. As inferred from the wells data there is no significant change of ground water during measurements at different times.

In the dam site the test pits excavated in the present study indicates that ground water is shallow and is in the range of 3-4 m below the ground. The lithology of the aquifer is alluvial deposits comprising of gravel, sand silt and clay material of high permeability.

3.1.4 Hydrochemistry of the reservoir water

The chemistry of reservoir water affects the embankment causing piping and the corrosion of the concrete structures such as, spillway and Tunnel. The chemistry of the reservoir water is also important to workout the suitability of the reservoir water for Irrigation. The most important composition of water which cause the above mentioned problem are Na^{+}, Ca^{++}, Mg^{++}, SO_4, Cl, HCO_3 and CO_3. During the present study, two water samples were collected form the Dam site for laboratory test. Based on the laboratory result of the current water sample the following Analysis was carried out;

Effect of the quality of reservoir water on concrete structures

The coexistence of sulphate and chloride ions in the reservoir water causes deterioration of the concrete structures such as spillway and tunnel. The corrossivity of the reservoir water can be determined from the corrosivity ratio coefficient, CR (Mahadevaswamy, 2002). If the CR value is greater than 1 the reservoir water is corrosive. In corrosive reservoir water conditions, while doing construction, a proper precaution has to be taken to reduce the effect of corrosion, According to Mahadevaswamy (2002), the value of the corrosivity coefficient can be determined from:

$$CR = \frac{0.028Cl + 0.021SO_4}{0.02(HCO_3 + CO_3)} \qquad \text{……..3.1}$$

Thus by applying this relation, it has been found that CR value for the water samples collected during the present study, is less than one. Therefore, the concrete structures are not affected by the water chemistry of the reservoir.

Irrigation water quality

The suitability of water for irrigation depends on the effect of the chemical constituent of the reservoir water on plant and the soil. The most important characteristics of irrigation water are total concentration of soluble salts, proportion of sodium to other cations, concentration of potentially toxic element to plants, and bicarbonate concentration as related to the concentration of calcium plus magnesium.

The total soluble salts are commonly indexed by the electrical conductivity and TDS of the water. The sodium hazard (when the concentration of sodium is far more grater than the concentration of the major cations it will affect the structure of the soil by reducing its permeability) is expressed by the sodium adsorption ratio (SAR). For the present study two water samples were collected from the Awash river and were tested for its chemical composition in the Ministry of Water Resources, Addis Ababa. The results thus obtained are presented in Table 3.4 and Table 3.5.

Table 3.4 Result of the chemical quality of the water from Tandaho Dam Site

Parameters	Result
Turbidity (NTU)	1850.0
Total solids105^{o}C (mg/l)	2800.0
Total Dissolved Solid 105^{o}C (mg/l)	359.0
Electrical Conductivity (μS/cm	548.0
P^H	7.95
Ammonia (mg/l NH_3)	0.38
Sodium (mg/l Na)	80.0
Potassium (mg/l K)	4.8
Total Hardness ((mg/l $CaCO_3$	105.6
Calcium (mg/l Ca)	29.4
Magnesium (mg/l Mg)	8.1
Total Iron (mg/l Fe)	0.02
Manganese (mg/l Mn)	0.02
Fluoride (mg/l F)	1.82
Chloride ((mg/l Cl)	40.3
Nitrite (mg/l NO_2)	-
Nitrite (mg/l NO_3)	0.65
Alkalinity (mg/l $CaCO_3$)	177.7
Carbonate (mg/l CO_3)	Trace
Bicarbonate (mg/l HCO_4)	216.7
Sulphate (mg/l SO_3)	45.65
Phosphate (mg/l PO_4)	0.123

$$SAR = \frac{Na^{+}}{\sqrt{(Ca^{++} + Mg^{++})/2}} \qquad \text{.........3.2}$$

Where the cations are in units of meq/l

% Na=100*(Na +K)/(Ca+Mg+Na+K)

Sodium consternation is important in the classification an irrigation water because sodium react with soil to reduce its permeability. Soil containing a large proportion of sodium with carbonate at the predominant anion are termed alkali soils; those with chloride or sulfate as the predominant anion are saline soils (Todd, 2001)

Table 3.5 Water quality Awash river at Tendaho Dam Site

Sample	SAR	EC(uS/cm)	HCO_3 (mgl)	%Na
Dam site	26.12	548	216,7	60

Wilcox (1955) referred by Todd (1980) has classified quality of water for irrigation as indicated in Table 3.5

Table 3.6 Wilcox water quality classification for irrigation

Water class	%Na	EC
Excellent	<20	<250
Good	20-40	250-750
Permissible	40-60	750-2000
Doubtful	60-80	2000-3000
Unsuitable	>80	>3000

The water quality of the Awash River at Tendaho Dam site is within the permissible limits as suggested by the Wilcox classification (Table 3.5).

Chapter IV GEOLOGICAL SETTINGS

4.1 Regional geological Setting

The Afar triangle is triple junction, where the three-rift system the red Sea, East African and the Gulf of Aden converges. According to Alebachew Beyene and Mohamed G. Abdelsalam (2005) the geological units of the Afar Depression and marginal areas classified into four broad groups (Fig. 4.1): (1) Neoproterozoic basement, Mesozoic sedimentary rocks, and Eocene–Miocene basalts; (2) Miocene igneous rocks; (3) Pliocene volcanic rocks; and (4) Quaternary volcanic and sedimentary rocks (Varet, 1978).

4.1.1 Neoproterozoic Basement, Mesozoic sedimentary rocks and Eocene–Miocene basalts

The Neoproterozoic basement, which represents part of the Arabian–Nubian Shield, is prevalent on the periphery of the Afar Depression. The Arabian–Nubian Shield covers vast terrain to the north and northwest of the Afar Depression in eastern Eritrea and northern Ethiopia, respectively (Vail, 1985; Berhe, 1990; Stern, 1994). The Neoproterozoic rocks of the Arabian–Nubian Shield also occupy parts of the Danakil and Ali-Sabieh Blocks.

Mesozoic sedimentary rocks that get progressively younger towards the south and southwest on the Ethiopian and Somalian Plateaux, respectively, overlie the Neoproterozoic basement rocks. On the Ethiopian Plateau, the Mesozoic sedimentary rocks comprise Early Jurassic Adigrat Sandstone, Middle Jurassic Abay Limestone, Late Jurassic Antalo Limestone and Cretaceous Debre Libanos Sandstone (Varet, 1978 and Tefera et al., 1996,).

The pre-rift group (Neoproterozoic basement/Mesozoic sedimentary rocks) either does not exist in the Afar Depression or are covered beneath the Pliocene and Quaternary volcanism and sediments. Within the Afar Depression, many structural features are found to suggest the influence of pre-existing fabric in its evolution. Black et al. (1974) identified the ~ 900 km long pre-Miocene Marda Fault Zone, which continues from the Somalian Escarpment towards the Indian Ocean, to be in perfect alignment with the Red Sea western margin and the Erta Ale axial ranges. Purcell (1976) further proposed from gravity and seismic studies that the Marda Fault Zone is a major volcano-tectonic lineament aligned along the lithospheric weakness zone, that could be projected along the Erta Ale axial range to the Red Sea or along the ErtaAle axial range into the highlands of Eritrea where it corresponds with Neoproterozoic faulting and a significant facies change in the Jurassic limestone. Tectonism on the Marda Fault Zone itself is extinct, but it is still influencing the Pleistocene volcano-tectonic activities in the Afar Depression (Purcell, 1976). In addition, some isolated pre-rifting structural trends are located around the Afar Depression and could have local influence. Ghebreab (1998) and Ghebreab et al. (2002) identified N-trending Neoproterozoic shear zones at the northern tip of the Danakil Depression and further north along the Red Sea coast that were reactivated by younger structures. Collet et al. (2000) indicated reactivation of Neoprotorozoic structures that transect the western Afar margin, the Danakil Block and the western margin of the Arabian Plate.

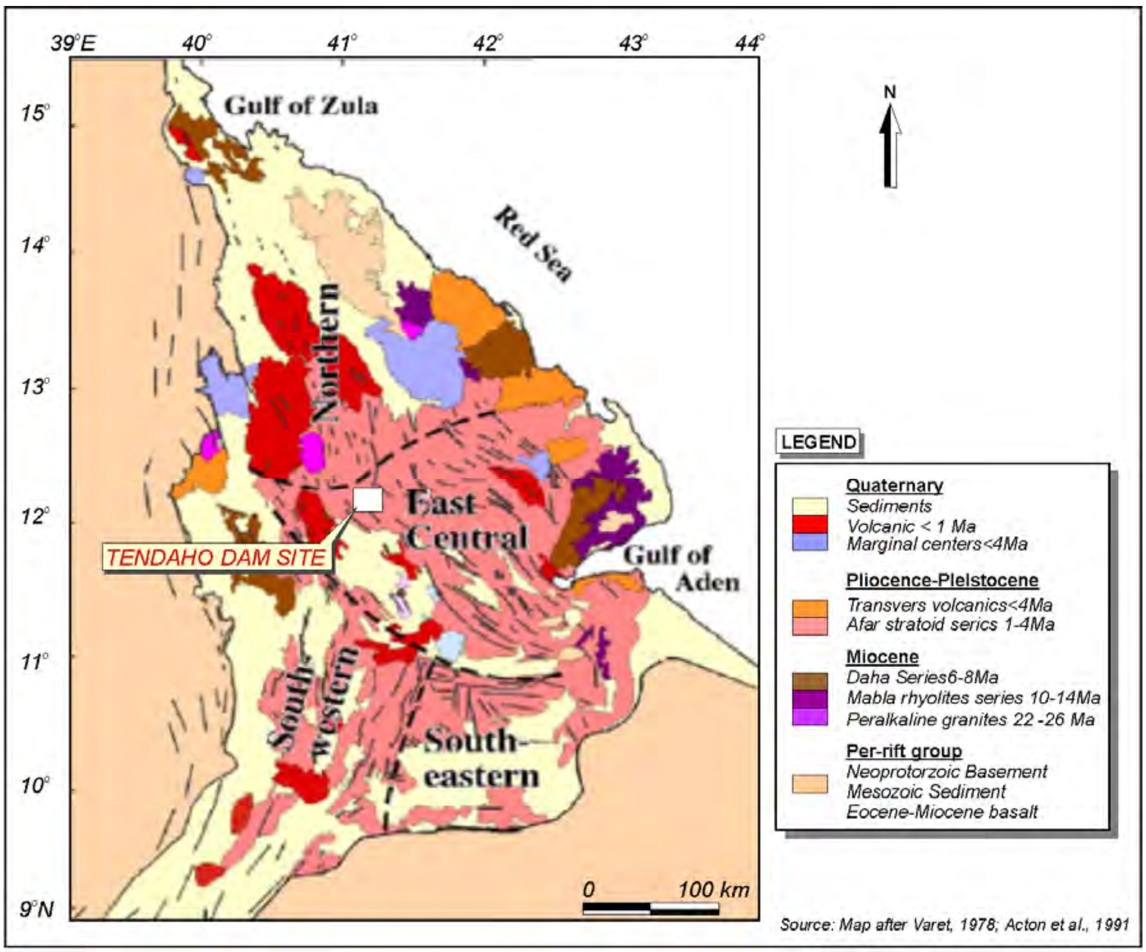


Fig. 4.1 Geological map of the Afar Depression

The first known volcanism since the Neoproterozoic time appeared in northern Somalia and the first important volcanism occurred at the southern and western margin of the proto-Afar during the late Mesozoic (Mohr, 1975). The flood basalts of the Trap Series on the African Plate covers ~500,000 km2 and these are ~2000 m thick (Hofmann et al., 1997; Kazmin and

Byakov,2000). The Trap Series was extruded onto an erosional surface and sometimes found inter-bedded with fluvial, lacustrine and sub-aerial sedimentary rocks near or above sea level (Civetta et al., 1975; Pallister, 1987; White and McKenzie, 1989).Berhe et al. (1987) have identified three stages of volcanism in the Trap Series at ~50–40, ~40–30, and ~30–21Ma whereas Ebinger et al. (1993) proposed that the main phase of volcanism occurred between 45 and 30 Ma. Hofmann et al. (1997) concluded from 40Ar/39Ar age dating in the Ethiopian Plateau that the Trap Series volcanic rocks were erupted over a short period of time at~30 Ma.

4.1.2 Miocene igneous rocks

Eocene–Miocene flood basalts cover the Mesozoic sedimentary rocks on both the Ethiopian and the Somalian Plateaux and some parts of the marginal areas (Fig.4.1). Flood basalts of ~25–15 Ma found within the Afar Depression are deeply weathered and intensely faulted. These occur in limited area around the Gulf of Tajura and on the Ali-Sabieh Block (Varet, 1978; Vellutini, 1990; Acton et al., 1991). Alkaline to per-alkaline intrusive rocks are found along the western and eastern Afar margins and the northern part of Afar (Fig.4.1). These granites display clear intrusive contact with the Neoproterozoic basement, Jurassic Limestone and old Trap Series basalts (Varet, 1978). Younger Miocene igneous rocks within the Afar Depression include the Mabla and Dalha Series (Fig.4.1). The Mabla Series consists of rhyolites and ignimbrites with minor intercalation of basalts (Vellutini, 1990). These were erupted along N–S trending vents (Varet, 1978; Vellutini, 1990). The Dalha Series is a basaltic sequence up to 800 m thick found inter-bedded with rare detrital sedimentary rocks and ignimbrites (Varet, 1978).

4.1.3 Pliocene–Pleistocene volcanic rocks

Pliocene–Pleistocene volcanic rocks cover most of the Afar Depression (Fig.4.1). They are by far the most important geological units in terms of coverage and preservation of igneous features and tectonic activities. The most significant series of these is the Afar Stratoid Series, which is separated from Dalha Series by a nonconformity suggesting a prolonged erosion period and reduced magmatic activity (Varet, 1978). The Stratoid Series covers more than 2/3 of the Afar Depression. About 2/5 of these volcanic rocks are basalts frequently found to be porphyritic, vesicular, and tholeiitic in their geochemical nature (Barberi et al., 1974; Varet, 1978). The thickness of the Stratoid Series reaches up to 1500 m with individual varying from 1 to 6 m (Varet, 1978; Tefera et al., 1996).

4.1.4 Quaternary volcanic and sedimentary rocks

Quaternary volcanic rocks in the Afar Depression are composed of basaltic flows, scoria cones, and silicic rocks (Tefera et al., 1996). In most places, basaltic fissure eruptions were followed by central eruptions that produced differentiates of basalt comprising alkaline and per-alkaline silicic rocks (Varet, 1978; Tefera et al., 1996). However, the rift-parallel axial ranges (within the SE-propagating Manda Hararo–Gobaad and the NW-propagating Asal–Manda Inakir rifts) in the northern and east-central Afar are dominated by basalts, which are ~1 Ma old. The axial ranges are forced along fissures showing symmetric magnetic anomalies that are underlain by thin oceanic-type crust, and get progressively younger from the marginal zones towards the axial zones (Barberi and Varet, 1977). Because these show characteristics similar to mid-oceanic ridges, Barberi and Varet (1977) considered the axial ranges to be equivalents of oceanic spreading centers–NE, trending volcanic centers transverse the NW–SE rifts found along the eastern and western Afar margins (Fig.4.1). These transverse volcanic centers are associated with fracture zones equivalent to oceanic fractures and composed of alkali basalts with inclusions of peridotite nodules indicating deeper mantle source (Barberi and Varet, 1975, 1977). Central volcanoes called marginal centers are also found in the Afar margins (Fig.4.1). These are characterized by the occurrence of summit calderas and are mainly composed of trachytic and rhyolitic rocks (Varet, 1978). Lacustrine deposits dominate quaternary sedimentary rocks in the Afar Depression. Significant lacustrine sedimentary rocks were deposited in the central Afar along the Manda Hararo–Gobaad rift zones between _12 and 1 ka (Rognon, 1975). Lacustrine sedimentary rocks of _180–200 m thickness cover the Awsa plain in the east-central Afar (Fig. 4.1; Varet, 1978).

4.2 Geology of the area around Dam site

The Tendaho Dam is located in the central Afar, where East African, Red sea and Gulf of Aden rift system converge. The area is mainly comprises of volcanic rocks of sea-floor spreading. The basaltic rock covers the study area, which are flood basalts of Afar Group of the Ethiopian volcanic Series.

The physiography of the study area mainly comprises of elevated ridges of basaltic hills which forms steep to gentle slopes of the reservoir rim area or low lying plain areas. The width of the Awash river is characterized by wide flood plain, however at the proposed Dam site it becomes narrow (Fig.4.2). The geology of the Dam site and reservoir area comprises of the recent sediment of alluvium deposits on the flood plain.

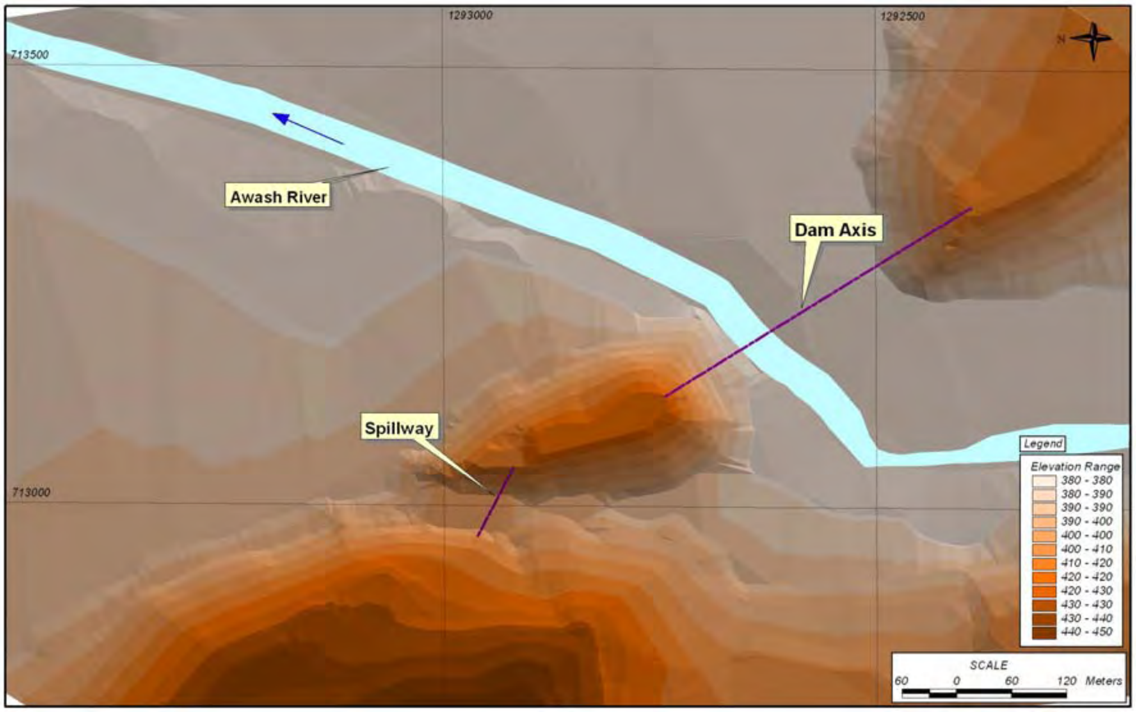


Fig. 4.2 Triangular irregular network (TIN) model of the Dam Site showing general topography of the area.

The local geology of the Dam site and the surrounding area can be expressed as; basaltic rock covers the abutments and recent alluvium and lake sedimentary rock covers the reservoir area. The abutment consists of series of vertical basaltic flows, which are separated by flatter slopes on the weaker pyroclastic layer. The slopes of the abutment are in the range of 20^0 to 28^0. The rocks exposed on the abutments consist of different types of basaltic rocks, which varies from; prophyritic, scoriaceous, vesicular and aphanitic types. These basaltic rocks show clear flow bandings and varied degree of weathering. The river section and most of the flood plain in the reservoir area is covered by the recent alluvium deposits, which are mainly comprises of gravel, sand, silt and clay. The lake sedimentary deposits, which are well exposed downstream of the Dam site, comprises of siltstone, mudstone, sandstone and conglomerate.

The river section of the Dam site is characterized by alluvium soils, which consists of gravel, sand, silt and clay. From the surface and subsurface exploration carried out at the dam site it may be noticed that three basic units are present at the dam site, these are, alluvium deposits in the river section, underlain by lake sedimentary and the basaltic bed rock. Fig. 4.3 presents the geologic setup of the dam site area and the cross section along the dam axis is presented through Fig. 7.1, Chapter 7. The geological structures around the dam site, as observed during the filed work, are present in Fig. 4.4. The general description of the various units present at the dam site is as follows;

4.2.1 Basaltic rocks

The basaltic rock sequence is characterized by light color, fractured, tilted, jointed and variation in degree of weathering. General trend of the basalt is NNW-SSE to NW-SE and dip mostly in the SW at 3^0 to 5^0. The volcanic sequence of basaltic rock is the major rock unit, which covers most of the elevated areas of the reservoir and the abutment slopes. The various type of basaltic rocks exposed on the abutments and on the higher reaches in the reservoir area are apanitic, porphyrtic, scoriaceous basalt, volcanic breccias and tuff. The tuff unit is well exposed in the upper portions of the ridges. The aphanitic basalts are comparatively fresh with local variations in weathering grades. The aphanitic basalts are highly jointed and characterized by open joints, filled with clay material. The scoriaceous basalt is moderately weathered and intensively jointed, with small openings exposed into the excavations. The porphyritic basalt, which is exposed at the foot of the abutments along the riverbank, is highly weathered and moderately jointed.

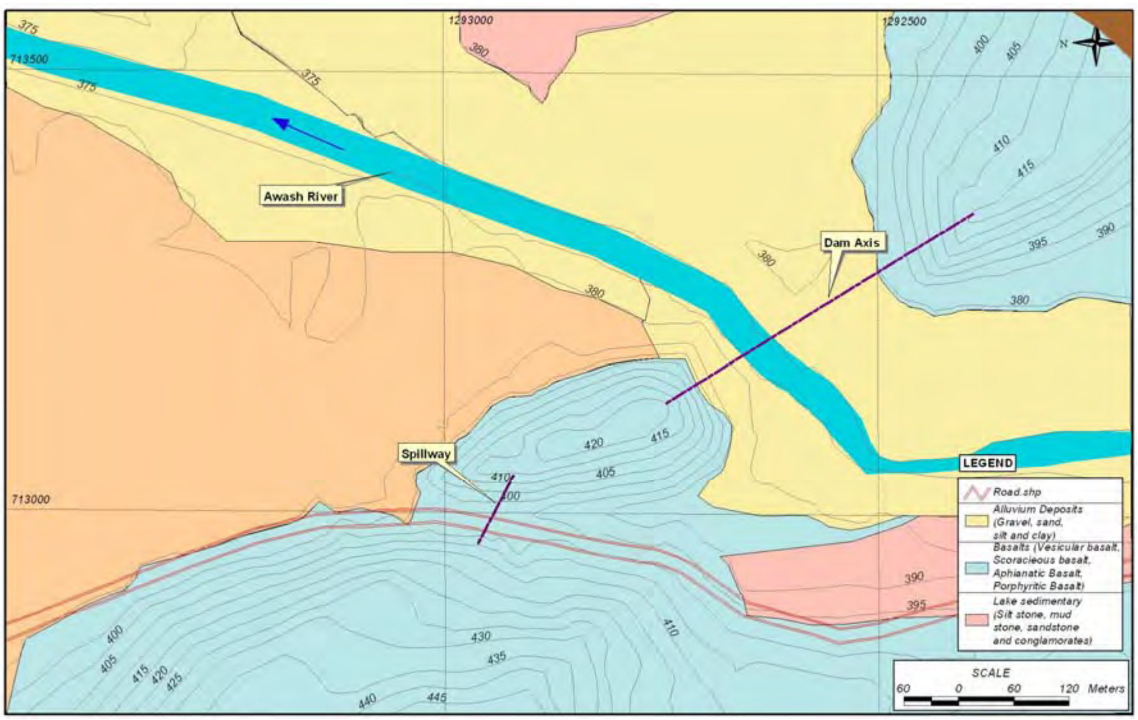


Fig 4.3 Geological Map of the dam site

The volcanic breccia and tuff is present in between the aphenitic and vesicular basalts in the upper reaches of the abutments. These rocks are highly weathered and very poor in strength.

4.2.2 Lake sedimentary rock

The lake sedimentary rock is exposed downstream of the dam site and along the newly constructed highway (around 6km from the dam site in upstream direction). The major units are conglomerate, sandstone, siltstone and mudstone, which are well exposed along the down stream of the dam in the tunnel outlet. This unit is observed downstream of the dam site and upstream (Fig. 4.3). The boreholes drilled in the Dam axis at the river section indicate the presence of sedimentary rocks under the alluvium deposit. The thickness of this unit is 20m.

4.2.3 Recent Alluvial deposits

The reservoir area is bounded by faulted basalt and in filled by alluvium deposits. The alluvium deposits such as gravel, sand, silt and clay cover the low-lying area of the reservoir and along the maximum flood plain of the river. The aerial coverage near to the dam site is governed by the topography. The lateral extension along the left bank is estimated to be 50m and on the right bank it is around 300m, which is extending upstream and downstream. (Fig 4.3). The thickness of this unit is in the range of 4 to 10m.

4.3 Seismcity of the Study Area

The Tendaho dam site lies in the central Afar of highly complex faulted zone. In this area the East African, the Red Sea and the Gulf of Aden rift system radiate. Many faults scarps are of recent date and the area is seismically active .The major faults have the same direction as Red sea, North-West to South –East direction but they are crossed by East African Rift faulting of NNE to SSW trend and other faults. In Tendaho area the major faults trends North to south and Northwest – Southeast direction (Fig4.3) .The Awash river flows along NNE fault direction and Tendaho graben are supposed to be formed by the NNW tensional faults and it form the SW boundary of the structures. The E-W faults are reported in the saddle on the right abutment of the Dam and North-South faults on the left abutment which are following rift valley trend (Gibb,1975).

In general, geological structures of the study area are controlled by tectonic activity along the Ethiopian rift, Red Sea and Gulf of Aden. In relation to these geological structures the area has been seismically active.

The method of assessing likely earthquake intensity and frequency at a given site are complex, required reasonable judgment and collection of geological and seismic data .

Fig. 4.4 Major faults around the dam site area

Due to this complexity, for structure with lesser magnitude, the tendency is to relay upon seismic risk map. The map area often published in national building codes with recommended the engineering precaution to be taken in each rank of hazard shows 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 deposited 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 (Fig. 4.5) 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 Johnson and Degraff (1988), these seismic intensity zones are related to the ground acceleration as follows;

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

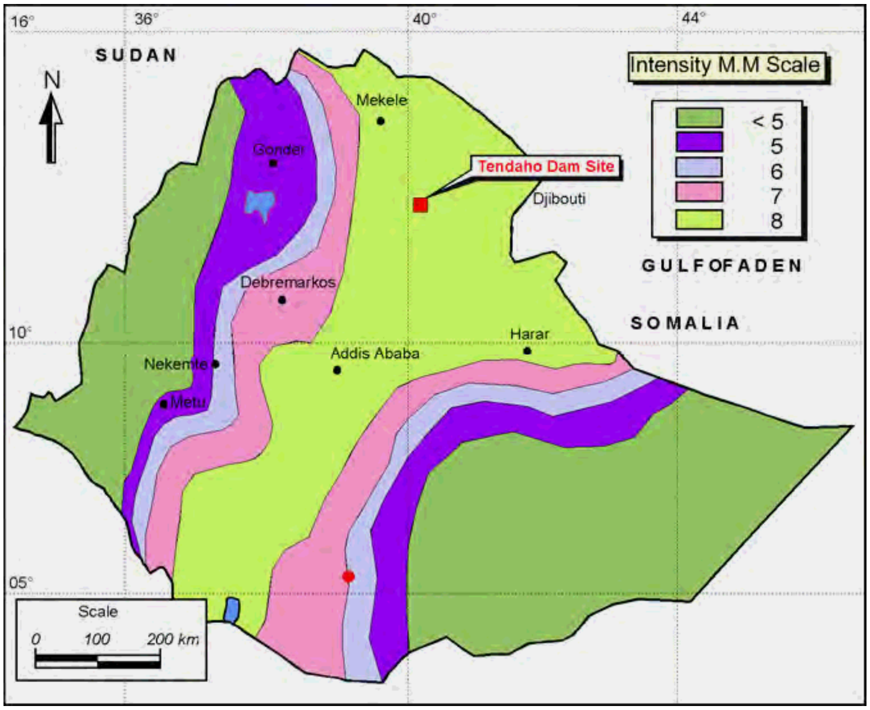


Fig 4.5 **Seismic risk map of Ethiopia 100 years return period, 0.99 probability.**
(After Laike Mariam Asfaw, 1986)

The Tendaho Dam Project area falls in the intensity scale 8, thus accordingly the estimated ground acceleration as per Johnson and Degraff will be 0.2g. The intensity scale 8 indicates that the project area lies in the high seismic risk zone. The Project Authorities was consider 0.18g factor in the design of the dam.

4.4 Rock weathering

Weathering is the process by which the rock disintegrates to form soil by physical or chemical processes. Weathering plays a significant role in altering or controlling the engineering properties of intact rock as well as rock mass. Therefore, the depth and degree of weathering are important parameters to be considered while deciding the stripping limit for laying the foundation. As observed during the present study the rocks exposed on the abutments are affected by different degree of weathering. Along the core trench the porphyritic basalts, which is found at the bottom, is highly weathered. Whereas, the scoriaceous basalt is slightly to moderately weathered. The aphanitic basalts exposed on the abutments are comparatively fresh with variation in grade at certain places. The tuff unit found on the top is moderately weathered.

Chapter V ENGINEERING GEOLOGICAL APPRAISAL OF THE DAM FOUNDATION

5.0 Preamble

The rock mass strength is greatly affected by the presence of discontinuity planes and the degree to which they are weathered. Thus, the performance of the rock as foundation material greatly depends upon these properties. The term 'soil' in soil engineering is defined as an unconsolidated material, composed of solid particles, produced by the disintegration of rocks. Soils are formed by weathering of rocks due to mechanical disintegration or chemical decomposition (Arora, 1997).

Tendaho Dam site is located in area where the river Awash passes through relatively narrow valley bounded by two ridges. The Dam site is covered partly by rocks and partly by soils in the abutments and the river section. The rocks exposed on the abutments are mainly basalts, which are intruded by the structural discontinuities like, joints, faults and contact joints. The rock at places is sheared with varied degree of weathering. Most of the soils covered in the project area are transported and deposited by the Awash River.

5.1 Engineering Geological Characterization of Rocks

In order to evaluate the suitability of the rock mass as a foundation material it is essential to characterize the rock mass based on certain engineering properties like strength, deformability and permeability characteristics of the rock mass. For the present study the rocks exposed in the foundation area were classified using 'rock mass classification system' (RMR). The strength and the deformability characteristics of the rock mass were evaluated based on its RMR value. Further, available bore hole data and the water pressure tests were utilized to characterize the sub-surface rock mass in the foundation area.

5.1.1 Geological Description

The foundation of Tendaho Dam comprise of five parts, left abutment, valley floor on the left bank, the river channel bed, valley floor on the right bank and right abutment. The left abutment ridge is occupied by volcanic sequence of basaltic rocks. These rocks are highly jointed, moderately weathered and contains inter bedded palesoil layers. The river floor on the left bank is covered by top soil and riverine alluvium. The thickness of the alluvium along the left bank is about 7m. The river channel is covered by recent alluvium of silty sandy soils, which has a thickness of about 13m. Along the right abutment in the river section thick alluvium deposits are present, which extends in the upstream direction upto 300 m. The depth of this alluvium deposit is about 15 m along the dam axis in the river section and it decreases towards the right abutment. On right abutment basaltic rock is exposed which is highly jointed and moderately weathered. A detailed description on geology of the dam site area has already been presented in Chapter-4.

5.1.2 Discontinuities

The important discontinuity property or character/factors are orientation, spacing, continuity, surface characteristics, the separation of discontinuity surface and the thickness and nature of the infilling material. Strength of rock mass greatly depends upon the shear strength of the discontinuity surfaces and also on the properties/factors viz, orientation, spacing, continuity, surface characteristics, the separation and the infilling material. Detail assessment of joint condition is very important for the evaluation of the Dam foundation as it may influence the foundation conditions in the following ways;

i) Permeability of rock mass is controlled by the presence of discontinuities.
ii) The system of discontinuities promotes weathering.
iii) Stability of the rock mass is influence by the orientation of discontinuities.
iv) The rock mass strength and quality is also affected by the spacing of the discontinuities.
v) The surface characteristics of discontinuity influence the shear strength of the rock mass.
vi) The separation and filling of discontinuity may have profound influence on the strength and permeability of the rock mass.

Left Abutment

Right abutment

Contact between tuff and basalt

Contact zone

Plate 5.1 Core trench excavated exposure joint, fractures and contact

In the present study discontinuities data has been collected from the excavated core trench in the aphanitic, vesicular and porphyritic basalt in the left and right abutments. The preferred orientation of the discontinuity planes as observed are presented in Table 5.1

Table 5.1 Preferred orientations, as observed on Left and Right Abutments

Location	***Preferred Orientation of Discontinuity Planes Dip Direction/ Dip Amount***				
	J1	***J2***	***J3***	***J4***	***J5***
Right Abutment	N240°/8°	N70°/8°	N30°/6°	N190°/10°	N100°/3°
Left Abutment	N264°/6°	N38°/14°	N114°/8°	N75°/10°	-

5.1.3 Weathering

Weathering is the process by which the rock disintegrates to form soil by physical or chemical processes. Weathering plays a significant role in altering or controlling the engineering properties of intact rock as well as rock mass. Therefore, the depth and degree of weathering are important parameters to be considered while deciding the stripping limit for laying the foundation. As observed during the present study the rocks exposed on the abutments are affected by different degree of weathering. Along the core trench the porphyritic basalts, which is found at the bottom, is highly weathered. Whereas, the scoriaceous basalt is slightly to moderately weathered. The aphanitic basalts exposed on the abutments are comparatively fresh with variation in grade at certain places. The tuff unit found on the top is moderately weathered.

5.2 Rock Mass Classification of Dam Foundation

Rock mass classification is a widely used, economical and extremely useful basis for determining engineering properties. 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 classification do. The stability and deformability of the structures 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 presence 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.

Numerous methods have been developed to guide the judgments of rock qualities for engineering purposes.

During feasibility, preliminary design and excavation /construction stages when very limited information on rock mass properties is available, the use of rock mass classification system can provide considerable in put on engineering properties of the rock mass. Using rock mass classification schemes it is possible to estimate support requirement during excavation and estimate strength and deformation properties of the rock mass.

The rock mass strength and deformability governed by the presence of discontinuity planes joint, fault, and bedding planes, in addition the spacing and filling material also significant effects the engineering properties of the rock mass. Though number of rock mass classification schemes are available however, classification system proposed by Bieniawiski is widely used.

Geomechanics classification or Bieniawski or rock mass rating (RMR) consider the following six parameters to classify rock mass;

i) Uniaxial compressive strength of rock (UCS)
ii) Rock Quality Designation (RQD)
iii) Spacing of discontinuities
iv) Condition of discontinuities
v) Ground water condition
vi) Orientation of discontinuities

For the present study the data pertaining to RMR has been collected from both abutment along the excavated core trench at several locations (Fig. 5.1) and based on the conditions the ratings for each of the 6 parameters were assigned from the standard RMR table and are added to get the RMR value.

The uniaxial compressive strength (UCS) has been determined by Schmidt Hammer and the Barton and Choubey, 1977, relation has been used to work out UCS from the Schmidt rebound number;

$$\log_{10}(\sigma_c) = 0.00088\gamma R + 1.01 \qquad \text{........ 5.1}$$

Where; 'σ_c', uniaxial compressive strength in Mpa, 'γ' is the Dry rock density in KN/m^3 and 'R' is the Schmidt rebound number.

The Rock Quality Designation (RQD) has been determined by Palmstorm's Volumetric Count method (Palmstrom, 1982), according to which,

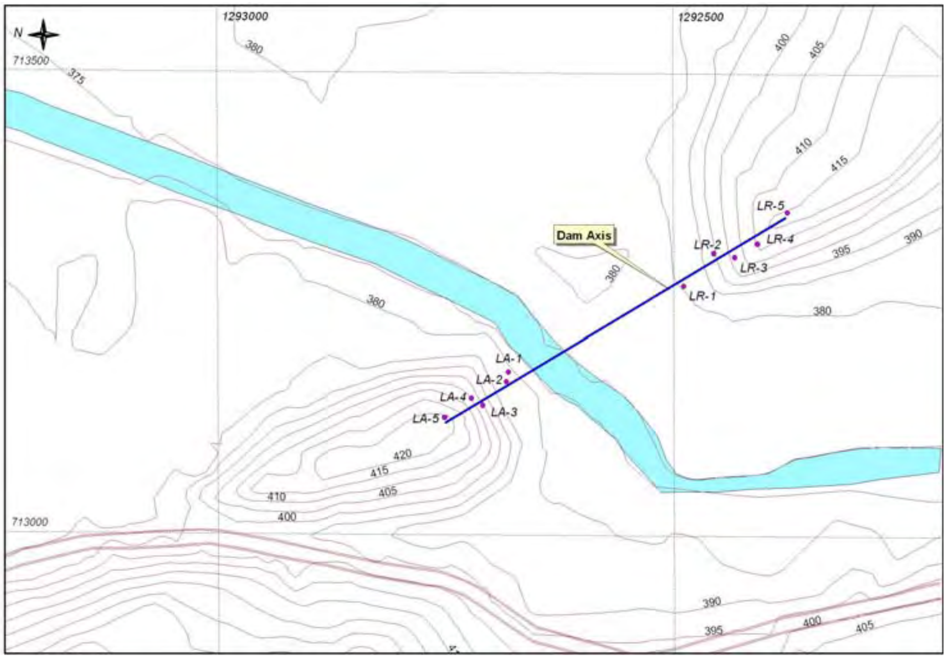


Fig. 5.1 RMR data collection location points

RQD = 115 – 3.3 Jv ……… 5.2

Where; 'RQD' is the Rock Quality Designation (%), 'Jv' is the total number of discontinuities greater than 10cm in length in 1m cube of Rock mass.

For other parameters, like; Spacing, Condition of discontinuities, Ground water condition, and Orientation of discontinuities visual observations and measurements have been made and accordingly, the ratings were assigned from the standard RMR table. The RMR data collected from various locations is summarized in Table 5.2. Based on the RMR data the rocks exposed on the abutments may be classified as fair rocks.

5.3 Engineering properties of rocks

In civil engineering structures rocks are used as construction and foundation material. As far as the strength of rock is considered it is strong and competent enough to carry the load applied without much changes as compared to soils. However, if the rock mass is jointed, fractured and weathered it may be deformed or displaced when load or structures is constructed on it. The important engineering properties of rock used for design of civil structure such as Dams are rock mass deformation, rock mass shear strength, rock quality designation and permeability of rock mass.

Table 5.2 RMR data collected from various locations on the Abutments

RMR Data points	***Parameters Ratings***										***RMR***	***Rock Mass Class***
	UCS (Mpa)			***RQD***			***Sp.***	***Con***	***GWC***	***Ori***		
	SHV	***UCS***	***Ra***	***Jv***	***RQD***	***Ra***						
Left Abutment												
LA1	25	39.81	4	11	78.7	17	8	20	10	-5	54	Fair
LA2	30	46.77	4	12	75.4	17	10	10	15	-5	51	Fair
LA3	18	25.46	4	15	65.5	13	10	10	15	-5	47	Fair
LA4	45	100.0	7	16	62.2	13	10	10	15	-5	50	Fair
LA5	60	214	12	14	68.8	13	15	20	15	-5	70	Good
Right Abutment												
RA1	35	60.25	7	15	65.5	13	8	10	15	-5	48	Fair
RA2	50	128.8	12	8	88.6	17	10	10	15	-5	49	Fair
RA3	55	165.9	12	10	82.0	17	10	10	15	-5	59	Fair
RA4	25	36.3	4	10	82.0	17	8	10	15	-5	49	Fair
RA5	40	77.62	7	11	78.7	17	8	10	15	-5	52	Fair
UCS – Uniaxial compressive strength, SHV – Schmidt hammer Value, Ra – Rating, Jv- Volumetric count, RQD – Rock quality designation, Sp. – Spacing of discontinuity, Con. – Condition of discontinuity, GWC – ground water condition, Ori – Orientation of discontinuity, RMR – Rock mass rating												

5.3.1 Rock Mass Deformation

The closure of discontinuities, plastic and elastic deformation of the intact rock that comprises rock mass, under applied static or dynamic load is known as rock mass deformation. The deformation of rock mass can be measured using the modulus of deformation E_d .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 (elastic)(Jonson, 1988). In the present study Modulus of deformation 'Ed' of the rock Mass has been determined empirically by using relation proposed by Agarwell et al (1991). According to Agrawell et al modulus of deformation can be expressed in terms of 'RMR'as;

$$Ed = 10^{(RMR-30)/50} \qquad \text{……….} \; 5.3$$

The modulus of deformation of the rock mass exposed at the foundation area is presented in Table 5.3.

5.3.2 Rock mass Shear strength

All rock masses contain discontinuities like joint, shear zones, bedding and contact zones. At shallow depth, where stresses are low failure of the intact rock material is minimum and the behaviour of the rock mass is controlled by sliding on the discontinuities (Hoek and Bray, 1981).

Table 5.3 Shear strength Parameters and Modulus of Deformation 'Ed' as determined from RMR

S.No	***RMR Data Location Elevation***	***RMR***	***Shear strength Parameters***				***Modulus of Deformation 'Ed' in (kg/cm^2) as determined by equation 5.3***
			C (Range)	***Φ (Range)***	***C****	***Φ****	
Left Abutment							
LA1	381	54	200-300	25 - 35	2.7	32	3.02
LA2	383	51	200-300	25 - 35	2.55	30.5	2.63
LA3	400	47	200-300	25 - 35	2.35	28.5	2.18
LA4	405	50	200-300	25 - 35	2.5	30	2.51
LA5	420	70	300-400	35 - 45	3.5	40	6.3
Right Abutment							
RA1	380	48	200-300	25 - 35	2.4	29	2.29
RA2	395	49	200-300	25 - 35	2.45	29.5	2.4
RA3	400	59	200-300	25 - 35	2.95	34.5	3.9
RA4	410	49	200-300	25 - 35	2.45	29.5	2.4
RA5	415	52	200-300	25 - 35	2.6	31	2.75
C is the specific value of Cohesion for a given RMR as determined using Eq.* ***5.4*** *Φ* is the specific value of angle of friction for a given RMR as determined using Eq.* ***5.5***							

For the stability of rock mass shear strength parameters, namely cohesion and angle of friction, are very important as they provide the resistance to the sliding of rock mass under the influence of gravity. The shear strength parameters at different location along the left and right abutment have been determined from RMR. The Shear strength parameters of rock mass as determined from RMR are present in Table 5.3. The standard RMR table gives the range in which the cohesion and angle of friction of the rock mass will fall. Further, in order to get the cohesion and angle of friction for a specific value of RMR, Bieniawiski, 1976 has proposed the following relations.

$$C^* = 0.05\ RMR \qquad \text{………. 5.4}$$

$$\Phi^* = 0.5RMR + 5 \qquad \text{………. 5.5}$$

C^* and Φ^* are the specific value of cohesion and angle of friction for a given RMR value.

Shear Strength of the Rock mass by Hoek and Brown Failure Criteria

For the present study an attempt has been made to empirically estimate the strength of the rock mass. For this purpose the Hoek and Brown (1980) technique has been utilized. Hoek and Brown (1980) developed an empirical approach to determine the strength of the jointed rock mass and formulated a failure criterion for jointed rock mass. Based on results of number of projects this criterion was modified by Hoek & Brown in 1988 and later by Hoek et al. (1992). For this empirical method Hoek and Brown utilized Bieniawiski's Rock Mass rating System (RMR) to work out the material constants. The Hoek-Brown criterion for jointed rock mass is;

$$\sigma_1' = \sigma_3' + \sigma_c \left(mb \frac{\sigma_3}{\sigma_c} + S \right)^a \qquad \ldots\ldots\ 5.6$$

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

For the present study RMR, collected from various locations (Table 5.2 and Fig.5.1) on the abutments has been used as an input parameter for strength estimation. The uniaxial compressive strength has been determined by Schmidt hammer at all representative sites from where RMR has been collected. The value of material constant 'mi' has been directly adopted from the standard table (Hoek and Brown, 1980). The other material constant mb and S are determined by using GSI as;

Average RMR = 52.9, m_i = 17 (for Basalt), GSI = RMR_{89} – 5 = 52.9 – 5 = 47.9

$$\frac{m_b}{m_i} = \exp\left(\frac{GSI - 100}{28}\right), \quad S = \exp\left(\frac{GSI - 100}{9}\right), \quad \mathbf{a} = \mathbf{0.5}$$

The confining effective principal stresses (σ_3') considered for the determination of major principal stresses (σ_1') and the corresponding computed major principal stresses (σ_1') are shown in Table 5.4. Later the shear strength parameters were determined by utilizing the Mohr-Coulom's diagram (Fig.5.2).

$$\sigma_1' = \sigma_3' + \sigma_c \left(m_b \frac{\sigma_3'}{\sigma_c} + S \right)^{0.5} \qquad \ldots\ldots\ldots\ 5.7$$

Thus the shear strength parameters of the rock mass as determined from Hoek and Brown Failure criteria are presented in Table 5.4. The average representative value for cohesion is 2.9 MPa and for angle of friction is 38^o.

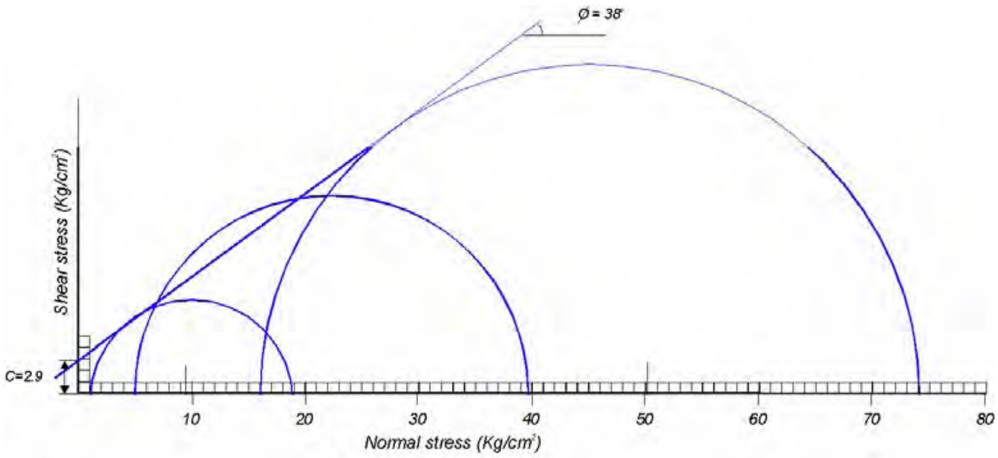


Fig. 5.2 Average shear strength parameters for rocks exposed at dam site.

5.3.3 Rock Quality Designation (RQD)

Rock quality designation is the percentage obtained by dividing the summed lengths of all core pieces equaled to or greater than 10cm long by the core interval length (Johnson, 1988). Rock mass quality is an important geological factor for design and construction of a dam project. The rock mass quality is highly influenced by the presence of geological structures; such as joints, faults and shear zone in addition to the grade of weathering.

There have been notable attempts for accurate rock mass quality classification, which not only evaluate the properties of rock masses and rock engineering geological properties in the dam site, but also judge the range of utilizable rock masses around the dam and determine the reasonable excavation depth of the dam foundation. The two well-known rock mass classifications are Rock Quality Designation (RQD, Deere, 1968) and Rock Mass Rating (RMR, Bieniawski, 1973). The quality of the rock mass, as observed from the drill hole data (Table 5.5), is better in left abutment sections as compared to the quality of the rock mass in right abutment sections. The quality of rock mass is comparatively better in the riverbed section. During the present study, RQD has also been determined by using Palmstorm's empirical approach. The result thus obtained indicates that on surface the quality of rock mass is comparatively better than what observed through bore holes (Table 5.2).

5.4 Permeability of the Dam Foundation

Permeability is an important parameter in designing of a dam project. During the feasibility stage the Project Authorities carried out a total of 24 permeability tests. One falling head permeability test was conducted in the alluvium deposit and the rest 23 tests were conducted

Table 5.4 Major Effective Principal Stresses as determined by Hoek and Brown Failure Criteria.

RMR	UCS (Mpa)	Material constants		Confining effective stress(σ_3) (Mpa)	Major effective Axial stress(σ_3) (Mpa)	Shear strength Paramete	
		mb	S			Cohesion (Mpa)	Angle of friction (ϕ) (º)
Left Abutment							
54.0	39.81	2.751	0.0035	1.0	11.72	2.0	36
54.0	39.81	2.751	0.0035	5.0	28.52		
54.0	39.81	2.751	0.0035	10.0	43.17		
51.0	46.77	2.471	0.0025	1.0	12.00	2.1	33
51.0	46.77	2.471	0.0025	5.0	29.15		
51.0	46.77	2.471	0.0025	15.0	56.70		
47.0	25.46	2.142	0.0016	1.0	8.45	2.2	34
47.0	25.46	2.142	0.0016	5.0	21.54		
47.0	25.46	2.142	0.0016	15.0	43.62		
50.0	100.00	2.384	0.0022	1.0	17.14	2.3	35
50.0	100.00	2.384	0.0022	5.0	39.85		
50.0	100.00	2.384	0.0022	10.0	59.06		
70.0	214.00	4.871	0.0205	1.0	45.49	2.3	34
70.0	214.00	4.871	0.0205	10.0	116.59		
70.0	214.00	4.871	0.0205	18.0	158.35		
Average Values For Left Abutment							
54.4	85.208	2.790	0.0036	1.0	17.25	**2.3**	**35**
54.4	85.208	2.790	0.0036	7.0	48.11		
54.4	85.208	2.790	0.0036	15.0	74.94		
Right Abutment							
48.0	60.25	2.220	0.0018	1.0	12.84	2.9	34
48.0	60.25	2.220	0.0018	5.0	30.98		
48.0	60.25	2.220	0.0018	12.0	52.14		
49.0	128.80	2.301	0.0020	1.0	14.55	2.8	35
49.0	128.80	2.301	0.0020	5.0	43.92		
49.0	128.80	2.301	0.0020	6.0	48.55		
59.0	165.90	3.288	0.0060	1.0	27.67	2.7	36
59.0	165.90	3.288	0.0060	5.0	58.79		
59.0	165.90	3.288	0.0060	10.0	84.97		
49.0	36.30	2.301	0.0020	1.0	10.28	2.8	35
49.0	36.30	2.301	0.0020	5.0	25.50		
49.0	36.30	2.301	0.0020	13.0	45.99		
52.0	77.62	2.561	0.0028	1.0	15.68	2.8	34
52.0	77.62	2.561	0.0028	5.0	36.79		
52.0	77.62	2.561	0.0028	14.0	66.91		
Average Values For Right Abutment							
51.4	93.77	2.507	0.0026	1.0	17.06	**2.9**	**39**
51.4	93.77	2.507	0.0026	5.0	39.61		
51.4	93.77	2.507	0.0026	15.0	74.57		
Overal Average for the Dam Site							
52.9	89.49	2.645	0.0031	1.0	18.16	**2.9**	**38**
52.9	89.49	2.645	0.0031	5.0	39.75		
52.9	89.49	2.645	0.0031	16.0	77.73		

in the rock mass sections. The permeability of the rock mass may be assessed by following various techniques however, Lugeon's criteria is most commonly used as it gives more reliable values for the first orientation. According to Lugeon, for dams higher than 30m, the water loss in water pressure test should not exceed 1 liter in 1 min. per 1 meter of the hole at 10 atmospheres pressure which should act at least for 10 min.

Permeability results are described in terms of Lugeon units; one Lugeon is equal to a flow of 1 lit/m/min at pressure of 1 MN/m2. A Lugeon unit is approximately equal to a coefficient of permeability of 10^{-7} m/s. According to Lugeon (1933) a rock absorbing less than one Lugeon unit can be considered as watertight. The results of permeability tests on rock units are shown in Table 5.5. A perusal of Table 5.5 shows that permeability of the rock mass is high to very high in the Abutment sections of the dam.

Table 5.5 Water Pressure test and Rock Quality Designation

Bore hole number	**Location**	**Elevation (m.a.l)**	**Test conducted depth (m)**	**RQD (%)**			**Perme-ability (Lugeon)**	**Remarks on Permeab-ility ****
				Min.	**Max**	**Average**	**Average**	
Abutment Section								
BH-TT2	Left Abutment	421.73	23.75-28.57	36	65	50.5	245	Very High
BH-TT2	Left Abutment		43.75-48.57	36	66	51	3.68	Low
BH-TT2	Left Abutment		54.3-59.25	37	61	49	135	Very High
BH-TT2	Left Abutment		61.05-64.05	42	87	64.5	331.67	Very High
BH-TT2	Left Abutment		66.1-70	20	90	59.5	24.73	Medium
BH-TE	Right Abutment	410.339	21.5-25.8	-	-	50	316	Very High
BH-TE	Right Abutment		44.85-49.70	-	-	72	3.85	Low
BH-TE	Right Abutment		50.7-55.7	-	-	72	89.5	Very High
BH-TE	Right Abutment		56.7-61.2	13	72	42.5	33.01	High
BH-TE	Right Abutment		64.85-70	13	30	21.5	168.16	Very High
River Section								
BH-TC	Left Bank	374.41	23.75-28.57	42	90	66	1.8	Impervious
BH-TC	Left Bank		29.75-34.75	42	67	54.5	1.75	Impervious
BH-TC	Left Bank		36-40			67	1.82	Impervious
BH-TD	Left Bank	374.15	23.75-28.57	25	66	45.5	0.39	Impervious
BH-TD	Left Bank		29.39-33.15	25	25	25	0.53	Impervious
BH-TD	Left Bank		37.2-42.2	24	66	45	3.14	Low
BH-TD	Left Bank		46.0-51.15	62	64	63	0.97	Impervious
BH-TD	Left Bank		66-70	11	63	37	4.11	Low
BH-TC1	Left Bank	377.791	24-29.2			72	2.16	Impervious
BH-TC1	Left Bank		32.6-36.2	22	83	52.5	0	Impervious
BH-TC1	Left Bank		36.2-40	83		83	5.26	Low
BH-TF	Right Bank	377.652	39-44	45	91	68	0.31	Impervious
BH-TF	Right Bank		61-65	37	56	46.5	4.29	Low
River section falling head test		-	On alluvial deposit <10	-			18	Medium

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

Whereas, the rock under the river section of the Dam foundation is comparatively impervious to low in its permeability. The top alluvium cover of the river section has shown medium permeability. Therefore, the abutments need a proper attention as far as the permeability is concerned. In order to improve the permeability condition along the abutment section of the dam foundation proper grouting by injection of cement is necessary.

5.5 Slope Stability of the Abutments

The slope stability of the abutments is very important for the safe functioning of any dam project. The instability of abutment slopes may pose serious problems during construction stage. Identification of instability of the slopes in the initial stages may help in evolving proper remedial measures. Therefore, it is essential to carry out the stability analysis of the abutment slopes in the initial stages of investigation and planning.

5.5.1 Discontinuity Analysis

Discontinuities are structural weakness planes upon which movement can take place. The presence or absence of discontinuities has a very important influence upon the stability of rock slopes and the detection of these geological features is most critical part of the stability investigation. (Hoek et al, 1977).

The rock mass exposed on the abutment slopes is traversed by discontinuity planes mainly, joints and faults. In order to work out the preferred orientations of these discontinuity planes, structural data, mainly, joints has been collected from both the abutment slopes. For each discontinuity plane the azimuth of the 'Dip-Direction' clockwise in relation to magnetic North from 0° to 360°, has been measured, whereas, amount of dip was measured along it’s true dip direction in the vertical plane.

Later, the structural discontinuity data has been stereographically analyzed. Fig. 5.3 and Table 5.1 presents the preferred orientations, as observed on both the abutment slopes.

5.5.2 Geometry and Geology of the Critical slope Section

For the detailed stability analysis cross sections has been prepared along the abutment slope sections. The geology and the geometry of slope sections in terms of slope direction and inclination, upper slope direction, inclination and the height of the slope are presented in Fig.5.4.

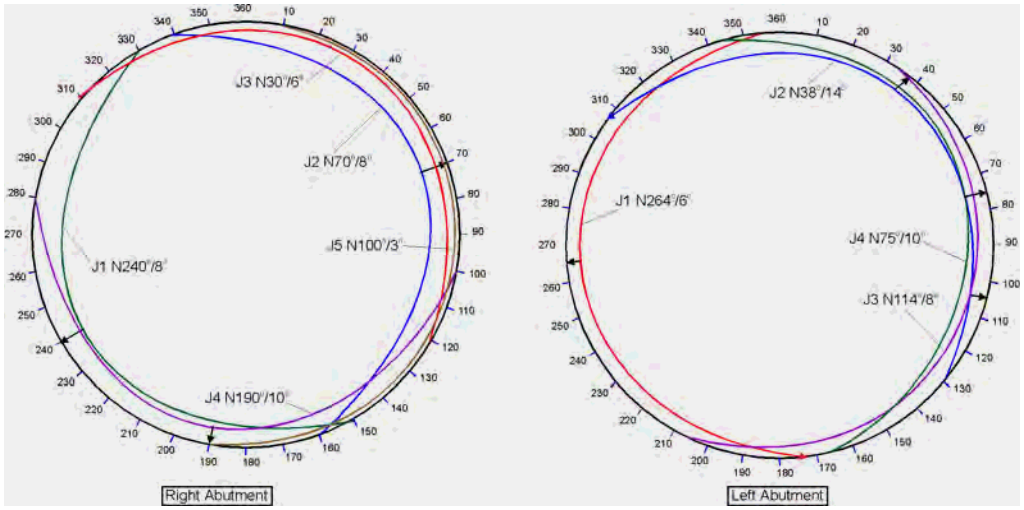


Fig. 5.3 Preferred orientation of discontinuity planes as observed on the abutments

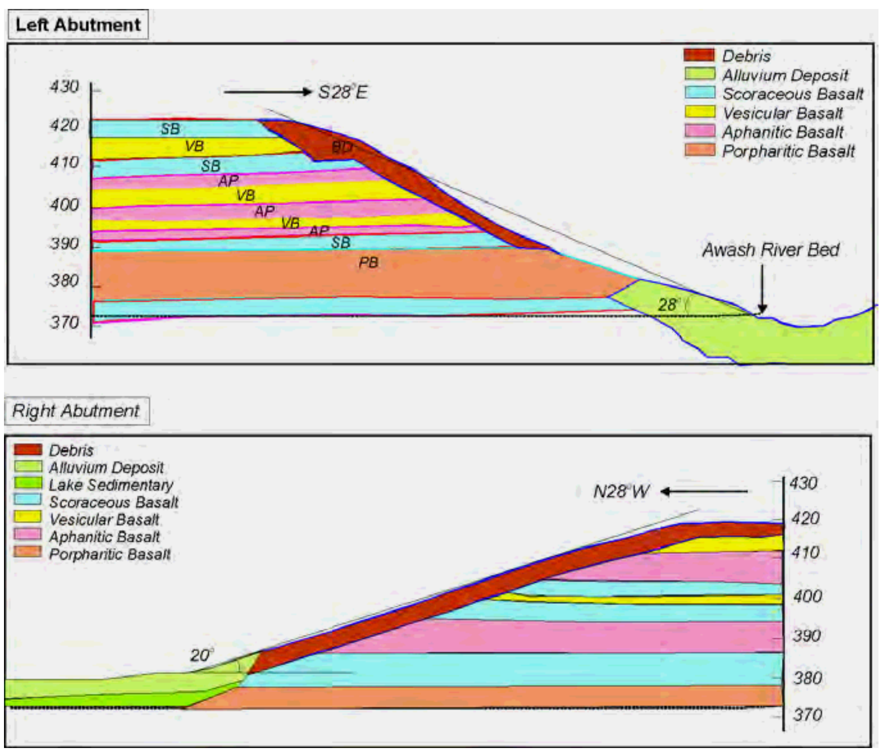


Fig. 5.4 Geometry and geological sections of abutment slopes along dam axis

5.5.3 Kinematic Check

Different types of slope failure are associated with different geological structures and it is important that the slope designer should be able to recognize the potential stability problems during the early stage of project. Markland developed a simple test which is designed to establish the possibility of a wedge failure in which sliding takes place along the line of intersection of two planar discontinuities. Plane failure is also covered by this test since it is a special case of wedge failure. If the contact is maintained on both planes, sliding can only occur along the line of intersection and hence this line of intersection must daylight in the slope face.

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

Plane failure $\alpha_f > \alpha_p > \phi$ ...5.8

Wedge failure $\alpha_f > \alpha_i > \phi$ ...5.9

Where:
α_f is the slope angle
α_p is the dip of the potential failure plane
α_i is plunge of the line of intersection
ϕ is the angle of internal friction of the two wedges forming plane

For the kinematics check, Markland test has been applied to the abutment slope sections. Structural data, along with slope inclination and a ‘phi circle’ corresponding to angle of friction of the rock mass has been plotted on equal area projection 'Schmidt Net'. The angle of friction has been estimated from the RMR data. Fig. 5.5 presents the stereo plots to demonstrate Markland test for abutment slope sections.

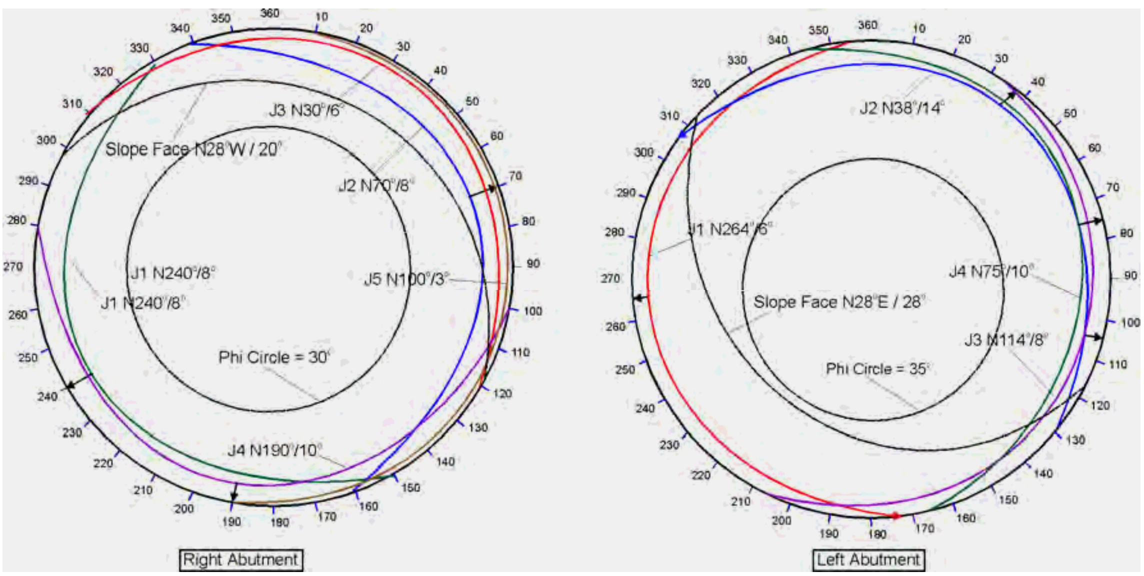


Fig. 5.5 Kinematics check for potential mode of failure in right and left abutment slopes

It has been found that both the slope sections do not satisfy the kinematics condition for plane or wedge mode of failure. Thus, there is no possibility of plane or wedge mode of failure in both the abutment slopes. However, there may be some localized block failures at left abutment, between El. 376 - 388 m as the rock mass is highly jointed within this zone.

5.6 Engineering Geological Characterization of Soils

Soils in general can be defined as a heterogeneous accumulation of solid grains that are not cemented together. For engineering purpose soils are subdivided based on distribution of various grain-sizes and the plasticity nature of the fine particles, as coarse grained and fine grained, respectively. A standard method is used for identification and classification of soil in to categories or groups represented by symbol that have distinct Engineering characteristics. This facilitates a common understanding of soil behavior just by knowing the classification. From the classification it is possible to estimate permeability, shear strength, volume change potential of soil when it interact with water, etc (ASTM, 1996).In addition to this during construction stage soil classification may be a useful guide in planning and estimation of compaction nature of soil, workability and equipment required for the excavation. For soil classification many standards are available however, for the present study ‘Unified Soil Classification’ (USCS) has been adopted.

For the present study six soil samples, three each from both the banks in river section has been collected for the laboratory testings.

5.6.1 Mechanical and Physical properties of soils of the Dam foundation.

5.6.2 Physical properties of soils of the Dam foundation

Index properties

Data obtained from index test together with description of visual observation are often sufficient for design purposes for minor structure. This information is used also in making preliminary designs for determining probable cost of a major structure and to limit the amount of detailed testing (ASTM, 1996). Atterburg limits and consistancey test are index properties for determination of fine-grained soil whereas for coarse gained, grain size distribution and relative density are the important index properties.

Grain-size Analysis

For the present study grain size analysis has been carried out on six samples collected from the river section in the dam foundation area. For the grain size analysis ASTM standard has been followed. The dry sieve analysis has been conducted in Tehadho Dam and Irrigation Project Laboratory and the Department of Engineering Geology Laboratory (A.A.U). For fine fraction analysis hydrometric test has been conducted in Ministry of Water Resources Laboratory, Addis Ababa. The results thus obtained are presented in Tables 5.6 and Fig. 5.6.

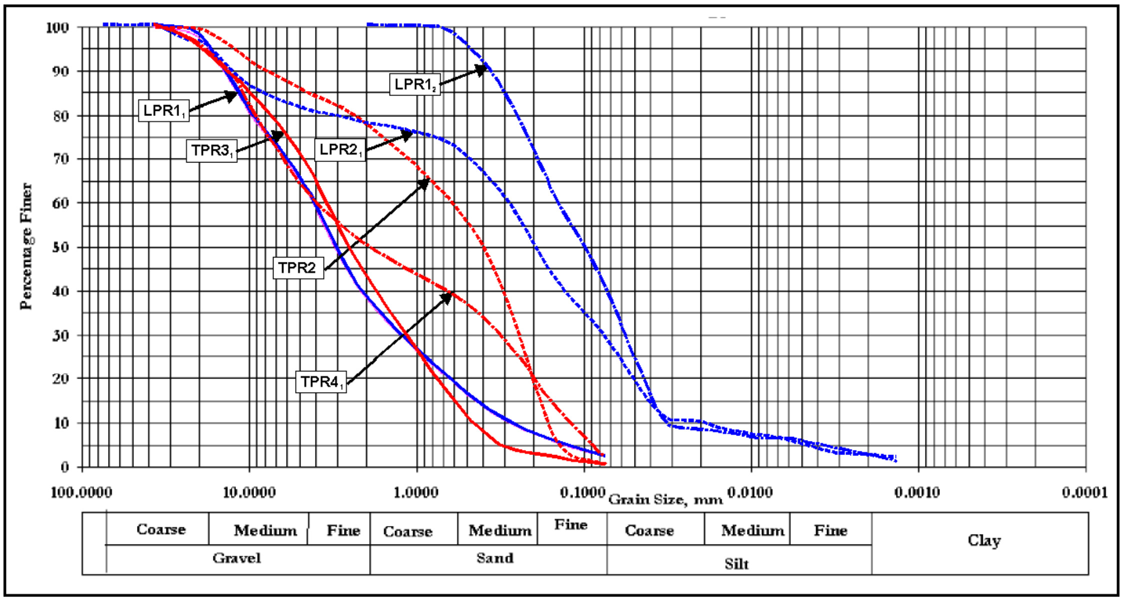


Fig.5.6 Gradation of the soils present in the dam foundation area

Table 5.6 Classification of soils of the foundation of the Dam based on USCS

Sample	Depth (m)	Location	Plasticity	Soil classification (USCS)	Fine content (%)	Coefficient of uniformity	Coefficient of curvature	Soil type
TPR2	0.5-1	Right bank	None	Sandy gravely soil	< 5	3.15	0.64	SP
$TPR3_1$	0.9-1.1	Right bank	None	Sandy gravely soil	< 5	7	0.82	SP
$TPR4_1$	2.1-2.9	Right bank	None	Sandy gravely silt soil	< 5	26.7	0.19	SP
$LPR1_1$	0.5-1	Left bank	None	Sandy gravely silt soil	< 5	13.3	1.63	SW
$LPR1_2$	1.5 – 2.9	Left bank	None	Sandy gravely silt soil	-	2.73	0.67	SP
$LPR2_1$	0.1-1.5	Left bank	None	Silty clay soil	-	4.53	0.82	SC

Atterburg Limits

Atterburg in 1911, gave an idea of the consistency of cohesive soils and proposed a number of tests for defining their properties, the tests are known as consistency tests. According to him the consistency of a soil can pass through liquid, plastic, semisolid and solid states depending on the presence of water in the soil. The Atterburg limits such as liquid and plastic limit is determined numerically in the laboratory. This provides a useful basis based on which fine-grained soils can be classified. For the present study the soil samples collected from the dam site shows that the soils are non-plastic.

A perusal of Table 5.6 indicates that the soils present in the river section, along the right bank in the foundation area, are classified as 'poorly graded sands' (SP) as per the Unified Soil Classification System. Whereas, along the left bank side there is a variation in the gradation as the soils are falling in SW, SP and SC classes.

5.6.3 Mechanical Properties of Soil of the Dam foundation

The suitability of a soil for a particular civil engineering structure depends on its engineering properties. Therefore the performance of any engineering work will depend on the correct assessment of engineering properties of soils. Two engineering properties of soils are particularly important for engineering works; these are compressibility and the shear strength of the soil.

For the present study sincere efforts were made to evaluate the properties of the foundation material based on the realistic field observations supported with experimental data. The tests required for the determination of engineering properties of the soils needs elaborate tests, which require sufficient time, adequate resource and financial support. Thus, all these factors made these elaborate tests beyond the reach of the present study. However, efforts are made to characterize the foundation soils by utilizing secondary data and the available empirical techniques.

Unconfined Compressive strength and Consistency of the Foundation Soils

From the standard penetration test result of the bore holes (Table 5.7) the unconfined compressive strength of the soils of the Dam site and its consistency is estimated and is presented in Table 5.8. A perusal of Table 5.8 indicates that the soils presents along left river bank are comparatively soft and have low unconfined compressive strength. However, this may be due to the fact that the test was conducted at shallow depth. As the depth of the test increase the unconfined compressive strength and consistency increases. In bore hole BH-TG_1 the penetration resistance is greater than 50, which indicates that the soils are hard and the unconfined compressive strength is greater than 400 kN/m^2. This indicates the compactness of the soils under the influence of overburden thickness.

Table 5.7 Standard Penetration Test Data in Dam Foundation Area

Location	Bore hole	Depth tested (m)	Layers	No. Blow for 150mm penetration	Consistency	Unconfined Compressive Strength (kN/m^2)	Angle of Shearing Resistance 'ϕ' Based on Table 5.10
Along Dam Axis right bank	BH_TF	6.46-6.91	1st	14	Stiff	100 -200	30^o – 40^o
			2nd	23	Very Stiff	200 - 400	30^o – 40^o
			3rd	42	Hard	> 400	35^o – 45^o
		9.75-10.25	1st	17	Very Stiff	200 - 400	30^o – 40^o
			2nd	19	Very Stiff	200 - 400	30^o – 40^o
Left bank upstream of axis	BH-TC$_1$	3.1-3.6	1st	2	Very Soft	< 25	25^o – 32^o
			2nd	3	Soft	25 - 50	25^o – 32^o
			3rd	6	Medium	50 -100	27^o – 35^o
Right bank downstream of axis	BH-TG	8.25-8.66	1st	22	Very Stiff	200 - 400	30^o – 40^o
			2nd	>50	Hard	> 400	> 45^o
Right bank upstream of axis	BH-TG$_1$	11-11.15	1st	>50	Hard	> 400	> 45^o

Table 5.8 Estimation of unconfined compressive strength and consistency

Consistency	Unconfined compressive kN/m^2	Penetration resistance (No. of blow)
Very soft	< 25	< 2
Soft	25 to 50	2 to 4
Medium	50 to 100	4 to 8
Stiff	100 to 200	8 to 15
Very Stiff	200 to 400	15 to 30
Hard	Over 400	Over 30

Compressibility, Shear Strength and Permeability of the soils

A perusal of Table 5.6 indicates that the soils present in the river section, along the right bank in the foundation area, are classified as 'poorly graded sands' (SP) as per the Unified Soil Classification System. Whereas, along the left bank side there is a variation in the gradation as the soils are falling in SW, SP and SC classes. Thus based on the classification a fair idea can be made about the engineering performance of these soils as the foundation material (Table 5.9).

Tables 5.9 Engineering performance of foundation Soils

Sample	Depth (m)	Location	Soil type	Engineering Performance of Foundation Soils***			
				Permeability	Compressibility	Shear strength	Workability
TPR2	0.5-1	Right bank	SP	Pervious	Very low	Good	Fair
TPR3$_1$	0.9-1.1	Right bank	SP	Pervious	Very low	Good	Fair
TPR4$_1$	2.1-2.9	Right bank	SP	Pervious	Very low	Good	Fair
LPR1$_1$	0.5-1	Left bank	SW	Pervious	Negligible	Excellent	Excellent
LPR1$_2$	1.52-2.9	Left bank	SP	Pervious	Very low	Good	Fair
LPR2$_1$	0.1-1.5	Left bank	SC	Impervious	Low	Good to fair	Good
**** Based on Criteria Proposed by Indian Standard Code System							

Shear Strength of the Foundation Soils

An attempt has been made to estimate the shear strength of the foundation soils by using the Standard Penetration test data. The angle of shearing resistance (ϕ) of the cohesion less soil depends upon the number N. In general, the greater the N-value, the greater is the angle of shearing resistance. Table 5.10 gives the average value of 'ϕ' for different ranges of 'N'-values.

Table 5.10 Standard Correlation between N-Value and angle of shearing resistance 'ϕ'

Standard Penetration Number 'N' value	Denseness	Angle of Shearing Resistance 'ϕ'
0 - 4	Very loose	$25^0 – 32^0$
4 - 10	Loose	$27^0 – 35^0$
10 - 30	Medium	$30^0 – 40^0$
30 - 50	Dense	$35^0 – 45^0$
> 50	Very Dense	$> 45^0$

5.7 Liquefaction of Foundation soils

The phenomenon when the sand loses its shear strength due to oscillatory motion is known as liquefaction of sands. The structures resting on such soils may sink. In the case of partial liquefaction, the structure may undergo excessive settlement and the complete failure may not occur. The soils most susceptible to liquefaction are the saturated, fine and medium sands of uniform particle size. When such deposits have a void ratio greater than the critical void ratio and are subjected to a sudden shearing stresses, these decrease in volume and the pore pressure increases. The soil momentarily liquefies and behaves as a dense fluid. Extreme care shall be taken while constructing structures on such soils. If the deposits are compacted to a void ratio smaller than the critical void ratio, the chances of liquefaction are reduced (Arora, 1997).

At Tandaho dam foundation Alluvium soil deposits are present in the dam foundation area. The deposits towards the right bank are thick and they extends over a large area in the upstream direction.

From the penetration resistance test the liquefaction potential of the foundation soils can be estimated. Seed and Idriss have given a plot of saturated penetration resistance 'N' against depth, with maximum ground acceleration as a third variable for a quick appraisal of the likelihood of liquefaction of a deposit (Fig. 5.7). From this plot for a known standard penetration resistance and depth, the maximum ground acceleration needed to cause liquefaction can be obtained.

A perusal of Table 5.9 indicates that soils in the foundation area may be liquefied under a ground acceleration greater than 0.15. Therefore, it may be advisable to remove the top soil upto a depth of about 7m. However, this assessment is based on an empirical technique proposed by Seed and Idriss. More detailed systematic studies would be required to take final decisions on this account.

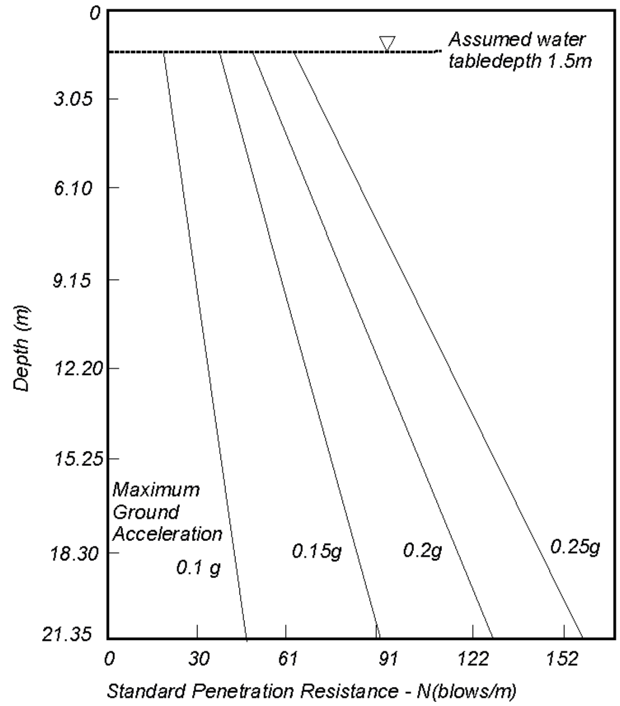


Fig. 5.7 Liquefaction potential versus saturated penetration resistance No. (Seeds and Idress, 1971)

5.8 Engineering Geological Mapping of The Dam Site

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

Table 5.11 Ground Acceleration causing Liquefaction based on Average Penetration Resistance ‘N’

Bore hole	Depth (m)	Average penetration resistance “N”	Ground acceleration causing Liquefaction (g)
BH-TC_1	3.1-3.6	4.5	< 0.1g
BH-TF	6.46-6.91	32.5	< 0.15 g
	9.75-10.25	19	< 0.1 g
BH-TG	8.25-8.66	>50	> 0.15 g
BH-TG_1	11-11.15	>50	> 0.15 g

The engineering geological mapping of the Tendaho Dam site has been carried out on the basis of engineering characterization of rock and soils. For mapping purpose the rocks have been characterized based on the strength, weathering grade and jointing intensity (RQD).

Whereas the soils of the dam site area has been classified based on the unified soil classification system. Fig. 5.8 presents the engineering geological map of the dam site.

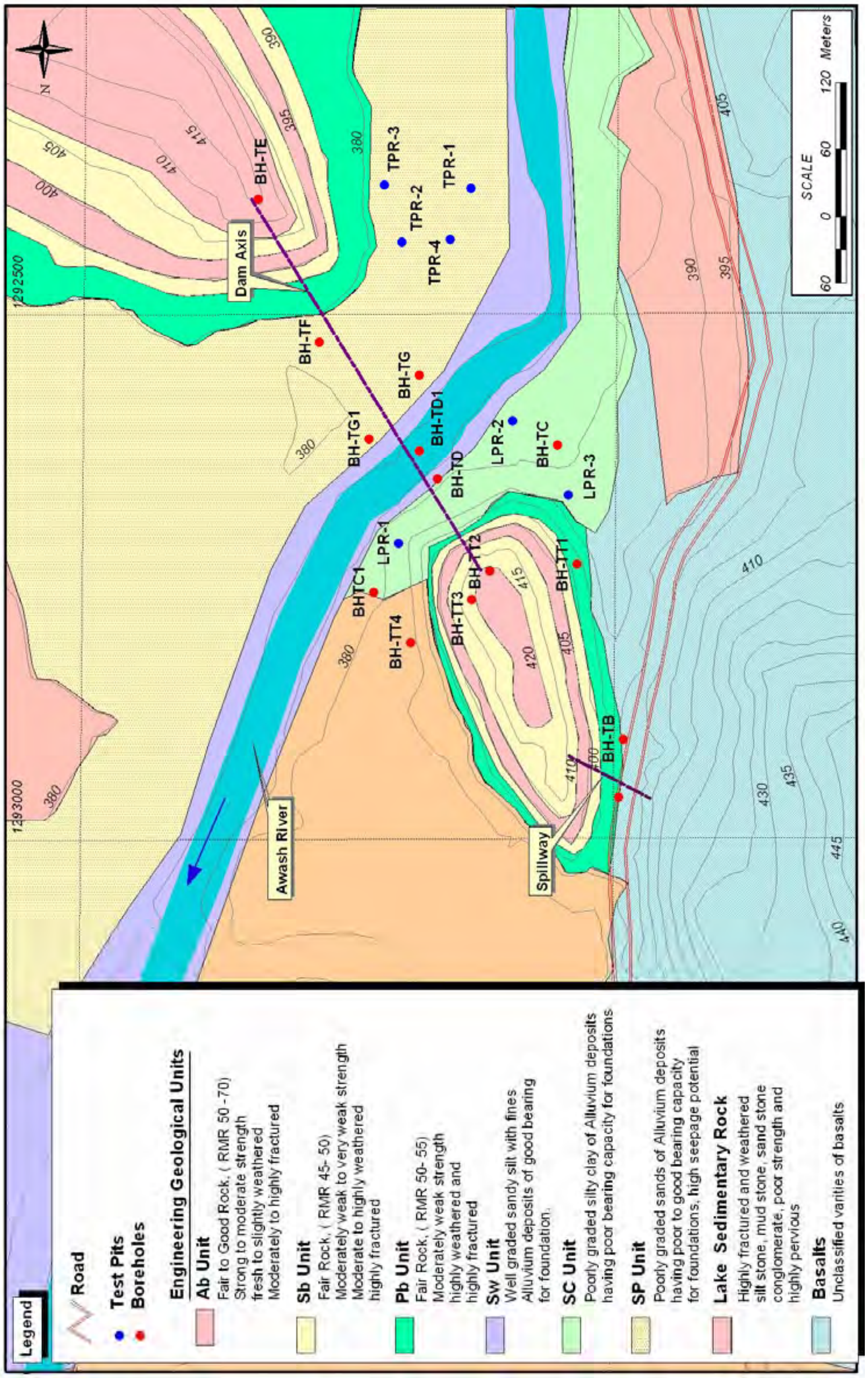


Fig. 5.8 Engineering Geological map of the dam site

Characterization of Rocks and Soils of the Dam site

For mapping purpose the rocks of the dam site have been characterized based on the strength, weathering grade, jointing intensity (RQD) and its geomechanical classification.

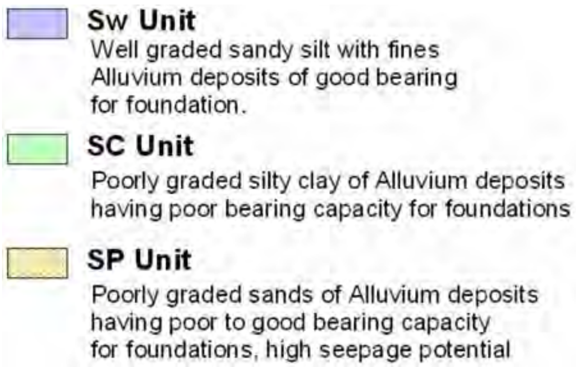
For the estimation of the strength of the rock Schmidt hammer has been used. To classify the rocks based on its strength, classification proposed by Hoek and Bray (1977) has been utilized (Table 5.10). The weathering grade and the joint intensity have been visually observed and accordingly qualitative assessment has been made. The RMR value has been determined based on Bieniawiski's Geomechanics classification system (Table 5.2). Thus based on the above criteria the rocks of the dam site has been classified into four classes;

Table 5.10 Classification of rocks based on strength (Hoek and Bray, 1997)

No.	Description	Rock Type	Uniaxial Compressive strength (UCS) MPa
R1	<u>*Very week rock*</u> – crumbles under sharp blows with geological pick point, can be cut with pocket knife	Chalk, rock salt	1.0 - 25
R2	<u>*Moderately weak rock*</u> – shallow cuts or scraping with pocket knife with difficulty, pick point indents deeply with firm blow.	Coal, schist, silt stone	25 - 50
R3	<u>*Moderately strong rock*</u> – knife can not be used to scrape or peel surface, shallow indentation under firm blow from pick point	Sandstone, slate, shale	50 - 100
R4	<u>*Strong Rock*</u> – hand held sample breaks with one firm blow from hammer end of geological pick.	Marble, granite,gneiss	100 - 200
R5	<u>*Very strong rock*</u> – requires many blows from geological pick to break intact sample.	Quartzite, dolerite, gabbro, basalt	> 200

Source: Hoek and Bray, 1997

The soils of the dam site area has been classified based on Unified Soil Classification System and the relative suitability of the material for engineering use; such as it's bearing capacity and potential for seepage. Thus based on this criterion the soils of the dam site has been classified into three classes;



Chapter VI EMBANKMENT DAM AND CONSTRUCTION MATERIAL

6.0 Introduction

The main criteria for the selection of an embankment type of dam are the existing geological conditions at the proposed dam site and the availability of suitable construction materials within the economic distance from the dam site. Out of these two criteria, the availability of the construction material within the economic distance has a greater influence over the techno-economic feasibility of the project. The availability of the construction material is a decisive factor for the embankment type, it may be a zoned dam, in which the dam section consists of central impervious core and the outer pervious material shell, or a rock fill dam. The rock fill dams are basically zoned sections with an impervious earth core for water tightness and rock fill material in the shells to provide stability to the dam. In addition other factors, which has an influence over the selection of type of dam are the geological conditions at the dam foundation, water tightness of the foundation and reservoir and the engineering geological properties of the rock and soils in the foundation area.

For Tenaho Dam Project different groups and individuals has conducted several studies in the past. Based on the techno-economic studies different types of Embankment Dams were recommended. Sogreah (1965) and Halcrow (1989) recommended to adopt a Rock fill dam. This selection of dam type was primarily based on the availability and suitability of the construction material. These workers supported their argument with a fact that the dam site lies in a zone of high seisemicity and the rock fill dam would be more stable than an earth dam under earthquake loading. Further, studies made by Alexander Gibb (1975) and UZGIPROVODKHOZ (1985) suggests that sufficient quantity of suitable material is available for construction of an earth dam in the borrow area at a reasonable distance from dam site. Both studies, therefore, recommends for the construction of a Zoned earth dam with Clay core. Based on previous studies and the recent studies conducted by the project authorities, zoned Earth fill dam has been finally considered for the construction at the Tendaho dam site.

6.2 Earth fill Dams

Earth fill dams have been used since the early days of development to store water for irrigation. The main advantage of earth fill dam is that it involves the utilization of the natural existing material near to the dam site with minor processing. The major disadvantage is the hydraulic failures including, failures due to overtopping and erosion of the slopes by waves, wind, rain and presence of tail water near to the downstream slope. Therefore, while selecting an earth fill type dam a proper care has to be taken while designing the free board, for which proper calculations are to performed. In addition the spillway have to be designed with sufficient capacity to overcome the peak flood. Besides, the proper care has to be taken for the upstream and down stream slope protection. The earth dam may be designed as a homogenous or zoned type depending upon the quality and the quantities of the various materials available in the borrow areas at economic distances.

6.2.1 Homogenous Type of Dam

Homogeneous type of embankment dams is composed, essentially, of the same material throughout the embankment. The material used for construction of this type of dam must be sufficiently impervious to give an adequate water barrier. However, these dams are only for low to moderate height as they have very low slopes. Further, it is very rare that sufficient quantity of homogenous materials would be available within the economic distance from the proposed dam site.

6.2.2 Zoned Type of Dam

A zoned type earth dam is composed of more than one type of naturally available material. This is the most common type of a rolled fill dam in which zones of materials of considerably more pervious material forms the outer shell and a relatively impervious material forms the central core. The pervious zones may consist of sand, gravel cobbles or rock materials, while the core consists of an impervious soil such as clay, silt or clay gravel mixture. This type of dam is selected when the impervious material is not available in the sufficient quantity near to the Dam site.

6.3 Rock fill Type of Dam

The rock fill dam consist of three basic elements; (i) a loose rock fill dump, which constitutes the bulk of the dam and resist the thrust of the reservoir, (ii) Impervious facing of the upstream slope with concrete, timber, steel and (iii) Rubble masonry between (i) and (ii) to act as a cushion for the membrane and resist destructive deflections.

6.4 Selection of Type of Dam

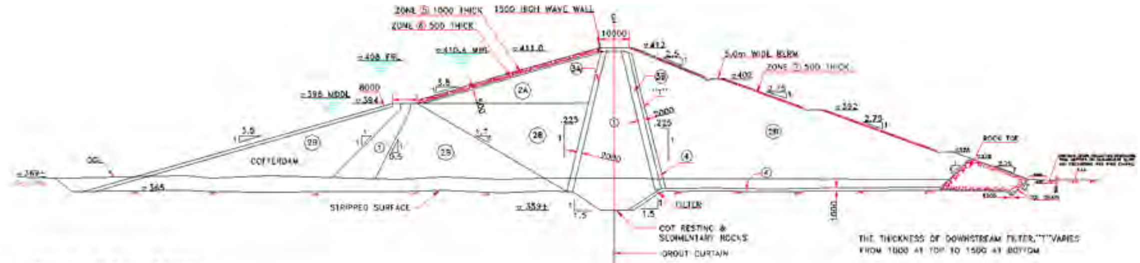
Based on the previous and the recent studies conducted by the Project Authority, construction of a Zoned type of earth fill dam has been considered for the Tendaho Dam site. This selection of dam is mainly based on two consideration, first availability of suitable construction material in adequate quantity within the proximity of the dam site and second, a need for a flexible structure which may withstand the earthquake loading, as the site falls within seismic prone area.

The height of the proposed dam is limited to a crest elevation of 412.0 m. The height of the dam is 42.5m above riverbed and 53.0 m above the deepest foundation level. A 1.5 m parapet wall is provided along the upstream edge of the crest. The dam embankment crest is 10 meters wide and 412.1 m long; the upstream and downstream slopes of the embankment are taken as 3.5: 1 and 3:1, respectively. The upstream cofferdam with a crest elevation of 394.0 will form an integral part of the main dam (Main Report on Tendaho Dam, TDIP, 2005).

The zoning of dam section is provided by centrally located clay core having 5 m width at top and 10m width at the bottom with zones of filter material both on upstream and downstream of core followed by outer shells of sandy gravel materials. Dumped rock riprap of 1 m thick is proposed for upstream slope protection and hand place rock riprap of 50 cm thick will be used for the protection of downstream slope. A section of Dam and zoning is shown in (Fig. 6.1).

6.5 Construction Material

The primary purpose of a dam project is to impound water across the valley. For the safe functioning of any dam project it is necessary that the dam is impervious enough to hold back the stored water and it must be stable to withstand the horizontal thrust exerted by the impounded water. Thus, selection of an appropriate construction material for various zones of an embankment is an important consideration for the safe functioning of a dam project. The availability of pervious and impervious construction material in sufficient quantities and appropriate quality within the economic distance from the dam site will be a deceive factor for the selection of a dam type. Particularly for embankment dam, techno-economic feasibility of the project will mainly depend upon the quality and quantity of the available construction materials.



ZONING OF DAM SECTION

ZONE	DESIGNATION	MATERIAL DESCRIPITION
1	IMPERVIOUS CORE	BLENDED SOIL CLAY & SANDY GRAVEL BY EQVAL WEIGHTS
2A	UPSTREAM SHELL	SANDY GRAVEL SILT < 6%
2B	DOWNSTREAM SHELL	SANDY GRAVEL
3A	TRANSITION ZONE	FINE SAND – FILER CRITERIA NOT REQUIRED
3B	TRANSITION FILTER	FINE SAND – FILER CRITERIA REQUIRED

4	COARSE FILTER	COARSE SAND, PEBBLE & GRAVEL UP TO 75 SIZE AND SILT < 5%
5	DUMPED ROCK RIPRAP	MAX. SIZE 1000 MM 40–50% > 800 MM 50–60% > 300–800 MM 0–10% > 300 MM SAND & ROCK DUST < 5%
6	BLANKET BELOW U/S RIPRAP	CRUSHED ROCK OR NATURAL GRAVEL, GRADED FROM 4.75–80
7	HAND PLACED ROCK RIPRAP	FLAT OR STRATIFIED ROCK FRAGMENTS

NOTES :-

1–Elevations are in meters and all dimension in millimetres unless specified otherwise

2–Topography of valley does not give rise to maximum section, as shown on the drawing. The Section is meant to illustrate Scheme of zoning and to be used for Stability analyses. For section close to maximum section, refer drawing No. 05-02-07

3–The stripped level below shell zone is tentative. The excavation depth will be so limited that insitu unit weight of soil matches with target compacted unit weight in this zone.

4–For details near dam crest, wave wall, and Rock toe, refer to drawing No. 05-02-08

5–Upstream face of the cofferdam is provided with 800 thick layer of rock fragments available as waste material at rock quarry/crushing plant.

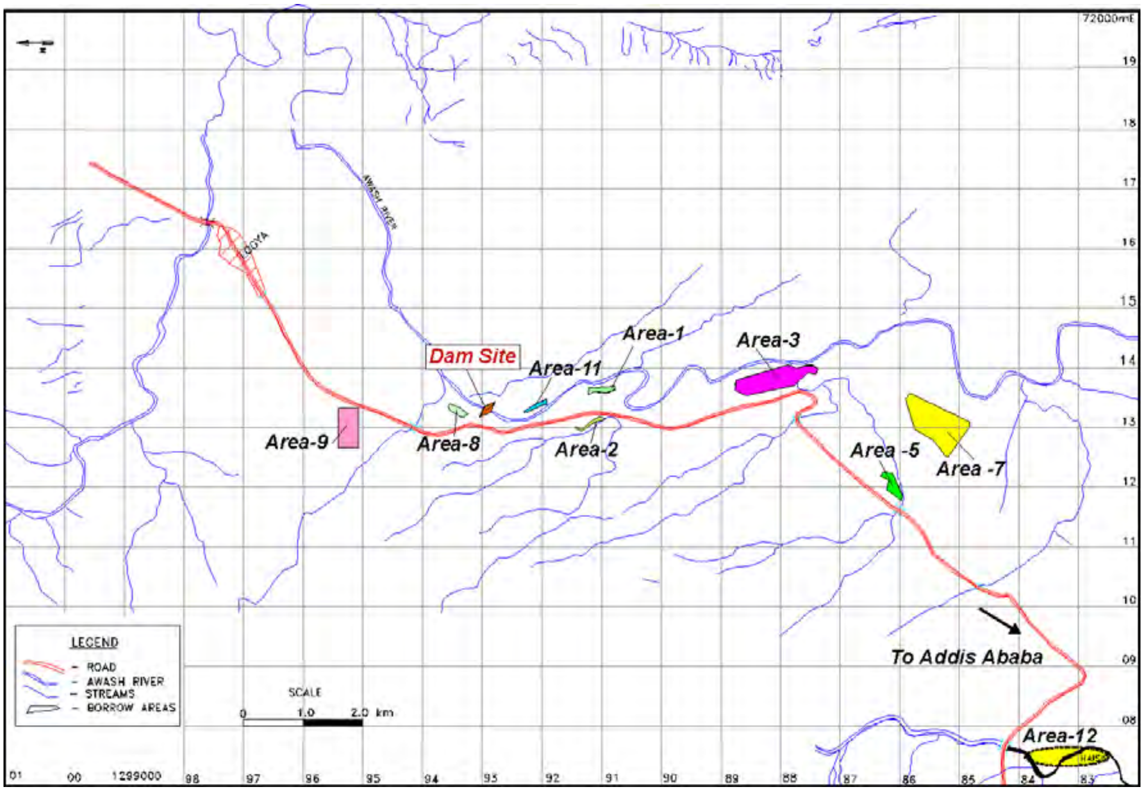
Source: Tendaho Dam and Irrigation Project Authority

Fig. 6.1 A Cross section of Proposed Tendaho Damshowing composition of various zones.

6.5.1 Selection of the Potential Construction Material Sites

Based on the previous studies and the present studies conducted by the Tendaho Dam Project Authorities, initially, total 16 potential sites for various types of construction material required for Tendaho Dam has been identified. Out of these, 12 sites were investigated and representative samples were collected by the Project Authorities for laboratory testing, to establish the quality standards of these materials (Fig.6.2). The construction material available at these sites involves material for central core, shells, coarse filter, fine filter and riprap. From these identified sites, three sites may provide material for clay core and six sites may provide sufficient quantity of material for shell material. Besides, one site may provide sufficient material for coarse and fine filter. Further, four sites have been identified for the riprap material. Table 6.1 presents location details and other relevant information for identified potential material source sites.

As a first priority, Project Authorities have identified Area-5 as a potential construction material site for the clay core. Area-1, Area-3, Area-7 and Area-11 has been identified as potential source for the construction material for shell area (Table 6.1 and Fig. 6.2). In addition, Area-2 and Area-8 are also potential sources in case to supplement the required quantities for the shell. Area-7 may provide suitable quantities of material for the shell and Area-12 for fine and coarse filter materials.



(Source: Tendaho Dam and Irrigation Project Authority)

Fig. 6.2 Location of potential construction material sites.

6.5.2 Construction Material for Clay Core

The first essential requirement for the core material is the availability of suitable material at an economic distance from the dam site. The borrow areas for the impervious zone have to be located near the dam site. Later it has to be investigated for available quantity and relevant soil properties. The important soil properties to be considered are; permeability, compacted density, shear strength, compressibility, flexibility and erosion resistance.

As per the proposed design of the Dam the core will be centrally located having 5 m width at the top and 10m width at the bottom with zones of filter material both on upstream and downstream of core. The construction material for the clay core will be available from borrow Area 2, Area 5 and Area 9. As per the Project Authorities clay material available at Area 5 is more suitable and may be sufficient to cover the entire volume requirements of the core.

During the present study three soil samples were collected from the borrow Area 5 for the determination of the suitability for the clay core. Two samples (TP-1 and TP-2) were collected from the Northwestern part of the Area 5 at a depth of 0.6 – 1.0m and 1.5 – 2.0 m respectively. Whereas, the third sample (B-1) for blending has also been collected from Area 5.

Table 6.1 Location details and other information for identified potential material source sites.

Area	Location Coordinates	Aerial coverage (x1000 m^2)	Distance from Dam site (Km)	Description of the Available Material
Area-1 Shell Material	1291000 N, 713500 E	20 - 30	0.6 - 1.2	The central part of the area is mainly covered by fine yellowish sandy silt. In the southern part of the area river gravels are available. In addition pockets of uniform sand and weathered rock in the form of scree material over the road slopes are available. Area identified suitable to provide sandy gravel and gravelly sand material for the 'Shell'.
Area-2 Shell and Clay core Material	1291000 N, 712900 E	20 - 25	1.7 - 2.3	The surface area is mainly covered by fine sandy silt. The dry stream bed has a shallow depth of fine silty sand. The hills surrounding the area have have soil gravelly sand and rock cover on the western side and weathered rock in the form of scree on the slope of the main highway on the Eastern side. Area identified suitable to provide sandy gravel and gravelly sand material for the 'Shell' and Clay core material. The depth range of the material is 0.75 to 1.3 m and average overburden depth is 1.0 m.
Area-3 Shell Material	1288000 N, 713800 E	40	4.5	The area mainly consists of fine sands, sandy gravel and different sizes of pure sand all located in the small hillocks. In the eastern side several layers of fine to coarse sand, sandy gravel and gravelly sand layers are present. In the Northern part of the area mainly sand with very little gravels are present. Area identified suitable to provide sandy gravel and gravelly sand material for the 'Shell'.
Area-4 Shell Material	1287000 N, 711700 E	20 -30	6.8 - 7.5	The area is mainly covered with yellowish fine grained silt. Several pockets of uniform wind blown sand are also located at the foot of the hills surrounding the area. Test pits indicates yellowish fine grained silty material upto a depth of 1.5m. In the southern area test pit indicates fine to medium grained gravelly sand underlying 1.7 m depth of yellowish clayey silt. Area identified suitable to provide fine to medium sand. This site may be used as second priority to supplement the fine and medium sand for shell, as the winning of sands from this area require considerable effort and systematic removal of overburden and exploitation of the sand.
Area-5 Clay core Material	1286000 N, 712000 E	60 - 80	7.5 - 8.3	The area is bounded by 25 – 30 m wide dry river bed on the Southern and Eastern end. The test pits in the area indicated the presence of different layers of clayeysoil and sandy gravelly material upto 1.5 m from the ground overlying a clay soil. The dry stream banks also show a thick layer of clay material. Area selected for Clay core and will be sufficient for the volume requirements of the proposed dam core. The depth range is 0.65 to 1.7 m with average overburden depth of 1.25 m.
Area-6 Shell Material	1285500 N, 712000 E	40 - 50	8.5	The area being 200 m on the eastern side of the main highway bounded by a dry stream bed on the Northern side and a basaltic hill on the southern and Western side. The test pits indicates predominantly silt and sand upto a depth of 2.8 m overlying a clayey soil. Area identified is suitable to provide fine to medium sand. This site may be used as second priority to supplement the fine and medium sand for shell, as the winning of sands from this area require considerable effort and systematic removal of overburden and exploitation of the sand.
Area-7 Shell Material	1285000 N, 713000 E	Very vast	8.5	The area consists of several hills and a dry streambed and a plane area. The hills consists of silty material with a cover of sandy gravel material. The hill side indicates a uniform sand while in the dry stream bed section a thin layer of sand overlying a thick layer of clay is present. Area identified suitable to provide sandy gravel and gravelly sand material for the 'Shell'.
Area-8 Shell Material	1293500 N, 713200 E	25 - 35	1.0	The area is a flat land bounded by Awash river on the Eastern side, the main highway on the western side, a dry stream bed on the Northern side and a small hill on the Southern side. Test pits on Eastern side near Awash river indicate presence of clay soil below 1.2 m from the ground underlying thin layers of silty sand and gravelly sand. Area identified suitable to provide sandy gravel and gravelly sand material for the 'Shell'.
Area-9 Clay core Material	1295000 N, 713000 E	15 - 20	2.6	The test pits indicated a highly plastic brown clay underlying layers of gravelly sand and silty clay below 1.2 m. Material suitable for Clay core, the depth range is 0.2 to 1.2 m and average overburden depth is 0.7 m.
Area-10 Not specified for use	1299000 N, 716000 E	15	7.5-8.5	The area is a flat land with few vegetation cover adjacent to Logia river and the dry river bed. The test pits on the Eastern and Western side of the area indicate stiff moderately to highly plastic clayey silt material starting from 0.5 m depth from the surface. Currently not specified for use.
Area-11 Shell Material	1292000 N, 713500 E	25	0.2	The area consists of series of small hillocks 4 – 6 m height. The test pits indicates a mixture of silt, sand, river gravel and few pebbles sites. Area identified suitable to provide sandy gravel and gravelly sand material for the 'Shell'.
Area-12 Fine and coarse filter and concrete works	1283500 N, 707000 E	150	17	The area comprises of very vast with 35 m wide dry dry river bed . The material available is fine sand, medium and coarse sand, different sizes of gravels and pebbles. Suitable for fine and coarse filter and concrete works.

(Source: Tendaho Dam and Irrigation Design Project, Report on Construction Material, June 2005)

Quality of the Core Material

The core is the impervious barrier constructed within the main dam body. Its primary purpose is to check the seepage of water through the dam. This function will only be performed effectively if the construction material, to be used for the core, has desirable engineering properties. Therefore, it becomes essential to know the properties such as, permeability, compacted density, shear strength, compressibility, flexibility and erosion resistance. Besides, the index properties, such as grain size distribution and Atterberg's limits are also equally important as a fair idea of engineering performance of the material can be known by classifying the soils.

Quality of the Core Material as per the test Results of Project Authorities

In order to determine the quality of the clay core construction material available at borrow Area 5 Project Authorities have conducted laboratory tests on the samples collected from 6 test pits. The tests include classification tests; grain size distribution and Atterberg's limits, free swell and linear shrinkage. Besides, tests were also performed to determine the engineering properties; viz, permeability, consolidation and shear strength properties of the representative soil samples. The results thus obtained are presented in Table 6.2.

The classification tests conducted by the Project Authorities on 6 soil samples collected from the borrow Area 5 suggests that percentage of fines varies in the range of 66.4 to 96.4. Also the liquid limit varies in the range of 27 to 98. Free swell varies from no swelling potential upto 125. Accordingly the soil samples from borrow Area 5 has been classified as CL, CH and MH as per the Unified Soil Classification (USC) system. According to Indian Standard Code (10) (12169-1987) 'CL' soils (Inorganic clays of low to medium plasticity) are classified as 'suitable soils' for impervious core and 'CH' (Inorganic clays of high plasticity) are classified as 'fairly suitable soils' for impervious core. However, 'MH' (Inorganic silts or high plasticity) soils are considered as 'poor material' for the impervious core (Bharat Singh and Varshney, 1995).

Engineering Properties of the Core Material

A perusal of Table 6.2 indicates that the 'Maximum dry density' (MDD) varies in the range of 1.38 – 1.82 t/m^2 which is within the permissible limits of 1.35 – 1.5 t/m^2, as suggested by USBR for CH type of soils. Also, 'Optimum Moisture Content'(OMC) varies from 12 to 29% which is within the permissible limits of 2.5 – 1.2%, as suggested by USBR. However, the permeability values as obtained from the laboratory tests show higher values when compared with the permissible limit of 10^{-6} cm/sec for highly plastic type of soils (CH), as suggested by USBR. The shear strength parameter values varies in the range, Cohesion 'C' (0.06 – 0.11 kg/cm^2) and angle of shearing resistance 'ϕ' (18^0 –31^0). According to USBR 'C' value is on lower side as the range for cohesion is (0.7 – 1.38 Kg/cm^2), whereas, the ‘ϕ’ value is on the higher side of the permissible range of (14.15^0 – 24.43^0).

Table 6.2 Index and Engineering Properties of the Clay Core Material from Borrow Area-5

		Tests conducted by Project Authorities						Test conducted during Present Study		
Sample No.		**A5 TP-1**	**A5 TP-2**	**A5 TP-3**	**A5 TP-4**	**A5 TP-5**	**A5 TP-6**	**TP-1**	**TP-2**	**B-1**
Depth (m)		1.75-1.95	2.00-2.15	1.80-2.00	2.10-2.40	2.1-2.4	1.5-2.1	0.6-1.0	1.5-2.0	0.5
Properties										
Specific Gravity		2.75	2.76	2.77	2.45	2.74	2.73	2.7	2.7	2.7
Grain Size Distribution	*Gravel*	-	-	-	-	-	-	-	-	24.67
	Sands	33.6	27.2	14.4	4	9	3.6	13.06	17.58	26.73
	Fines	66.4	72.8	85.6	96	91	96.4	86.94	82.42	48.60
Atterberg’s Limits	*WL*	49	53	56	27	98	71	39.8	45.6	40.2
	PI	21	30	30	10	59	39	23.7	21.1	17.4
Classification (USC)		CL	CH	CH	CL	MH	CH	CL	CL	CL
Free Swell		55	115	100	-	125	110	-	-	65
Linear Shrinkage		7.1	-	20.6	-	-	-	-	12.97	11.18
Compaction	*MDD*	-	1.7	1.82	1.43	1.38	1.43	1.62	-	1.37
	OMC	-	19.3	12.4	25.25	33	28.5	23	-	30.20
Permeability (cm/sec)		-	3.7×10^{-9}	-	-	3.9×10^{-9}	1.44×10^{-8}	-	-	Impervious
Consolidation	m_v	-	-	-	-	-	-	-	-	-
	cc	-	-	-	-	0.41	-	-	-	-
Shear Strength	*C(kg/cm^2)*	-	0.11	0.06	0.064	-	-	-	-	-
	ϕ	-	18	31	31	-	-	-	-	-
Remarks (as per the present study)		Suitable for core*	Fairly suitable soils for core*	Fairly suitable soils for core*	Suitable for core*	Poor core material*	Fairly suitable soils for core*	Suitable for core*	Suitable for core*	Suitable for core*

***** *According to Indian Standard Code (10) (12169-1987)-based on Unified Soil Classification System.*
WL – Liquid Limit, PI – Plasticity Index, MDD – Maximum dry density, OMC – Optimum moisture content, m_v *- coefficient of volume change, cc – Compression index, C – cohesion and* ϕ *- angle of shearing resistance, USC – Unified Soil Classification*

Quality of the Core Material as per the present study

During the present study 3 soil samples (TP-1, TP-2 and B-1) were collected from borrow Area 5 and the laboratory tests were conducted to determine the index and engineering properties of the soils (Table 6.2). The tests for the present study were performed at Tendaho Dam and Irrigation project laboratory, at Engineering geology laboratory, Addis Ababa University and in the Minster of Water Resources, Addis Ababa.

Determination of Index Properties

Data obtained from index test together with description of visual observation are often sufficient for design purposes for minor structure. This information is used also in making preliminary designs for determining probable cost of a major structure and to limit the amount of detailed testing (Earth Manual).

Grain-size Analysis

A combined grain size analysis and hydrometric test were performed on two samples collected from the borrow Area 5. The particle size distribution curves thus, obtained are presented in Fig. 6.3.

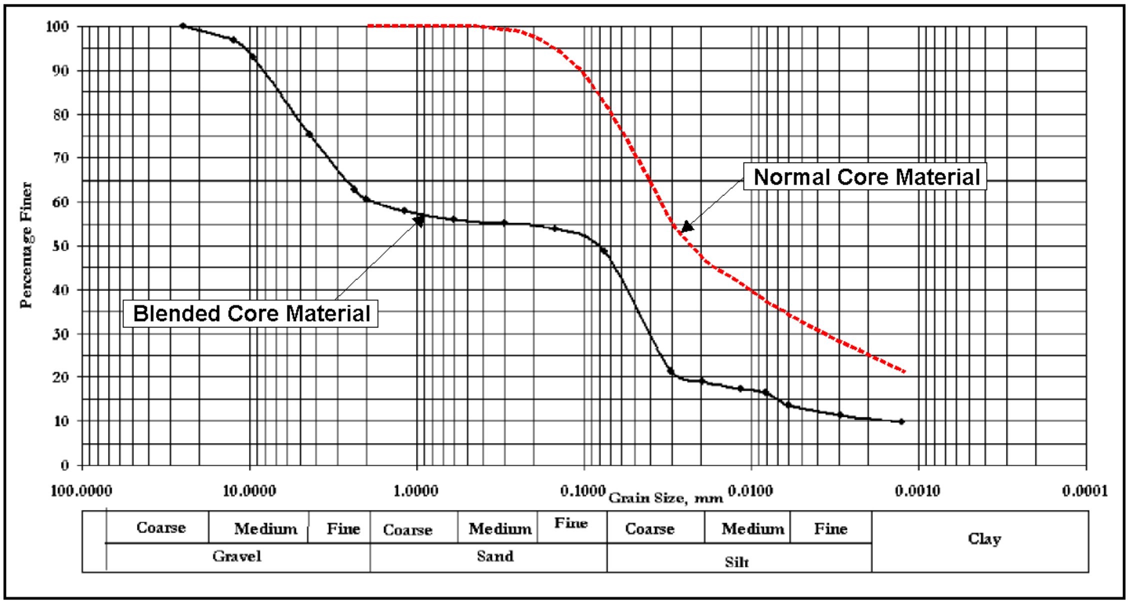


Fig. 6.3 Gradation curve for core material

Consistency Limits

The consistency of a fine grained soil is the physical state in which it exists. It is used to denote the degree of firmness of a soil. Consistency of a soil is indicated by such terms as soft, firm or hard. In 1911, a Swedish agriculture engineer Atterberg mentioned that a fine grained soil can exist in four states; namely, liquid, plastic, semi-solid or solid-state. The water contents at which the soil changes from one state to the other are known as consistency limits or Atterberg limits. The Atterberge limit such as liquid and plastic limit can be determined numerically in the laboratory. Based on the consistency the soils can be classified into various engineering classes and thus, a fair idea can be obtained for it's engineering performance.

In order to determine the consistency of the soils from the borrow Area 5, disturbed samples were collected and the tests were performed in the laboratory. The tests were performed to determine the liquid limit and the plastic limits of the clays of the borrow Area 5. Based on the liquid limit (W_L) and plastic limit (W_P), the plasticity index is determined as the difference of liquid limit and plastic limit;

Plasticity Index, $I_p = W_L - W_P$ …….6.1

The test results on consistency of soils from borrow Area 5 are presented in Table 6.3.

Based on the results a plasticity chart as per the Unified Soil Classification System has been prepared (Fig. 6.4) and which was later utilized to classify the soils. A perusal of Fig. 6.4 indicates that all the three samples collected from borrow Area 5 falls above the 'A line' in CL zone, which implies that the soils are clays of intermediate plasticity.

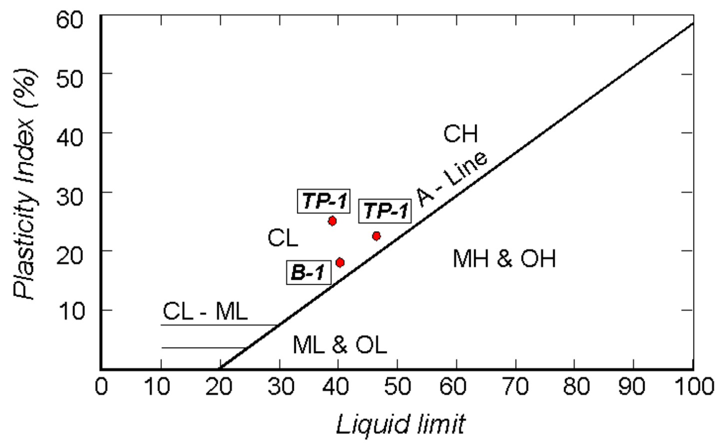


Fig 6.4 Activity Chart for Core Samples

Swelling Potential (Sp)

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 can also be indicated by the plasticity index and shrinkage limit (Ranjan, 1991).

Seed, Woodweard and Lundren (1962) has empirically related the swelling potential (Sp) with the plasticity index (I_P) by the equation;

$$Sp = 60\ K\ I_P^{2.44}$$ ……..6.2

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

Table 6.4 shows the swelling potential of clays of borrow Area 5.

Activity of the soils

The amount of water in the soil depends on the type of clay minerals. Determination of activity of soil is one of the most important index properties for fine-grind soils because it measures the water holding capacity of clay soils (Ranjan, 1991). Activity is defined as the ratio of plasticity index to the percentage of clay fraction smaller than two microns.

Table 6.3 Consistency of soils from borrow Area 5

Sample No	Depth (m)	Atterberg limits			Unified soil classification
		Liquid Limit (W_L)	Plastic Limit (W_P)	Plasticity Index (I_P)	
Tp-1 (Sandy silty clay)	0.6- 1	39.8	16.0	23.7	CL Clays of intermediate Plasticity
Tp-2 (silty clay)	1.5-2	45.6	24.5	21.1	CL Clays of intermediate Plasticity
B-1 (Blended)	0.5	40.2	22.7	17.4	CL Clays of intermediate Plasticity

Table 6.4 Swelling potential of clays of borrow Area 5

Sample No	Depth (m)	Plasticity Index (I_P)	Swelling Potential (Sp)	Expansivity
Tp-1 (Sandy silty clay)	0.6- 1	23.7	4.8	Medium
Tp-2 (silty clay)	1.5-2	21.1	3.67	Medium
B-1 (Blended)	0.5	17.4	2.29	Medium
Sp **Expansivity** **Sp** **Expansivity** < 1.5 Low 5 – 25 High 1.5 – 5 Medium > 25 Very High				

Activity (Ac) = Plasticity index (I_P)/ % finer than 2 micron …….6.3

Activity of the soils may be classified as;

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

For the borrow Area 5, Activity was thus, determined and is presented in Table 6.5.

Perusal of Table 6.6 indicates that the soil sample TP-2 is normal whereas after blending it will be active thus it may show swelling characteristic.

Table 6.5 Classification of Soils of the borrow Area 5

Soil type	Clay proportion	Plasticity Index (Ip)	Activity Number (Ac)	Activity classification
Tp-2 (silty clay)	25	21.1	0.84	Normal
B-1 (Blended)	11	17.4	1.58	Active

<u>Engineering Properties of the Core Material</u>

The primary purpose of the core material is to check the seepage of water through the dam. The important engineering properties, which influences the performance of the core material are permeability, compacted density, shear strength, compressibility, flexibility and erosion resistance. All these properties require elaborate laboratory tests. For the present study sincere efforts were made to evaluate the properties of the construction material based on the realistic field observations supported with experimental data. The tests required for the determination of engineering properties of the soils needs elaborate tests, which require sufficient time, adequate resource and financial support. Thus, all these factors made these elaborate tests beyond the reach of the present study. However, efforts are made to evaluate the quality of the core material by utilizing secondary data and the available empirical techniques.

Permeability

The property of a soil, which permits flow of water through it, is called the permeability. Determination of coefficient of permeability is important for water retention structures like dam because water lost through the soil may result into removal of soluble solids or may cause internal erosion called piping. For the present study coefficient of permeability on blended core sample has been determined in laboratory of Minster of Water Resources, Addis Ababa whereas, for unblended clay core samples empirical technique proposed by Allen Hazene have been used.

According to Allen Hazane's formula;

$$K = C * (D10)^{2} \qquad \text{.......6.4}$$

Where, 'K' is the coefficient of permeability (cm/sec), 'C' is constant usually taken to be 100 and 'D_{10}' is the effective grain size (Arora, 1997).

The results thus obtained are presented in Table 6.7.

As per the USBR permissible limits the permeability for CH class of soils must lie within the range of 0.05 $\pm$ 0.05 x 10 $^{-6}$ cm/sec, however the values are on the higher side as determined by the Project Authorities. Therefore, appropriate correction is required while construction the core section. During the present study the permeability was determined for the blended proposed core material from borrow Area-5. For the blending purpose a ratio of 35: 65 was considered for sandy-gravel and fatty clay, respectively. In order to perform the permeability test compaction of the blended sample was done upto a compacted dry density of 1.842 gm/cc at 17.3% optimum moisture content. Constant head permeability method has been adopted to perform this test. The setup was kept for two weeks and no water drop was noticed. Thus, the soil may be considered as impervious. However, this may be because of the limitation of the test method. As per the standard practice falling head method has to be adopted for the low permeability soils.

Table 6.6 Permeability of Core Material

	Tests conducted by Project Authorities						Test conducted during Present Study		
Sample No.	**A5 TP-1**	**A5 TP-2**	**A5 TP-3**	**A5 TP-4**	**A5 TP-5**	**A5 TP-6**	**TP-1**	**TP-2**	**B-1**
Depth (m)	1.75-1.95	2.00-2.15	1.80-2.00	2.10-2.40	2.1-2.4	1.5-2.1	0.6-1.0	1.5-2.0	0.5
Properties									
D10 effective size	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0012
Permeability (cm/sec)	-	$3.7x10^{-9}$	-	-	$3.9x10^{-9}$	$1.44x10^{-8}$	-	-	Impervious*
Classification (USC)	CL	CH	CH	CL	MH	CH	CL	CL	CL
Suitability as core material	-	Fairly suitable **	-	-	Poor core material	Fairly suitable **	Suita-ble ***	Suita-ble ***	Suita-ble *

* *Under constant head method for 2-weeks their was no water drop, thus the soil may be considered as 'impervious'. However, this may be due to non availability of "falling head apparatus' the test was performed by 'constant head apparatus 'this may be the reason for the zero permeability of the material.*

** *The value within the permissible limits(0.05 $\pm$ 0.05x10^{-6}cm/sec) for 'CH'soils as per USBR., however the tested values are on higher sidewhich could be due to the presence of the fine fractions or may be due to possible testing errors.*

*** *As per classification only - According to Indian Standard Code (10) (12169-1987)-based on Unified Soil Classification System.*

Shear strength Properties

Shear strength of a soil is its maximum resistance to shear stresses just before the failure. It is the principal engineering property, which controls the stability of a soil mass under load. During the present study because of the limitation on time and resources it was beyond the reach of the present study to conduct the elaborate shear strength test on the soils of the core material. However, an attempt has been made to evaluate the shear strength property of the core material based on the classification tests and the secondary data from the Project Authorities. The test results as per the Project Authorities and estimations made during the present study are presented in Table 6.7.

The shear strength parameter values variy in the range, Cohesion 'C' (0.06 – 0.11 kg/cm^2) and angle of shearing resistance 'ϕ' (18^0 –31^0). According to USBR 'C' value is on lower side as the range for cohesion is (0.7 – 1.38 Kg/cm^2), whereas, the 'ϕ' value is on the higher side of the permissible range of (14.15^0 – 24.43^0).

Compaction

Compaction is the change of volume of soil produced by mechanical manipulation such as rolling, tamping or vibrating. The volume change is associated with change in volume of void and with limited change in volume of solid particles.

Table 6.7 Shear strength of the Clay Core Material from Borrow Area-5

		Tests conducted by Project Authorities						Test conducted during Present Study		
Sample No.		**A5 TP-1**	**A5 TP-2**	**A5 TP-3**	**A5 TP-4**	**A5 TP-5**	**A5 TP-6**	**TP-1**	**TP-2**	**B-1**
Depth (m)		1.75-1.95	2.00-2.15	1.80-2.00	2.10-2.40	2.1-2.4	1.5-2.1	0.6-1.0	1.5-2.0	0.5
Properties										
Classification (USC)		CL	CH	CH	CL	MH	CH	CL	CL	CL
Shear Strength	*C (kg/cm^2)*	-	0.11	0.06	0.064	-	-	-	-	-
	ϕ (^{o})	-	18	31	31	-	-	-	-	-
Remarks		Suitable for core*	Fairly suitable soils for core**	Fairly suitable soils for core**	Suitable for core**	Poor core material*	Fairly suitable soils for core*	Suita-ble for core*	Suita-ble for core*	Suita-ble for core*

***** *According to Indian Standard Code (10) (12169-1987)-based on Unified Soil Classification System.*
C – cohesion and ϕ - angle of shearing resistance, USC – Unified Soil Classification

Compaction increase the strength of the soil makes the soil partials together and reduce permeability by rearranging the grains and closing the voids. Therefore, compaction is an important engineering property for the core material.

During the present study three samples were collected from the clay core Area-5. The test was conducted at Tendaho Dam site laboratory, retested in Addis Ababa University and Ministry of Water Resources Laboratory, Addis Ababa by using standard Procter test and modified proctor test as per the procedures of ASTM standards. The results thus obtained are presented in Table 6.8 and Fig. 6.5.

Core material compacted at moisture content wet of optimum will have lower permeability, high flexibility or capacity to deform without cracking and lesser compressibility on saturation. However, USBR practice is to place the fill at 1 to 3% below optimum. As per US Army Corps Practice Compaction is done at or above optimum moisture content.

According to Middlebrooks the core material should satisfy the following criteria;

i) It must be placed at a density or a mois -ture, which will not allow further con -solidation on saturation.

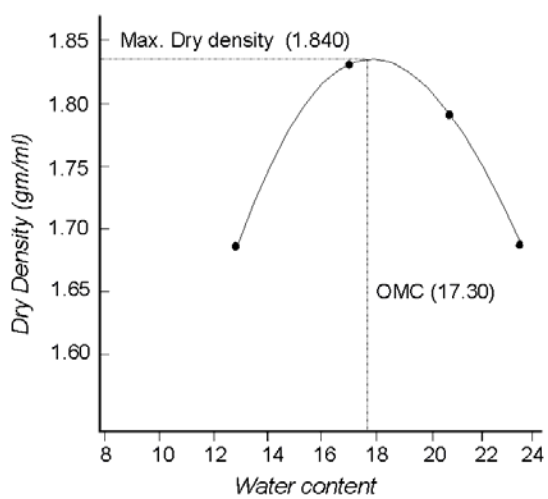


Fig. 6.5 Compaction curve for blended core material

ii) It must be sufficiently plastic so that differential settlement will not cause cracks to develop through it.

Table 6.8 Compaction Test Results of the Clay Core Material from Borrow Area-5

		Tests conducted by Project Authorities						Test conducted during Present Study		
Sample No.		A5 TP-1	A5 TP-2	A5 TP-3	A5 TP-4	A5 TP-5	A5 TP-6	TP-1	TP-2	B-1
Depth (m)		1.75-1.95	2.00-2.15	1.80-2.00	2.10-2.40	2.1-2.4	1.5-2.1	0.6-1.0	1.5-2.0	0.5
Properties										
Classification (USC)		CL	CH	CH	CL	MH	CH	CL	CL	CL
Compaction	*MDD*	-	1.7	1.82	1.43	1.38	1.43	1.62	-	1.37
	OMC	-	19.3	12.4	25.25	33	28.5	23	-	30.20

MDD – Maximum dry density, OMC – Optimum moisture content, USC – Unified Soil Classification

The USBR dams have been built with compaction moisture content for impervious zone ranging from 0.7 to 2.5% dry of optimum (Bharat Singh and Varshney, 1995).

The Project Authorities have classified the Core material as ‘CL’ soils (Inorganic clays of low to medium plasticity) and ‘CH’ (Inorganic clays of high plasticity), where as in the present study the soils of the core material have been classified as ‘CL’ (Inorganic clays of intermediate plasticity). Thus the core material, as per the Middlebrooks criteria, is suitable for core material and differential settlement may not cause cracks to develop through it.

Flexibility and Erosion Resistance

Two desirable properties to be looked for core material are the flexibility and erosion resistance. Flexibility means ability to deform without cracking. Non cohesive granular material cannot retain open cracks but such material are very pervious therefore they cannot be used in core. Flexibility increases with an increase in Plasticity Index (PI). However, very high values of PI may be associated with high compressibility. Erosive resistance is the ability of soil to withstand the erosive action of water leaking through possible cracks. Erosive resistance is mainly derived from two sources cohesion of the fines and the resistive action of coarse particles to the flowing water and their tendency to wedge up in the leakage channel. This effect is best obtained in a well graded sand gravel mixture with enough finer particles to provide imperviousness. In such a material the coarser particle within the crack obstruct the flow and prevent development of high velocities. In tough plastic clay the resistance to erosion is provided by the strong interparticle adhesion. A filter plays a very important role in safeguarding against failure by leakage. The eroded particles of the core flowing within the crack cannot pass through to the downstream filter, which gradually seal up the crack (Bharat Singh and Varshney, 1995).

For the present study an attempt has been made to estimate the erosion resistance based on the plasticity index of the material. As per Sherard, 1967 highly plastic tough clay (CL) with plasticity index greater than 20 are considered 'good material' as far as erosion resistance of core material is concerned. The plasticity index for the core samples tested by the Project Authorities have a plasticity index (PI) greater than 20, thus the core material may be considered as good material to withstand the erosive action of water leaking through possible cracks.

The tests conducted during the present study indicates plasticity index (PI) values greater than 20 for TP-1 and TP-2 samples which shows that core material is erosion resistant. However, for the blended sample the PI value is 17, which does not satisfies the Sherard's criteria. According to Bharat Singh, 1995 the erosion resistance for blended core material may be improved by addition of stone chips to the extent of 10 - 20%, provided the stone chips are locally available. However, according to Indian Standard Code (10) (12169-1987) 'CL' soils (Inorganic clays of intermediate plasticity) are classified as 'suitable soils' for impervious core (Bharat Singh and Varshney, 1995).

The available core material at borrow Area-5 does not contain sufficient quantity of coarse material, as can be noticed from the gradation of the material (Fig. 6.3). The fine fraction varies in the range of 66.4% to 96.4% with remaining constituent of sand fraction (4 – 33.6 %) only. As such no gravel fraction is noticed in any of the samples tested by the Project Authorities. Even under the present study the two samples collected from borrow Area-5 does not show any gravel fraction with a very small quantity of sands (13.06 –17.5 %), the major constituent of the sample is clay material (82.4 – 89.64 %). Thus, in the absence of coarse fraction the core material may be less resistant to the erosion and chances of piping will be increased. Therefore, there is a necessity to blend the material with appropriate proportion of coarse fraction. The coarse material for the core is readily available at borrow Area-5 which may be blended with the clay at the borrow area itself. The blending proportion for coarse material may be kept in a range of 30-40%. However, to work out the exact proportion of the coarse material further testing of the blended material would be required. During the present study the filter criteria for the core were applied on a blended sample by taking a coarse fraction of 35%.

6.5.3 Construction Material for Shell

The shell is the bulk mass of the embankment dam, which is primarily required to provide the stability to the dam and to withstand the horizontal thrust exerted by the impounded water behind the dam. The first essential requirement for the shell material is the availability of suitable material at an economic distance from the dam site. The borrow areas for the shell have to be located near the dam site. Later it has to be investigated for available quantity and relevant soil properties.

As per the requirement for shell section of the dam it is recommended to use sound and resistant to erosion rock fragments, gravels and cobbles available around the dam site. According to the Project Authorities reports on construction material, borrow Area-1, Area-3, Area-7 and Area-11 has been identified as potential source for the construction material for shell area (Table 6.1 and Fig. 6.2). In addition, Area-2 and Area-8 are also potential sources in case to supplement the required quantities for the shell.

Quality of the Shell Material

The shell is the bulk of the embankment which is constructed on either side of the impervious core. Its primary purpose is to provide the stability to the main dam by virtue of its weight and to withstand the thrust of the impounded water. This function will only be performed effectively if the construction material, to be used for the shell, has desirable engineering properties.

Quality of the Shell Material as per the test Results of Project Authorities

In order to determine the quality of the shell material available at borrow Area-1, Area-3, Area-7 and Area-11 Project Authorities have conducted laboratory tests on the samples collected from different test pits at all potential borrow areas. The tests include classification tests; grain size distribution and Atterberg's limits, free swell and linear shrinkage. Besides, tests were also performed to determine the engineering properties; viz, permeability, consolidation and shear strength properties of the representative soil samples. The results thus obtained are presented in Table 6.9 and Table 6.10.

The classification tests conducted by the Project Authorities on the soil samples collected from the borrow Area-1, Area-3, Area-7 and Area-11 suggests that most of the soils falls in SW, GW and SP, GP class of Unified Soil Classification system, according to Indian Standard Code (10) (12169-1987) these classes falls in 'very suitable' and 'fairly suitable' category, respectively. However, the soils from borrow Area-7 falls in SW class as per the Unified Soil Classification system which make the soils very suitable for pervious shell.

Engineering Properties of the Shell Material

A perusal of Table 6.11 indicates that the values of permeability for all the tested samples varies in the range of $10^{-4} – 10^{-2}$ cm/sec which are close to the permissible limits (1.5×10^{-5} to 3×10^{-3} cm/sec) as suggested by the USBR. The angle of internal friction for the tested samples varies in the range of 38^o to 48^o where as the USBR recommended angle of internal friction is 36.5^o. Also the value of cohesion (C) for the tested samples for shell material varies in the range of 0.02 to 0.31 kg/cm^2 whereas as per USBR value for cohesion is 0.22 kg/cm^2.

Table 6.9 Index Properties and Classification of the Shell Material as Determined by the Project Authorities.

Borrow Area	Location	Depth	Atterberg's Limit		Free Swell	Linear Shrinkage	Cu	Cc	Classi-fication USC	Remarks*** As per present study
			WL	PI						
Area-1	A1 TP-1	2.40-2.50	80	42	115	-	-	-	MH	-
	A1 GS-1	0.1-1.0	-	-	-	-	19.4	0.1	SP	Fairly Suitable
	A1 GS-1	1.4-2.0	-	-	-	-	72.7	13.9	GP	Fairly Suitable
	A1 GN-2	0.40-0.75	-	-	-	-	2.08	1.14	SP	Fairly Suitable
	A1 GN-2	0.75-1.10	-	-	-	-	1.80	1.10	SP	Fairly Suitable
	A1 TP-3	0.90-1.2	-	-	-	-	4.81	0.001	GP	Fairly Suitable
	A1 TP-4	1.10-1.35	-	-	-	-	12.30	0.03	SP	Fairly Suitable
	A1 TP-5	1.10-1.30	-	-	-	-	10.0	0.01	GP	Fairly Suitable
	A1 TP-6	0.60-1.10	-	-	-	-	45.7	2.70	GW	Very Suitable
Area-3	A3 TP-2	1.15-1.30	-	-	-	-	2.4	0.81	SP	Fairly Suitable
	A3 TP-3	0.30-0.65	-	-	-	-	2.90	0.17	SP	Fairly Suitable
	A3 TP-4	0.00-1.00	-	-	-	-	2.88	0.14	SP	Fairly Suitable
	A3 TP-5	0.00-0.65	-	-	-	-	5.43	0.06	SP	Fairly Suitable
	A3 TP-6	0.00-0.70	-	-	-	-	6.15	0.02	GP	Fairly Suitable
	A3 TP-7	0.80-1.20	-	-	-	-	2.28	1.14	SP	Fairly Suitable
	A3 TP-8	1.30-1.90	-	-	-	-	2.80	0.84	SP	Fairly Suitable
	A3 TP-11	0.80-1.40	-	-	-	-	15.6	0.52	SP	Fairly Suitable
Area-7	A7 TP-1	2.50-2.65	107	70	175	-	5.0	1.8	CH	-
	A7 TP-2A	Side slope	-	-	-	-	9.80	1.27	SW	Very Suitable
	A7 TP-2	1.60-2.30	-	-	-	-	13.25	1.28	SW	Very Suitable
	A7 TP-3	0.80-1.20	-	-	-	-	10.33	1.30	SW	Very Suitable
	A7 TP-4	2.0-2.50	-	-	-	-	9.55	1.75	SW	Very Suitable
	A7 TP-5	0.50-1.0	-	-	-	-	65.0	0.98	SW	Very Suitable
	A7 TP-6	0.20-0.50	-	-	-	-	13.33	3.44	GP	Fairly Suitable
	A7 TP-7	0.80-1.45	-	-	-	-	11.23	1.33	GW	Very Suitable
Area-11	A11 TP-1	0.1-1.2	-	-	-	-	7.0	0.57	SP	Fairly Suitable
	A11 TP-2	0.6-1.1	-	-	-	-	73.6	0.09	SP	Fairly Suitable
	A11 TP-3	0.3-0.9	-	-	-	-	3.94	0.01	GP	Fairly Suitable
	A11 TP-4	0.5-1.05	-	-	-	-	3.93	0.03	SP	Fairly Suitable
	A11 TP-5	1.0-1.30	-	-	-	-	2.70	0.01	GP	Fairly Suitable
	A11 TP-6	0.0-0.60	-	-	-	-	5.77	0.02	GP	Fairly Suitable

*** Suitability of Pervious core material as per IS code (10) (12169-1987) – (Bharat Singh and Varshney, 1995)

The tested values for permeability and shear strength parameters show variation with the permissible values of USBR. However, this variation may be because the tests were conducted only on the sand size particles (plus 4.75 mm).

Table 6.10 Engineering Properties of the Shell Material as Determined by the Project Authorities.

Borrow Area	Location	Depth	Compaction		Permeability (cm/sec)	Shear Strength Parameters	
			MDD	OMC		Cohesion 'C' kg/cm^2	'ϕ' (^{o})
Area-1	A1 TP-1	2.40-2.50	-	-	-	-	-
	A1 GS-1	0.1-1.0	-	-	-	-	-
	A1 GS-1	1.4-2.0	-	-	5.21×10^{-3}	-	-
	A1 GN-2	0.40-0.75	-	-	-	-	-
	A1 GN-2	0.75-1.10	-	-	-	-	-
	A1 TP-3	0.90-1.2	-	-	3.16×10^{-3}	0.04	44
	A1 TP-4	1.10-1.35	-	-	2.30×10^{-3}	0.12	41
	A1 TP-5	1.10-1.30	-	-	4.60×10^{-3}	0.04	44
	A1 TP-6	0.60-1.10	-	-	-	-	-
Area-3	A3 TP-2	1.15-1.30	-	-	-	0.19	38
	A3 TP-3	0.30-0.65	-	-	2.30×10^{-2}	0.05	46
	A3 TP-4	0.00-1.00	-	-	9.50×10^{-3}	0.14	45
	A3 TP-5	0.00-0.65	-	-	1.50×10^{-3}	0.08	44
	A3 TP-6	0.00-0.70	-	-	2.80×10^{-3}	0.02	48
	A3 TP-7	0.80-1.20	-	-	-	-	-
	A3 TP-8	1.30-1.90	-	-	-	-	-
	A3 TP-11	0.80-1.40	-	-	-	-	-
Area-7	A7 TP-1	2.50-2.65	-	-	-	-	-
	A7 TP-2A	Side slope	-	-	-	-	-
	A7 TP-2	1.60-2.30	-	-	-	-	-
	A7 TP-3	0.80-1.20	-	-	-	-	-
	A7 TP-4	2.0-2.50	-	-	-	-	-
	A7 TP-5	0.50-1.0	-	-	-	-	-
	A7 TP-6	0.20-0.50	-	-	-	-	-
	A7 TP-7	0.80-1.45	-	-	-	-	-
Area-11	A11 TP-1	0.1-1.2	-	-	-	0.18	40
	A11 TP-2	0.6-1.1	-	-	-	0.31	40
	A11 TP-3	0.3-0.9	-	-	5.92×10^{-4}	0.04	43
	A11 TP-4	0.5-1.05	-	-	2.84×10^{-3}	0.02	45
	A11 TP-5	1.0-1.30	-	-	5.54×10^{-3}	0.04	45
	A11 TP-6	0.0-0.60	-	-	4.09×10^{-3}	0.05	46

Quality of Shell Material as per the present study

During the present study 3 soil samples (TP-1, TP-2 and TP-3) were collected from borrow Area 7 and the laboratory tests were conducted to determine the index and engineering properties of the soils (Table 6.12). The tests for the present study were performed at Tendaho Dam and Irrigation project laboratory, at Engineering geology laboratory, Addis Ababa University and in the Minster of Water Restores, Addis Ababa.

Grain-size Analysis

A grain size analysis were performed on three samples collected from the borrow Area 7. The particle size distribution data and curves thus, obtained are presented in Table 6.11 and Fig. 6.6. Based on the gradation the soils from TP-1, Tp-2 and TP-3 are classified as SW (Cu 1.8, Cc 17.57), SP (Cu 50, Cc 0.6) and SP (Cu 65, Cc 0.8).

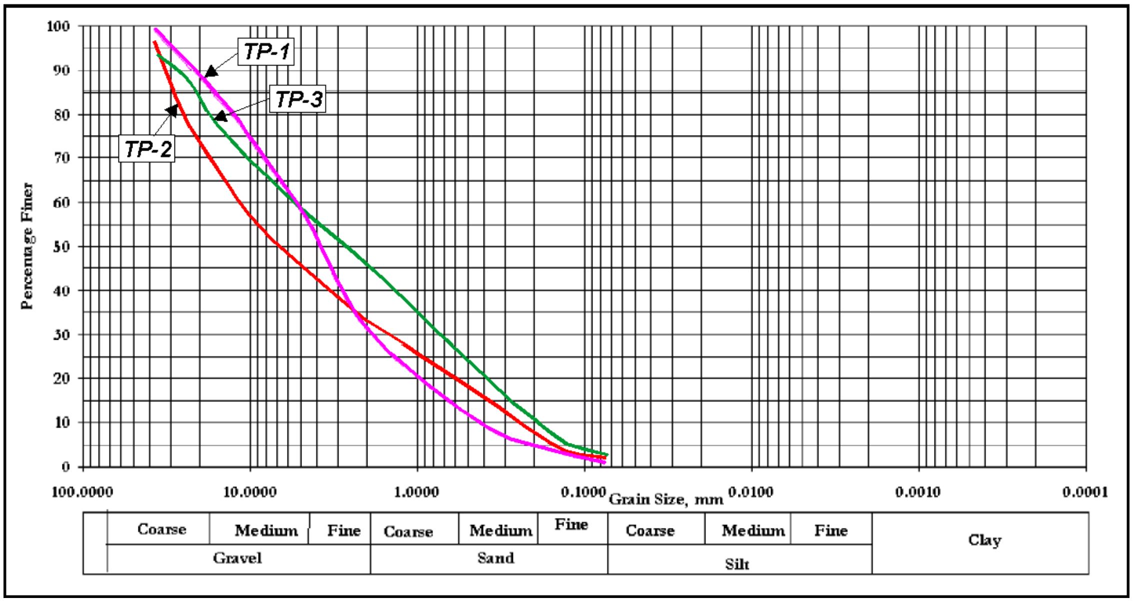


Fig.6.6 Gradation of shell material from borrow Area-7.

Table 6.11 The Particle size distribution and Classification of Shell Material as per the Present Study.

Borrow Area	Location	Depth	Cu	Cc	Classification USC	Remarks **
TP-1	Area -7	1.0 – 1.5 m	1.8	17.57	SW	Very suitable
TP-2	Area -7	0.1 – 1.6 m	50	0.6	SP	Fairly suitable
TP-2	Area - 7	0.0 – 1.5 m	65	0.8	SP	Fairly suitabl
** Suitability of the material according to Indian Standard Code (10) (12169-1987)						

The grain size distribution analysis conducted during the present study on the soil samples collected from the borrow Area-7 suggests that the soils falls in SW and SP class of Unified Soil Classification system, according to Indian Standard Code (10) (12169-1987) these classes falls in ‘very suitable’ and ‘fairly suitable’ category, respectively. Which is in close agreement of the findings made by the Project Authorities.

<u>Engineering Properties of the Shell Material</u>

The primary purpose of the shell material is to provide the stability to the dam. The important engineering properties, which influences the performance of the shell material are permeability, compacted density and shear strength.

Compaction Test

For the present study compaction test was performed by the Modified Proctor compaction method on a mixed soil sample from borrow Area-7. The computed test result from the density-moisture plot (Fig. 6.7) indicates a maximum dry density (MDD) of 1.62 gm/cm^3 can be achieved at an 'Optimum Moisture Content' of 20.0%.

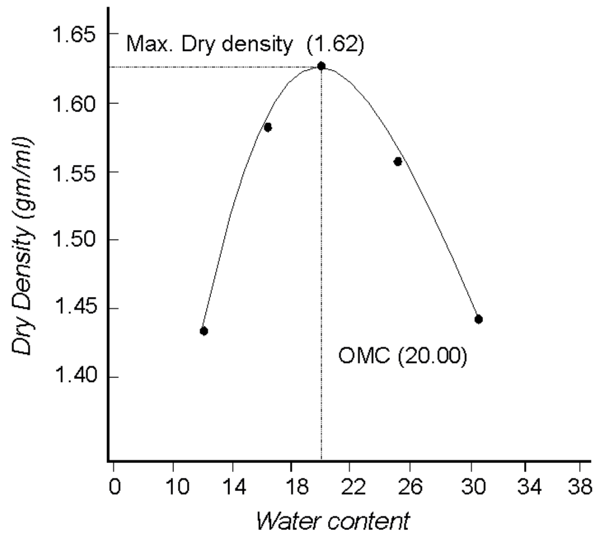


Fig. 6.7 Compaction curve for shell material

Permeability

In order to perform the permeability test compaction of the sample was done upto a compacted dry density of 2.24 gm/cc at 8 % optimum moisture content. Constant head permeability method has been adopted to perform this test. The coefficient of permeability as computed comes out to be 2.025 x 10^{-3} and 0.06 x 10^{-3} for TP-1 and TP-2 samples, respectively. The value of permeability thus obtained are within the permissible limits (1.5 x 10^{-5} to 3 x 10^{-3} cm/sec) as suggested by the USBR.

Shear Strength Parameters

For the present study sincere efforts were made to evaluate the properties of the construction material based on the realistic field observations supported with experimental data. The tests required for the determination of shear strength properties of the soils needs elaborate tests, which require sufficient time, adequate resource and financial support. Thus, all these factors made this elaborate test beyond the reach of the present study. However, efforts are made to evaluate the quality of the shell material by utilizing secondary data.

6.5.4 Construction Material for Filter

The filter is the most important component of the dam, which protects it from piping failure. The design of filter has to satisfy the two basic criteria; (i) the soil particles from the protected zone should not pass through the pores of the filter material and (ii) the filter should be much more pervious than the protected low permeability zone, this is to provide effective relief to hydraulic pressure inside that zone. Filters are constructed of fine to coarse sand and coarser gravel, which is sufficiently resistant to chemical action of seepage water and to the mechanical action during the placement (Bharat Singh and Varshney, 1995).

At Tendaho dam site the available materials for filter, fulfills the above requirements. The construction material available for filter is mainly river deposit of well-rounded material of Sandy gravels. The filter material is available at borrow Area-12 and Dubti area. The borrow Area-12 is located near Harsis, a small town located on the left bank of Awash River about 17 km up stream of the dam site. The material which is available at this borrow area includes fine sand, medium and coarse sand, different sizes of gravel and pebbles. The gradation of the material, as per the tests conducted by the Project Authorities, is in the range of gravel; 26 % to 53.3 %, sand; 46 % to 74 % and very few fines. The Dubti borrow area is located downstream of the dam site, about 10 km on left bank of Awash River. The construction material available at Dupti site mainly comprises of fine sand material only. The gradation of the material, as per the present study, is in the range of no gravels, sand, 98.17 % and fines 1.83 %.

In order to check the suitability of the filter material, for the present study, representative samples were collected from borrow Area-12 and Dubti area to establish the suitability of the material for filter. From borrow Area-12 four samples and from Dubti area one sample was collected. The test result for gradation and classification of the filter material is presented in Table 6.12 and Fig. 6.8.

Table 6.12 Test Results for Gradation and Classification of Filter Material

Borrow Area	Location	Depth (m)	Grain Size Distribution			Atterberg's Limit		Cu	Cc	Classification USC
			Gravel	Sand	Fine	WL	PI			
Area-12 (Previous Study)	A12 TP-2	0.00-0.35	47.5	52.5	-	-	-	4.7	0.06	SP
	A12 TP-3	0.35-0.55	53.3	46.7	-	-	-	5.5	0.06	SP
	A12 TP-4	0.25-0.40	26	74	-	-	-	1.81	0.22	SP
	A12 TP-5	Surface	0.7	99	0.3	-	-	2.42	1.02	SP
Area-12 (Present Study)	TP-1	0.0 – 0.7	47.26	50.89	1.85	-	-	10.0	0.54	SP
	TP-2	0.0 – 0.5	23.58	75.87	0.55	-	-	11.67	0.61	SP
	TP-3	0.0 – 0.6	68.7	30.9	0.4	-	-	35.0	0.03	GW
	TP-4	Surface	10.78	88.00	1.22	-	-	3.67	1.16	SP
Dupti Area	Dupti -1	Surface	-	98.17	1.83	-	-	10.42	0.42	SP

Filter Criteria

The primary purpose of the filter material is to provide a safe drainage and to check the migration of the particles from the protected zone. Therefore it is essential that the material selected for the filter must qualify these two basic requirements. For this, various filter criteria are available which helps as a useful guide in selecting the appropriate material for the filter. In the present study an attempt has been made to apply these criteria to identify the best suitable filter material.

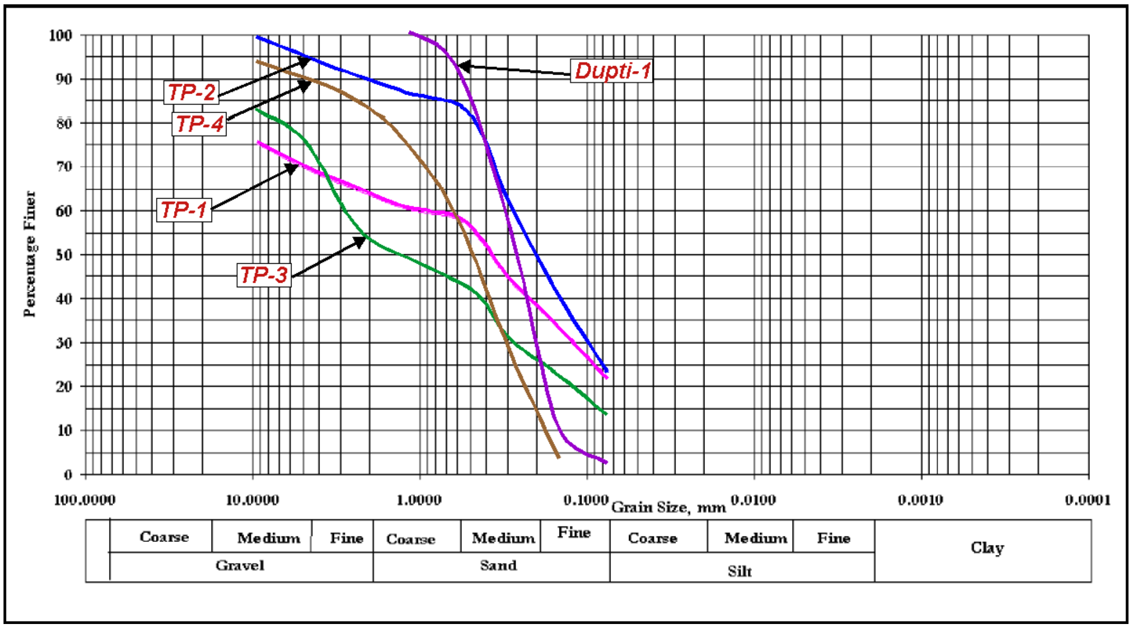


Fig. 6.8 Gradation of the filter materials from borrow Area-12 and Dupti Area.

The various filter criteria are discussed briefly here under;

Terzaghi's Criteria For Filter Selection

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

i) The 15% size of the filter material, D_{15}, must not be more than 4 or 5 times the 85% size, D_{85} of the protected soil to prevent the piping, i.e..

$$\frac{D_{15}\,of\ \ filter}{D_{85}\,of\ \mathrm{Pr}\,otected\ Layer} < 4\ to\,5 \qquad \text{……..6.5}$$

ii) The 15% size of the filter material, D_{15}, must be at least 4 or 5 times the 15% size, D_{15} of the protected soil, to ensure adequate permeability or,

$$\frac{D_{15}\,of\ \ filter}{D_{15}\,of\ \mathrm{Pr}\,otected\ Layer} > 4\ to\,5 \qquad \text{……..6.6}$$

Other requirements for a good filter are;

a) Its gradation curve should be approximately parallel to the gradation curve of the protected soil, especially in the finer range.

b) Filters should not contain more than 5% fines (-0.075 mm) and fines should be cohesion less. This is to ensure that filter remains adequately pervious and does not sustain a crack.

c) The filter does not have particles larger than 75 mm so as to minimize segregation.

d) If the base material ranges from gravel (over 10% > 4.75 mm) to silt (over 10% passing 75 μ), the base material should be analysed on the basis of gradation of fraction smaller than 4.75 mm.

The Terzaghi criteria have proved dependable over the years.

Recommendations For Filter Selection As Per Indian Standard (IS) Code

The recommendations for filter selection as per IS code are as follows;

i) $$\frac{D_{15}\ of\ filter}{D_{85}\ of\ base} < 5 \qquad \ldots\ldots 6.7$$

ii) $$\frac{D_{15}\ of\ filter}{D_{15}\ of\ base} > 4\ and < 20 \qquad \ldots\ldots 6.8$$

iii) $$\frac{D_{50}\ of\ filter}{D_{50}\ of\ base} < 25 \qquad \ldots\ldots 6.9$$

iv) The gradation curve of filter material should be nearly parallel to the gradation curve of the base material.

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

According to the U.S.Army corps of engineer recommendation for the filter criteria are;

i) $$\frac{D_{15}\ of\ filter}{D_{85}\ of\ base} \le 5 \qquad \ldots\ldots 6.10$$

ii) $$\frac{D_{50}\ of\ filter}{D_{50}\ of\ base} \le 25 \qquad \ldots\ldots 6.11$$

Sherard's Recommendations For Filter Design

a) The filter is successful in its function of arresting particles migration if;

$$\frac{D_{15}\ of\ filter}{D_{85}\ of\ base} < 9 \qquad \ldots\ldots 6.12$$

b) The size of the pore channel, which governs permeability is determined by the size of finer filter particles and will be represented by D_{15} size.

c) The filter gradation curve need not necessarily be parallel to the base material.

For filter criteria different particle sizes are used such as D15, D50 and D85 for filter material and the zone to be protected. Different criteria give different emphasis to these particle sizes. The particle size as deduced from the gradation curve for core material and filter material, to be used for Tendaho dam, are presented in Table 6.13 and Fig 6.3 & 6.8.

For the present study, filter criteria proposed by Terzaghi, India Standard, US Army Corps and Sherard has been applied to the filter material available at borrow Area-12 and Dupti area. A comparative assessment has been made and results are presented in Table 6.14. As a basic filter criteria it is required that the gradation curve of the filter material must be parallel to the gradation curve of the core material. However, Sherard's criteria is relaxed from this condition. A comparative gradation curve plot for filter and core material is presented as Fig. 6.9.

Table 6.13 Characteristic Particle Sizes of the filter and Core Material

Characteristic Particle Size	Filter Material		Core Material	
	Area-12	Dupti	Normal	Blended
D_{15}	0.22	0.14	0.0	0.007
D_{50}	0.5	0.21	0.025	0.08
D_{85}	-	-	0.007	6.5

Table 6.14 Summery result of the filter criteria

Criteria	Requirement	Filter Material Sites	Blended Material for Core		Normal Core Material	
			Result	Criteria	Result	Criteria
Indian	$\frac{D_{15}\ of\ filter}{D_{85}\ of\ base} < 5$	Area-12	0.032<5	Satisfied	31>5	Not Satisfied
		Dupti	0.012<5	Satisfied	21>5	Not Satisfied
	$\frac{D_{15}\ of\ filter}{D_{15}\ of\ base} > 4\ and < 20$	Area-12	31>20	Satisfied	NA	Not Satisfied
		Dupti	20=20	Satisfied	NA	Not Satisfied
	$\frac{D_{50}\ of\ filter}{D_{50}\ of\ base} < 25$	Area-12	6.25<25	Satisfied	20<20	Satisfied
		Dupti	2.65<25	Satisfied	5.4<25	Satisfied
US Army Corps of Engineers (1995)	$\frac{D_{15}\ of\ filter}{D_{85}\ of\ base} \leq 5$	Area-12	0.033<5	Satisfied	31.4>5	Not Satisfied
		Dupti	0.021<5	Satisfied	20<25	Satisfied
	$\frac{D_{50}\ of\ filter}{D_{50}\ of\ base} \leq 25$	Area-12	6.25<25	Satisfied	20<25	Satisfied
		Dupti	2.63<25	Satisfied	8.4<25	Satisfied
Terzagi	$\frac{D_{15}\ of\ filter}{D_{85}\ of\ base} < 4$	Area-12	0.034<4	Satisfied	31.4>4	Not Satisfied
		Dupti	0.021<4	Satisfied	20>4	Not Satisfied
	$\frac{D_{15}\ of\ filter}{D_{15}\ of\ base} > 4$	Area-12	31.4>5	Not Satisfied	NA	Not Satisfied
		Dupti	20>5	Not Satisfied	NA	Not Satisfied
Sherad's	$\frac{D_{15}\ of\ filter}{D_{85}\ of\ base} < 9$	Area-12	0.033<5	Satisfied	31.4>5	Not Satisfied
		Dupti	0.021<5	Satisfied	20<25	Not Satisfied

A perusal of Table 6.16 indicates that most of the criteria are satisfied when the blended material (35% coarse fraction) is used for the core. However, for unblended material most of the criteria are not satisfied. Thus, this justifies the need of blending of clay core material with the coarse fraction.

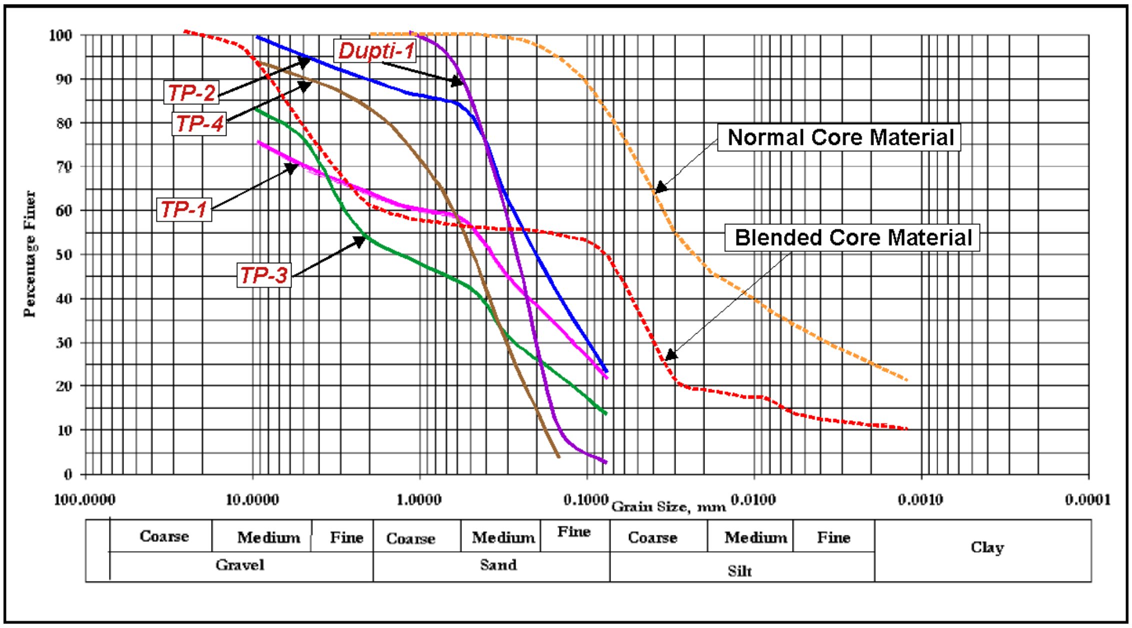


Fig. 6.9 Gradation curves of filter and core material

6.5.5 Construction Material for Riprap

Riprap is a layer provided on the upstream and downstream of the embankment to protect the embankment from the erosive action of wind, rain water and the wave action of reservoir. The material used to protect the upstream slope of embankment should be sound, resistant to erosion and sufficiently large to withstand the wave action from the reservoir. The different type of upstream slope protection measures commonly used are dumped stone Riprap, hand placed Riprap and soil cement slope protections. At Tendaho Dam site it is proposed to place a dumped stone riprap for the protection of the upstream embankment slope. Whereas, for the downstream slope protection it is proposed to use the hand placed riprap.

Dumped Stone Riprap for upstream slope Protection

Dumped riprap is preferred type of slope protection for earth dams. The rock fragment used for dumped riprap be composed of dense, sound durable rock with acceptable shape factor. The riprap must be underlain by bedding layers of fine material to act as filter to prevent the embankment material from being washed through the interstices in the riprap. The main criteria for the selection of riprap material depend upon the wave height generated in the reservoirs. For Tendaho Dam the maximum wave height has been calculated as 2.45 m. With this wave height and embankment slopes between 2:1 to 4:1 the average rock size to be used for riprap must be 0.61m (D_{50}) to the maximum weight of rock of 1814 Kg with layer thickness of 0.91m. In addition, Riprap should be well graded from a maximum size at least 1.5 times the average rock size to 2.5 cm spalls suitable to fill voids between rocks. Riprap blanket shall extend to at least 2.4 m below the lowest low water. Filter shall be provided between the riprap and embankment soils.

For Tendaho Dam site the Project Authorities have identified four potential areas for the riprap material. Three of these sites are located upstream of the proposed dam site at a distance of 0.3 km to 6.4 km. Whereas the remaining potential site is about 2.6 km downstream of the dam site. During the present study it was noted that the riprap material available at the sites proposed by the Project Authorities are not very suitable for the proposed riprap. The rock blocks available at these sites are rounded to sub rounded porphoretic basalt with poor gradation. Thus, the material available at these proposed sites may not be very suitable for the riprap.

During the present investigation an alternative site for dumped riprap has been identified. The site is about 14 km upstream from the proposed dam site along the highway to Addis Ababa. The rock present at site is aphenetic basalt with prominent three orthogonal joint sets. The rock is comparatively sound and less weathered. At the toe of the slope well graded Raveling slope material is also present, which may directly be transported with minimum sorting effort. The only demerit of the site is that it is comparatively beyond the economic haulage distance from the dam site. However, this extra hauling effort may be compensated by minimum excavation and sorting effort at this alternative site.

6.5.5 Overall Suitability of Construction Material

For Tendaho Dam most of the construction material is available in good quantity within the economic distance from the dam site. However, the quality of the construction material for the dam embankment needs to be improved by adopting certain measures, which are discussed hereunder;

Core Material

The soil samples from borrow Area 5 has been classified as CL, CH, MH and CL (USC system) as per the results of the project Authorities and the test conducted during the present study, respectively. According to Indian Standard Code (10) (12169-1987) ‘CL’ (Inorganic clays of low to medium plasticity) and ‘CH’ (Inorganic clays of high plasticity) are classified as 'fairly suitable soils' for impervious core. However, 'MH' (Inorganic silts or high plasticity) soils are considered as 'poor material' for the impervious core (Bharat Singh and Varshney, 1995).

As per the USBR permissible limits the permeability for CH class of soils must lie within the range of 0.05 $\pm$ 0.05 x 10^{-6} cm/sec, however the values are on the higher side as determined by the Project Authorities. Therefore, appropriate correction is required while constructing the core section. Core material compacted at moisture content wet of optimum will have lower permeability, high flexibility and lesser compressibility on saturation. As per US Army Corps Practice compaction is done at or above optimum moisture content. (Bharat Singh and Varshney, 1995).

The tests conducted during the present study indicates plasticity index (PI) values greater than 20 for TP-1 and TP-2 samples which shows that core material is erosion resistant. However, for the blended sample the PI value is 17, which does not satisfies the Sherard'criteria. According to Bharat Singh, 1995 the erosion resistance for blended core material may be improved by addition of stone chips to the extent of 10 - 20%, provided the stone chips are locally available. However, according to Indian Standard Code (10) (12169-1987) 'CL' soils (Inorganic clays of low to medium plasticity) are classified as 'suitable soils' for impervious core (Bharat Singh and Varshney, 1995).

The available core material at borrow Area-5 does not contain sufficient quantity of coarse material, as can be noticed from the gradation of the material (Fig. 6.3). Thus, in the absence of coarse fraction the core material may be less resistant to the erosion and chances of piping will be increased. Therefore, there is a necessity to blend the material with appropriate proportion of coarse fraction. The coarse material for the core is readily available at borrow Area-5 which may be blended with the clay at the borrow area itself. The blending proportion for coarse material may be kept in a range of 30-40%. However, to work out the exact proportion of the coarse material further testing of the blended material would be required. During the present study the filter criteria for the core were applied on a blended sample by taking a coarse fraction of 35%.

Shell Material

The classification tests conducted by the Project Authorities on the soil samples collected from the borrow Area-1, Area-3, Area-7 and Area-11 suggests that most of the soils falls in SW, GW and SP, GP class of Unified Soil Classification system, according to Indian Standard Code (10) (12169-1987) these classes falls in 'very suitable' and 'fairly suitable' category, respectively. However, the soils from borrow Area-7 falls in SW class as per the Unified Soil Classification system which make the soils very suitable for pervious shell.

The tested values for permeability and shear strength parameters show variation with the permissible values of USBR. However, this variation may be because the tests were conducted only on the sand size particles (plus 4.75 mm).

For the present study compaction test was performed by the Modified Proctor compaction method on a mixed soil sample from borrow Area-7. The computed test result from the density-moisture plot (Fig. 6.7) indicates a maximum dry density (MDD) of 2.24 gm/cm^3 can be achieved at an ‘Optimum Moisture Content’ of 8.0%.

Filter Material

For the present study, filter criteria proposed by Terzaghi, India Standard, US Army Corps and Sherard has been applied to the filter material available at borrow Area-12 and Dupti area. A comparative assessment indicates that most of the criteria are satisfied when the blended material (35% coarse fraction) is used for the core. However, for unblended material most of the criteria are not satisfied. Thus, this justifies the need of blending of clay core material with the coarse fraction.

Dumped Riprap Material

For Tendaho Dam site the Project Authorities have identified four potential areas for the riprap material. During the present study it was noted that the riprap material available at the sites proposed by the Project Authorities are not very suitable for the proposed riprap. The rock blocks available at these sites are rounded to sub rounded porphyritic basalt with poor gradation. However, during the present investigation an alternative site for dumped riprap has been identified. The site is about 14 km upstream from the proposed dam site along the highway to Addis Ababa. The rock present at alternative site is aphanitic basalt with prominent three orthogonal joint sets. The rock is comparatively sound and less weathered. At the toe of the slope well graded Raveling slope material is also present, which may directly be transported with minimum sorting effort. The only demerit of the site is that it is comparatively beyond the economic haulage distance from the dam site. However, this extra hauling effort may be compensated by minimum excavation and sorting effort to be made at this alternative site.

Carbonate Content in Core and Shell Material

During the present study core and shell material was tested with HCL at all borrow areas. The test indicates the presence of carbonate content in core and shell material, to be used for the embankment. The presence of carbonate in the shell and core material may result into solution action by the impounded reservoir water. This may result into piping of the material. Therefore, there is a need to adopt proper remedial measures to prevent the piping due to solution of carbonate material in the core and shell material. The blending of core material with a coarse fraction and inclusion of stone chips upto 10 – 20% may improve the erosion resistance of the core material. This may reduce the chances of piping in the core material. According to Bharat Singh and Varshney (1995) ‘‘The possibility of improving erosion resistance of the core material by blending a small percentage of bentonite and/ or by the addition of coarse graded fractions should be actively considered where satisfactory core material is not available, especially in the case of earthquake zones. With suitable construction procedures the additional cost may well be commensurated with increased confidence in the safety of the structure’’. Further, for the shell material the piping effect due to solution action of carbonate may be reduced if the permeability and the flexibility of the material are reduced. This may be achieved if compaction of the material is done at moisture content wet of optimum (Bharat Singh and Varshney, 1995). The methods discussed above are the indirect means of reducing the piping effect however, systematic study are required to know the rate and amount of carbonate solution action under varied hydraulic head.

Chapter VII FOUNDATION TREATMENT AND SEEPAGE CONTROLS

7.0 Instruction

Treatment of foundations for embankment dams is primarily directed towards control of adverse effects of seepage. These may be considered in two broad categories; rock foundations and foundations of disintegrated, alluvial material, which may vary from clay to silt, sand and gravel or boulder, and may be a heterogeneous mix or consists of layered formations. Alluvial foundations at each site present a unique problem and the available design methods and technologies have to be specific for site conditions to obtain an optimal solution. Rock foundations are generally considered as stable, unless there are seams of weak argillaceous shale or clay at a shallow depth. Under such foundation condition appropriate stability measures has to be adopted so as to ensure stability against sliding or uplift problems associated with pore water pressures. Alluvial foundation could consist of pervious material such as sand, gravel, boulder etc. or of clayey material. Foundations are seldom homogeneous and uniform; however, unless there is a clear indication of stratification, average conditions may be assumed for design purposes.

The primary object of any dam project is to impound water behind it. This water if seeps through the embankment, abutment or through the dam foundation in excessive quantity may damage the dam partially or fully resulting into failure of the project. Therefore, it is very important to control the seepage through embankment dam. Two approaches are followed to control the seepage through an embankment dam. The first approach is preventive whereas, the other approach is curative. In earth dam design practice both the approaches are followed. In preventive approach efforts are made in keeping the water out in so far as possible while in the curative approach a safe outlet is provided to water, which has entered in spite of the preventive measures. In preventive approach an impervious zone (generally called as core) or an impervious membrane is provided in the embankment. In foundations the commonly used methods are; cut off trenches, grout curtain, concrete diaphragm walls, sheet piles, other thin cut offs and impermeable upstream blankets. In curative approach the requirement is a drainage system such that seepage forces will not be able to cause soil migration and their magnitude and direction is such that they cannot cause embankment sliding or foundation blowout. For the embankment, zoning of the section is the most effective method. However, if an adequate quantity of free drained material is not available or is uneconomical embankment drainage is ensured through the rock toe, horizontal drainage blanket and chimney drain, they are used singly or in combination. For foundation drainage a horizontal drainage blanket, partially penetrating toe drains, vertical sand drains or relief wells may be provided (Bharat Singh and Varshney, 1995).

The foundation of Tenaho Dam comprise of five different parts, left abutment, valley floor on the left bank, the river channel bed, valley floor on the right bank and right abutment. The left abutment ridge is occupied by volcanic sequence of basaltic rock, which is moderately jointed, and partially weathered. It also contains inter bedded palesoil layer exposed on the upstream slope face between elevations ± 388m to ±378m .The river floor on the left bank is covered by top soil and riverine alluvium. This unit extends up to an elevation of ±367m along the axis and laterally extends ±373m on upstream and downstream of the dam axis. The river channel is covered by recent alluvial of silty sandy soil which has a thickness of about 13m over the lake sedimentary rocks. The right riverbank forms a wide expanse of alluvium, which extends up to the toe of the right abutment. The depth of this alluvium deposit, near to the river bank, is 15m and it is around 8m in the river section. The right abutment is covered by basaltic rock, which is intruded by the major joint system. The orientation of the joints is mainly across the flow direction.

For the present study a systematic assessment of the foundation conditions has been carried out and based on the conditions suitable remedial measures have been worked out.

7.1 Seepage and Leakage problems

Most of the seepage problems are related to the geological, hydrological and geotenchical conditions at the Dam foundation. Besides, the seepage problems are also associated with the main dam body also. Seepage through Dam foundation due to unfavorable geological conditions is an indicator of a risk of failure. Clevenger (1974) estimated 10% of Dam failures are associated with foundation seepage. Tendaho Dam site is located in the central part of the Afar region, where complex geological formation due to the three-rift systems is present. Therefore, most of the likely foundation problems at Tendaho dam site are associated with the geological setup of the area.

7.1.1 Seepage in the Abutment areas

Based on the surface and subsurface explorations an attempt has been made to assess the foundation conditions present at the abutments of the proposed dam. For this purpose surface explorations by taking traverses through the core trench and across the ridge were taken. Besides available borehole data has also been utilized to workout the subsurface geological condition. The abutments are composed of volcanic sequence of basaltic rock. The basalt of different varieties such as; porphyritic, vesicular, scoriaceous, aphanitic and tuff interlayered in varying thickness is well-exposed form bottom to top section on both the abutments. The contact between the different varieties of basalts are clear along the ridges and the rock mass is moderately jointed and partially weathered, intruded by open joints with varying spacing and continuity.

Left Abutment

The left abutment slope is moderately steep with an overall slope angle of about 28^o and the general slope direction is S28^{o}E. The height of the left abutment is about 52 m from the river bed level. The rocks exposed on the left abutment are different varieties of basalts such as; porphyritic, vesicular, scoriaceous, aphanitic and aphanitic interlayered in varied thickness (Fig.7.1).

The rocks are partly weathered and moderately jointed with varying spacing and continuity. A paleosoil layers at elevation of around ±388m and ±378m has been observed on the upstream face of the left abutment. However, this unit is not observed on borehole BH-TT2 result, which has been drilled from the top of the ridge and passes vertically through the center of the ridge down below the riverbed. This indicates that this paleosoil layer do not extend in the down steam of the ridges and may pinch out in between. The rocks within this zone are highly shattered and weathered to high degree. This zone could be the possible zone of excessive seepage and may pose stability problems under the influence of varied thrust of the impounded water. Thus there is an immediate need to provide suitable remedial measures to treat this weak zone, which forms a part of the dam foundation. The possible remedial measures for this zone are discussed later in this chapter. The contact zones between different verities of basalts exposed on the left abutment, may cause serious seepage across the abutment of the dam. Therefore, seepage through these contacts has to be controlled by following appropriate treatment methods.

The permeability test conducted in borehole (BH-TT2) by packer test, have indicated complete water loss from surface up to an elevation of ± 400m. Further below, again high water loss between elevation ± 400m to ±367m has been observed. Between elevation ±367m to elevation ±351m permeability varies from 15 to 1 Lugeon. Thus, on the basis of this permeability results the abutment ridge is classified as highly jointed and fractured from elevation ±421-400m, open and closely spaced from elevation of ±400 to ±367m and tight to widely spaced joints below elevation ±367m. Therefore, seepage along the abutment may be critical if it is not treated properly. Moreover, from the surface observation on the left abutment slope the volcanic rocks are highly weathered and jointed therefore stripping of the weathered rock upto a depth of fresh rock is necessary

Right Abutment

From the surface observation along the right abutment, unlike the left abutment, it is covered by intact rock blocks removed from the parent rock mass. The geology in general exposed on the right abutment slope is more or less similar to that of left abutment, the geological sequence as observed from the surface geological traces and the borehole data (BH-TE) is presented as Fig. 7.1. The rock mass is dissected by three major joint systems; which strike NNW-SSE with sub-vertical dips, strike East-West with dip of 70^0 towards north and Strike NE-SW with sub vertical dip. However, in addition to these two more joint sets with comparatively less persistence are also present. Since the flow direction of the river is NNE – SSW, part of joint system is along the flow direction and across, this may result into seepage problem if not treated adequately.

The permeability test in borehole (BH-TE) conducted by packer tests result indicate that from surface up to elevation ± 374m complete water loss has been observed. In the further below section between elevation ± 374m to ±349m high water loss (45 Lugeon) has been observed. Thus the result indicates that there will be a serious seepage problem along the abutment if not treated adequately.

Thus based on the surface and subsurface explorations the seepage condition along the abutments may be summarized as; on both the abutments variety of basaltic rocks are present, which are partly weathered and highly jointed. The permeability result indicates that in the top reaches the rocks are highly permeable as on both the abutments there was total water loss during the water pressure tests conducted within this reach. The permeability in the intermediate reach on both the abutments is again high as the Lugeon value is more than 45. Further, on the left abutment within ±388m and ±378m a zone of paliosoil is also present which is highly cracked and very weak in strength. Excessive seepage may take place through this zone as it extends further downstream, however, from the borehole data it has not been traced in the central part of the abutment. In addition to this, the various contacts between different varieties of basalts are the potential areas for seepage. Therefore, it is necessary to provide adequate seepage measures in the abutment section of the dam foundation, which are discussed later in this Chapter.

7.1.2 River section leakage problem

Surface geology of the river section, as observed during the filed work, is covered by recent alluvial deposits, which are characterized by gravel, sand, silt and clay soils. The subsurface geological conditions, during the present study, have been explored by exploratory test pits and by analyzing the drill hole data. In the river section the Project Authorities have conducted exploratory drilling at 7 locations; three in the left river bank (BH-TD, BH-TC, BH-TC1), one in the river channel section (BH-TD1) and three in the right bank (BH-TG1, BH-TG, BH-TF), respectively (Fig. 7.1 and Table 7.1).

The borehole results, along the left riverbank indicate 5-8 m thick alluvial deposit, which is overlying on the lakes sedimentary sequence which is about 20m thick. Further, below this unit weathered and fractured basalts are present. Along the river channel section the recent alluvium deposit is about 13m thick, which is overlying on the lake sedimentary rock of 10m thickness and below this unit basaltic rocks, which are highly fractured are present. Further, along the right river bank the thickness of the alluvial deposits is around 15m, lake sedimentary is 20m thick and below this basaltic rocks are present. Therefore, the basaltic rock will form a bed rock foundation for the proposed dam. The width of the alluvial deposits is comparatively wider and thick towards the right bank. Thus, this could be the possible potential area for excessive leakage. Two-insitu permeability test has been conducted in the alluvial deposits by falling head method in borehole BH-TG1 on the right bank. The result indicates a high lugeon value of 18, which shows the alluvial deposits are highly preamble. This may affect the foundation of the dam by piping and erosion of the foundation material. The Project Authorities in the riverbank area conducted resistivity profiling survey. These tests were conducted one on each bank. The results indicates presence of two geological faults. One fault is reported 40m upstream from the dam axis, which has a width of 40m and having a depth of about 15 to 20 m (Geological investigation of TDIP, 2005). According to the report the faults are extending to the lake sedimentary rocks, since the maximum thickness of alluvium deposit is <15m.

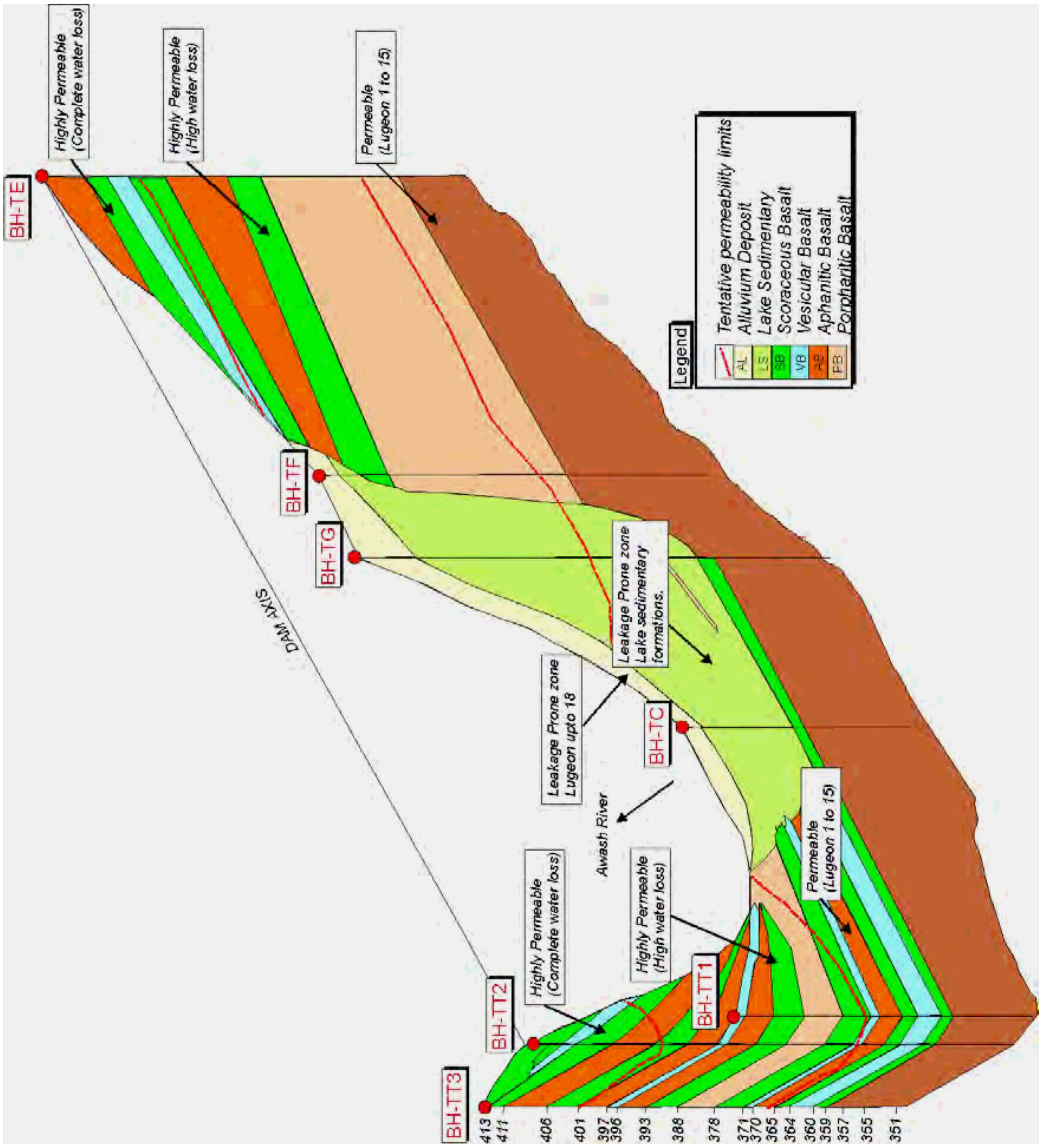


Fig. 7.1 **Geology in surface and subsurface ,Seepage condition through abutment and river section**

Therefore, there is a possibility of excessive leakage through these faults as these are wide open. However, the magnitude of such leakage will depend upon the general trend and the continuity of these fault zones. During the present study no detailed geophysical data on this account was made available.

The leakage conditions particularly in the river section can be summarized as ; the presence of Alluvium deposit in the river section may pose problems of excessive leakage. The thickness and the lateral extent of this deposit are more towards the right bank where its thickness is about 15 m and it extends some 300 m in upstream direction. Moreover, in the downstream direction it extends over a large area. Thus there is a possibility of free flowing conditions through this alluvium deposit. In addition to this, the leakage problem associated with the underlying lake sedimentary formations must also be addressed. As this zone comprises of highly jointed silt stone, mud stone and sandstone. The core recovery suggests that the rock mass condition in this sequence is of very poor quality. Thus, provides a possible zone for excessive seepage in the foundation area. Further, the basaltic bed rock is also highly jointed and the RQD value varies in a wide range of 22 –94%. This also suggests a possible seepage through the bedrock foundation. Moreover, limited Geophysical data also suggests the presence of two longitudinal faults around the dam foundation, which extends upto the lake sedimentary sequence. These possible faults are also potential zones of excessive seepage however, the magnitude of such seepage will entirely depends upon the orientation and lateral extent of these faults.

Thus based on the existing geological conditions present on the surface and subsurface, a systematic foundation treatment has to be carried out.

Table 7.1 Location and extent of different units present along the dam axis

Location	Borehole	Elevation (m)	Alluvial depth EL from	Sedimentary rock depth from EL	Basaltic rock depth from EL	Remark
Left river bank	BH-TD	374.15	374-367, 7m	367-358, 9m	Below 358	At foot
	BH-TC	374.41	374-367, 7m	367-358, 9m	Below 358	
	BH-TC1	377.79	374-367, 7m	367-358, 9m	Below 358	
Left Abutment	BHTT_1	397.855	397.8–395.475	-	395.475- 357.885	Tunnel inlet
	BHTT_2	421.73	-	-	421.73- 351.27	Top of the ridges
	BHTT_3	421.73	-	-	421.73-361.73	D/s of the ridges
	BHTT_4	382.906	-	382.906-375.796	375.796-352.906	
River section	BH-TD_1	370.505	370.505-358	358-345.5	345.5-337.7	
Right river bank	BH-TG1	377.531	377.51-362	362-327	-	
	BH-TG	337.146	377.146-365.6	365.6-323.65	323.65-298.4	
Right abutment	BH-TE	410.399	-	-	410.9-340.31	
	BH-TF	377.652	377.652-374	374-344	344-312	At foot
Source: Tendaho Dam and Irrigation Project, Geological and Geotechnical Report, 2005						

7.1.3 Seepage through the Embankment

The embankment dams are known to have seepage problems mainly because of the poor selection of the construction material or due to improper design considerations. Water seepage under pressure through soil voids is accompanied by a mechanical drag on the soil particle when these force exceeds, the soil grain movement may take place. A large percentage of the earth dam failures reported by the Sherard & colleagues was due to seepage (Bharat Singh & R.S Varshney ,1995). As a result, it is important to control the migration of soil particles resulting piping failure and embankment failure by saturation or seepage forces. The migration is mainly caused due to the lack of filter protection, poor compaction, in proper placement of pervious material in the embankment section and leaching of dispersive soils. The saturation of seepage forces is mainly due to the excessive pore pressure causing slope failure, liquefaction failures due to earthquake shocks, foundation blow out due to excessive uplift and sloughing of downstream toe due to saturation. Therefore, drainage of an embankment is necessary to provide a safe passage to the water, which has entered into the dam body, without developing excessive pore pressure. The construction material proposed to be used for the shell and core of the Tendaho Dam project has a potential for seepage. Certain methods have already been proposed through this study in Chapter 6, which may help to control the seepage through the embankment or by providing a safe passage to the water which has entered into the protected zone of the embankment. A detail description on this account is given later in this chapter.

7.2 Seepage controlling methods through dam foundation

As indicated in the above section, seepage problem is the most significant problem in the foundation of the dam. The seepage problems can be treated by employing seepage controlling methods in the dam foundation. The seepage control includes methods to reduce seepage force in the foundation area or to increase the path of seepage to guard against erosion of the foundation material. Other treatment includes employing such methods, which completely seals the permeable zones.

For the present study by utilizing the surface and subsurface exploration methods, a systematic evaluation of the anticipated seepage conditions through the dam foundation has been carried out. Based on the observations suitable remedial measures to control the seepage/ leakage problems from the foundation are discussed in the following paragraphs.

7.2.1 Curtain Grouting

The water pressure test conducted in the boreholes at abutments indicates very high to medium permeability. Also in the river section there is a need to check the permeability, particularly in the lake sedimentary formation. Therefore the most effective means of checking seepage through the abutments and the river section would be a provision of grout curtains. The curtain grouting is done in split spacing or closure method in which primary, secondary and tertiary sequence is done until water pressure test provides required water tightness of the test section.

Based on the geological and the seepage conditions prevailing on the abutments and the river section the project Authorities have proposed curtain grouting to a depth of 24 m. In the abutment sections the grout holes will be at an inclination of 30^0 from the vertical, dipping towards north on left abutment and dipping towards south on right abutment. In addition, descending pattern grouting technique is recommended. Three pattern of grouting has been proposed these are “A”,”B” and “C”. The ‘A’ pattern or the Primary holes configuration is followed as two rows of holes spaced 3m from each other and arranged at corners of 3 x 3m squares. After completing the primary holes, water pressure may be checked in test grout holes if the permeability exceeds by 3 Lugeons and grouting intake is greater than 35 kg/m^3, secondary holes (“B”) should be drilled at the intersection of diagonal of the square and grouting to be conducted through these secondary holes. Even after providing the ‘B’ pattern holes if the desired permeability is not attained in such cases tertiary grouting (‘C’ patter) may be provided.

According to the project Authorities the grout curtain in the abutments would be provided upto an elevation of 379 m. However, the results of the water pressure tests conducted on both the abutments indicates the necessarily of grout curtain below this level.

In the present study the depth of curtain grouting in the abutment has been estimated by utilizing Ewert’s (1985) relation;

$$D=1/3H + C \qquad \text{……7.1}$$

Where, ‘D’ is the depth of grout curtain, ‘H’ is the height of the Dam and ‘C’ is the variable constant depending up on the foundation condition and the height of the dam, the value of ‘C’ ranges from 8 to 25.

Based on the surface and subsurface observation the rock mass present in the dam foundation show poor RQD and very high permeability. In addition the rock mass in general, on both the abutments falls into ‘fair class’. Thus keeping all these facts in mind it may be justified to adopt value of constant ‘C’ as 20.

Depth of grout curtain: D = 1/3 x 42 + 20 = 34 m

Therefore, it may be safe to provide a grout curtain upto a depth of 34 m in the river section. However, this depth may be reduced on the abutment section as the seepage is a direct function of hydraulic head.

7.2.2 Diaphragm wall

On the left abutment within ±388m and ±378m a zone of paliosoil is present which is highly cracked and very weak in strength. Excessive seepage may take place through this zone as it extends further downstream, however from the borehole data it has not been traced in the central part of the abutment. There is a need to provide a proper treatment for this zone. If left untreated the embankment in immediate contact of this zone will always be saturated. This may result into piping or cracking due to variation in saturation of adjoining zones within the embankment. Further, the paliosoil under saturation conditions may further loose its strength, which may result into other related stability and seepage problems. Fig 7.2 (a) shows a schematic sketch of the paleosoil zone.

The possible effective treatment method is to provide a diaphragm wall, all along its extent, in the upstream reach. This can be done by excavating 2-3 m of the weak Palo soil and later it may be backfilled with a mixture of clay material with betonite. Other alternative is to constructing a RCC diaphragm wall (Fig.7.2(b)). Another alternative is to provide a radiating curtain grout along the periphery of the ridge. The grout holes may be placed just above the paliosoil zone and may be inclined at about 30^{o} from the vertical. Before finalizing this method it is necessary that test grouts should be performed to check weather the paliosoils accept grout or not.

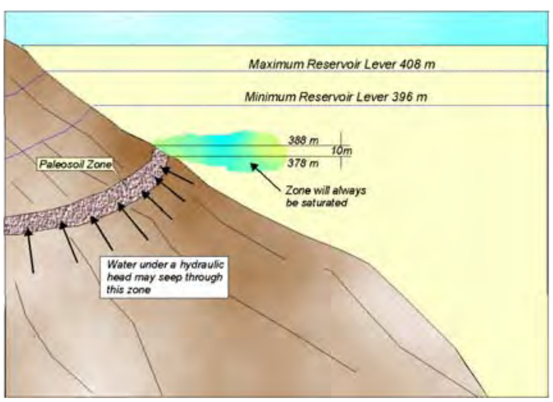


Fig 7.2 (a) A schematic sketch of the paleosoil Zone along the left abutment.

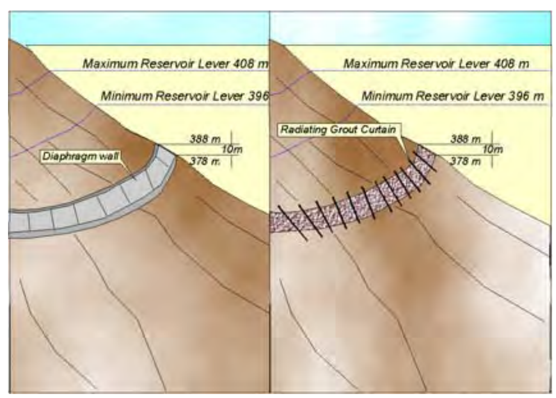


Fig 7.2 (b) Remedial measures to control seepage through paleosoil zone.

However, a detailed economic feasibility study will be required to finalize the most feasible method. These remedial measures will not only control the seepage problem across the ridge but also control the stability problem in the abutment ridges along the weak zones during reservoir draw down conditions.

7.2.3 Upstream Impervious Blanketing

The soils present in the foundation area have been classified as SP and SW, which are known to be pervious by their nature. Since, these soils forms the part of the foundation therefore these soils will be subjected to the maximum hydraulic pressures after the construction of the dam. As per the design proposal it is proposed to excavate these soils to its full depth below the core of the dam. Also in the shell area it is proposed to excavate these soils to a depth upto which it attains a relative density more than 55%. However, the proposed method of removing the soil from the foundation area may not be effectively reducing the seepage problem. Therefore, in order to reduce the hydraulic pressure at the base of the core and the shell section measures must be adopted to reduce the seepage force. The most economical and effective method of reducing seepage force in the base area of the embankment is by providing upstream impervious blanketing. The upstream impervious blanketing helps to minimize the possibility of subsurface erosion near to the base area and it helps in reducing the seepage force by extending the seepage path below the embankment. 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. The construction material for impervious blanket is available with in the economic distance from the dam site. The fine material may be available in suitable quantities from borrow Area-3 and Area-9.

7.2.4 Cut off Trench

Seepage through pervious foundation may be cut off by providing a trench extending to the bed rock and backfilled with the impervious material. This is the most positive means of controlling seepage and ensuring that no problem will be encountered either from piping through the foundation or heaving due to excessive seepage pressure at the down stream of the toe. The proposed design of the Tendaho dam envisages to construct a cut off trench in the base area of the core. As per the design the trench will be of trapezoidal section with a top width of 10m with sides inclined at 1:1.5.

7.2.5 Consolidation grouting

Consolidation grouting in the dam base area is mainly done to improve the physical properties of the foundation rocks. It helps in closing all the openings fishers, joints in the foundation, which not only weakens the foundation but also prevents the seepage through the open joints. Mistry (1983) proposed that consolidation grouting should be extended up to b/4 in the heel portion and b/3 towards the toe, where b is the base width of the dam along river section perpendicular to dam axis. The extension of consolidation grouting in the dam base area is mainly done because under the reservoir full condition the concentration of stresses is towards the toe region whereas, under the reservoir drawdown condition the stress concentrations is towards the heel portion of the dam. In the river bed section consolidation grouting is done upto a depth of 10-15 m whereas, in abutment section it is done upto a depth of 5-15m. However, at Tendaho Dam as per the design the Project Authorities proposed a consolidation grouting upto a depth of 8 m in the abutment and the river section of the foundation. At Tendaho dam, the foundation rocks are affected by open joints and the overall RQD result of the boreholes indicate that the rocks are of poor strength. Therefore, consolidation grouting is an effective remedial measure not only to increase the strength of the foundation rocks but also to control the seepage through the foundation.

7.3 Seepage Controls for Embankment

The importance of the seepage control through embankment is to control the water loss with in the acceptable limits and to ensure the safety of the Dam. Water seepage under pressure through soil voids is accompanied by a mechanical drag on the soil particle when these force exceeds, the soil grain movement may take place. A large percentage of the earth dam failures reported by the Sherard & colleagues was due to seepage (Bharat Singh & R.S Varshney ,1995). As a result, it is important to control the migration of soil particles resulting piping failure and embankment failure by saturation or seepage forces. The migration is mainly caused due to the lack of filter protection, poor compaction, in proper placement of pervious material in the embankment section and leaching of dispersive soils. The saturation of seepage forces is mainly due to the excessive pore pressure causing slope failure, liquefaction failures due to earthquake shocks, foundation blow out due to excessive uplift and sloughing of downstream toe due to saturation. Therefore, drainage of an embankment is necessary to provide a safe passage to the water, which has entered into the dam body, without developing excessive pore pressure.

Two approaches are followed to control the seepage through an embankment dam. The first approach is preventive whereas, the other approach is curative. In earth dam design practice both the approaches are followed in combination. In preventive approach efforts are made in keeping the water out in so far as possible while in the curative approach a safe outlet is provided to water, which has entered in spite of the preventive measures.

The effective seepage control methods for the Tendaho Dam are discussed in the following paragraphs.

7.3.1 Downstream Free-Draining Zone or shell

Downstream shell to act as a drain in zoned section depends on ratio of permeability of the pervious material to impervious material used in the core section. The material suitable for a pervious shell as per the I.S code [10] (12169-1987) is SW and GW. In the project area this material is available in adequate quantity and has been proposed for the utilization for the construction of the downstream shell. Thus, the downstream shell will be serving as a free drainage zone and will be effective in reducing the seepage forces developed in the dam body.

7.3.2 Rock toe and Drains

The downstream toe of an embankment dam is the most critical region in respect of seepage instability. The entire seepage tends to concentrate around downstream toe, if internal drainage is not provided. The soil mass in this region is subjected to excessive seepage forces. This may cause heaving and sloughing of the toe if not properly protected. Rock toes, drainage blanket and filter drains are provided on the downstream of the dam to; (i) Provide a controlled outlet to seepage, (ii) to lower the seepage line and keep it within the downstream face and (iii) to prevent piping and heave at the downstream toe and thus improves the stability of the dam against seepage.

For dams of low to moderate heights and where rock is available a rock toe of 1/4 to 1/3 the height of the dam can be provided. Since the quantity required in rock toe of a dam would be about 10% of the total quantity of fill required, the rock toe may be expensive in some situations. In order to check the migration of the particles from the shell and the foundation into the rock toe it has to be protected by the filters (Bharat Singh and Varshney, 1995).

At Tendaho Dam site there is no scarcity of suitable rock for the construction of the rock toe. As per the design a 12 m rock toe has already been proposed by the Project Authorities.

Toe drains are provided along the downstream toe of a dam in conjunction with a rock toe or horizontal drainage blanket to drain off the seepage water emerging from the body. Toe drains are also useful in collecting foundation seepage. The depth of the toe drains is usually 1.5 m with a minimum bottom width of 1 m and side slopes 1:1. A suitable slope is to be provided to this drain so that water can drain into natural stream.

7.3.3 Horizontal Drainage

For dams of low to moderate height horizontal drainage blankets are used to drain the embankment as well as the downstream portion of foundation. The length of horizontal drainage depends on the flow- net; however, U.S.B.R recommended that the length of the blanket be equal to three times the height of the dam. Moreover, it must be sufficient cross-section to convey the maximum quantity of seepage estimated to come through the dam section since it is supplemented by the chimney drains on the upstream side and with toe drains on the downstream side.

The suitable materials for the construction of the horizontal drainage is the coarse material having high permeability. The horizontal drainage must be protected by the filter so as to check the migration of any fine particles into the horizontal drainage section.

The foundation material at Tendaho Dam site has been classified as SP and SW, therefore there is a fair possibility of migration of fine particles from the foundation into the horizontal drainage. To protect the migration of the particles, provision of a filter in between the dam base and the horizontal drainage is desirable.

7.3.4 Chimney Drains

Chimney drains are the most important seepage controlling methods especially in the zoned embankment. Chimney drains intercepts all layers of the dam section in the seepage zone. Thus controls the seepage emerging in the downstream face of the dam. The chimney drains helps in reducing pore water pressure. Chimney drains may be vertical or inclined, upstream and downstream. An upstream inclined drain provided in an impervious fill makes the upstream portion behave more or less as a thin core. The water from a chimney drain has to be taken out through horizontal drains and a toe drain. The chimney drain has to be protected on all sides by filter layers. The advantage of providing chimney drains in the earth dam is to keep the downstream portion free from seepage when the reservoirs is full. The Chimney drains also helps in reducing the drawdown and construction pore pressure.

As per the Tendaho Dam design the seepage control through the embankment has been provided through impervious core, upstream transition zone, inclined chimney, filter downstream of core, horizontal filter, rock toe and toe drains. In addition, 2m thick transition zone on the upstream face of clay core has been provided to guard against internal piping.

Chapter VIII CONCLUSION AND RECOMMENDATIONS

8.1 Conclusion

Identification of possible adverse and unfavorable conditions in the dam foundation area in the initial stages of investigation helps in adopting proper remedial measures. The "Tendaho Dam" discussed in this study is an embankment dam, which will be constructed across the Awash river at Tendaho. The main purpose of this dam project is to provide irrigation to sugar cane plantation covering a total area of 60,000 hectares. Therefore, it becomes essential that the dam foundation must be stable against the loads imposed by the structure and the impounded water. Besides, the dam foundation should be free from any excessive seepage. Further, the construction material used to form the various zones of the embankment should be appropriate as possible and should be resistant to the piping failure.

The main objective of the present study was to determine the suitability of the dam foundation and the embankment against seepage. Besides, it was intended to study the suitability of the construction material. In addition an attempt has been made to identify the possible adverse seepage problems in the dam foundation.

The borrow Area-5 has been identified by the Project Authorities as the potential source for the core material. The soil samples from borrow Area 5 was classified as CL, CH and MH (USC system) as per the results of the project Authorities and the test conducted during the present study, respectively. According to Indian Standard Code (10) (12169-1987) 'CL' (Inorganic clays of low to medium plasticity) are classified as 'suitable soils' for impervious core and 'CH' (Inorganic clays of high plasticity) are classified as 'fairly suitable soils' for impervious core. However, 'MH' (Inorganic silts or high plasticity) soils are considered as 'poor material' for the impervious core. As per the USBR permissible limits the permeability for CH class of soils must lie within the range of $0.05 \pm 0.05 \times 10^{-6}$ cm/sec, however the values are on the higher side as determined by the Project Authorities. Therefore, appropriate correction is required while constructing the core section. Core material compacted at moisture content wet of optimum will have lower permeability, high flexibility and lesser compressibility on saturation. As per US Army Corps Practice compaction is done at or above optimum moisture content.

The tests conducted during the present study indicates that plasticity index (PI) values greater than 20 for TP-1 and TP-2 samples which shows that core material is erosion resistant.

However, for the blended sample the PI value is 17, which does not satisfies the Sherard'criteria. According to Bharat Singh, 1995 the erosion resistance for blended core material may be improved by addition of stone chips to the extent of 10 - 20%, provided the stone chips are locally available. However, according to Indian Standard Code (10) (12169-1987) 'CL' soils ((Inorganic clays of low to medium plasticity)) are classified as 'suitable soils' for impervious core.

The available core material at borrow Area-5 does not contain sufficient quantity of coarse material, as can be noticed from the gradation of the material. Thus, in the absence of coarse fraction the core material may be less resistant to the erosion and chances of piping will be increased. Therefore, there is a necessity to blend the material with appropriate proportion of coarse fraction. The coarse material for the core is readily available at borrow Area-5 which may be blended with the clay at the borrow area itself. The blending proportion for coarse material may be kept in a range of 30-40%. However, to work out the exact proportion of the coarse material further testing of the blended material would be required.

The soils from borrow Area-7 falls in SW class as per the Unified Soil Classification system which make the soils very suitable for pervious shell. The tested values for permeability and shear strength parameters show variation with the permissible values of USBR. However, this variation may be because the tests were conducted only on the sand size particles (plus 4.75 mm). For the present study compaction test was performed by the Modified Proctor compaction method on a mixed soil sample from borrow Area-7. The computed test result indicates a maximum dry density (MDD) of 2.24 gm/cm^3 which can be achieved at an 'Optimum Moisture Content' of 8.0%. Therefore, it would be advisable to follow this optimum moisture content while compacting the shell material.

In order to work out the effectiveness of the filter material, filter criteria proposed by Terzaghi, India Standard, US Army Corps and Sherard has been applied to the filter material available at borrow Area-12 and Dupti area. A comparative assessment indicates that most of the criteria are satisfied when the blended material (35% coarse fraction) is used for the core. However, for unblended material most of the criteria are not satisfied. Thus, this justifies the need of blending of clay core material with the coarse fraction.

For Tendaho Dam site the Project Authorities have identified four potential areas for the riprap material. During the present study it was noted that the riprap material available at the sites proposed by the Project Authorities are not very suitable for the proposed riprap. However, during the present investigation an alternative site for dumped riprap has been identified. The site is about 14 km upstream from the proposed dam site along the highway to

Addis Ababa. The rock present at alternative site is aphanitic basalt with prominent three orthogonal joint sets. The rock is comparatively sound and less weathered. The only demerit of the site is that it is comparatively beyond the economic haulage distance from the dam site. However, this extra hauling effort may be compensated by minimum excavation and sorting effort to be made at this alternative site.

During the present study core and shell material was tested with HCL at all borrow areas. The test indicates the presence of carbonate content in core and shell material, to be used for the embankment. The presence of carbonate in the shell and core material may result into solution action by the impounded reservoir water. This may result into piping of the material. Therefore, there is a need to adopt proper remedial measures to prevent the piping due to solution of carbonate material in the core and shell material. The blending of core material with a coarse fraction and inclusion of stone chips upto 10 – 20% may improve the erosion resistance of the core material. This may reduce the chances of piping in the core material. Further, for the shell material the piping effect due to solution action of carbonate may be reduced if the permeability and the flexibility of the material are reduced. This may be achieved if compaction of the material is done at moisture content wet of optimum. The methods discussed above are the indirect means of reducing the piping effect however, systematic study are required to know the rate and amount of carbonate solution action under varied hydraulic head.

For the present study a systematic assessment of the foundation conditions has been carried out and based on the conditions suitable remedial measures have been worked out.

The foundation of Tenaho Dam comprise of five different parts, left abutment, valley floor on the left bank, the river channel bed, valley floor on the right bank and right abutment. The left abutment ridge is occupied by volcanic sequence of basaltic rock, which is moderately jointed, and partially weathered. It also contains inter bedded paliosoil layer exposed on the upstream slope face between elevations ± 388m to ±378m .The river floor on the left bank is covered by top soil and riverine alluvium. This unit extends up to an elevation of ±367m along the axis and laterally extends ±373m on upstream and downstream of the dam axis. The river channel is covered by recent alluvial of silty sandy soil which has a thickness of about 13m over the lake sedimentary rocks. The right riverbank forms a wide expanse of alluvium, which extends up to the toe of the right abutment. The depth of this alluvium deposit, near to the river bank, is 15m and it is around 8m in the river section. The right abutment is covered by basaltic rock, which is intruded by the major joint system. The orientation of the joints is mainly across the flow direction.

Based on the surface and subsurface explorations the seepage condition along the abutments may be summarized as; on both the abutments variety of basaltic rocks are present, which are partly weathered and highly jointed. The permeability result indicates that in the top reaches the rocks are highly permeable as on both the abutments there was total water loss during the water pressure tests conducted within this reach. The permeability in the intermediate reach on both the abutments is again high as the Lugeon value is more than 45. Further, on the left abutment within ±388m and ±378m a zone of paliosoil is also present which is highly cracked and very weak in strength. Excessive seepage may take place through this zone as it extends further downstream, however, from the borehole data it has not been traced in the central part of the abutment. In addition to this, the various contacts between different varieties of basalts are the potential areas for seepage. Therefore, it is necessary to provide adequate seepage measures in the abutment section of the dam foundation.

The leakage conditions particularly in the river section can be summarized as; the presence of Alluvium deposit in the river section may pose problems of excessive leakage. The thickness and the lateral extent of this deposit are more towards the right bank where its thickness is about 15 m and it extends some 300 m in upstream direction. Moreover, in the downstream direction it extends over a large area. Thus, there is a possibility of free flowing conditions through this alluvium deposit. In addition to this, the leakage problem associated with the underlying lake sedimentary formations must also be addressed. As this zone comprises of highly jointed silt stone, mud stone and sandstone. The core recovery suggests that the rock mass condition in this sequence is of very poor quality. Thus, provides a possible zone for excessive seepage in the foundation area. Further, the basaltic bed rock is also highly jointed and the RQD value varies in a wide range of 22 –94%. This also suggests a possible seepage through the bedrock foundation. Moreover, limited Geophysical data also suggests the presence of two longitudinal faults around the dam foundation, which extends upto the lake sedimentary sequence. These possible faults are also potential zones of excessive seepage however, the magnitude of such seepage will entirely depends upon the orientation and lateral extent of these faults.

As indicated in the above section, seepage problem is the most significant problem in the foundation of the dam. For the present study by utilizing the surface and subsurface exploration methods, a systematic evaluation of the anticipated seepage conditions through the dam foundation has been carried out. Based on the observations suitable remedial measures to control the seepage/ leakage problems from the foundation are discussed in the following paragraphs.

The water pressure test conducted in the boreholes at abutments indicates very high to medium permeability. Also in the river section there is a need to check the permeability, particularly in the lake sedimentary formation. Therefore the most effective means of checking seepage through the abutments and the river section would be a provision of grout curtains.

Based on the geological and the seepage conditions prevailing on the abutments and the river section the Project Authorities have proposed curtain grouting to a depth of 24 m. In the abutment sections the grout holes will be at an inclination of 30^0 from the vertical, dipping towards north on left abutment and dipping towards south on right abutment. In addition, descending pattern grouting technique is recommended. According to the project Authorities the grout curtain in the abutments would be provided upto an elevation of 379 m. However, the results of the water pressure tests conducted on both the abutments indicates the necessarily of the grout curtain, to be provided below this level. In the present study the depth of curtain grouting in the abutment has been estimated by utilizing Ewert's (1985) relation and a safe depth of curtain grout has been worked out. It may be safe to provide a grout curtain upto a depth of 34 m in the river section. However, this depth may be reduced on the abutment section as the seepage is a direct function of hydraulic head.

On the left abutment within ±388m and ±378m a zone of paliosoil is present which is highly cracked and very weak in strength. Excessive seepage may take place through this zone as it extends further downstream, however from the borehole data it has not been traced in the central part of the abutment. There is a need to provide a proper treatment for this zone. If left untreated the embankment in immediate contact of this zone will always be saturated. This may result into piping or cracking due to variation in saturation of adjoining zones within the embankment. Further, the paliosoil under saturation conditions may further loose its strength, which may result into other related stability and seepage problems. The possible effective treatment method is to provide a diaphragm wall, all along its extent, in the upstream reach. This can be done by excavating 2-3 m of the weak Palosoil and later it may be backfilled with a mixture of clay material with betonite. Other alternative is to constructing a RCC diaphragm wall. Another alternative is to provide a radiating curtain grout along the periphery of the ridge. The grout holes may be placed just above the paliosoil zone and may be inclined at about 30^o from the vertical. Before finalizing this method it is necessary that test grouts should be performed to check weather the paliosoils accept grout or not. However, a detailed economic feasibility study will be required to finalize the most feasible method. These remedial measures will not only control the seepage problem across the ridge but also control the stability problem in the abutment ridges along the weak zones during reservoir draw down conditions.

The soils present in the foundation area have been classified as SP and SW, which are known to be pervious by their nature. Since, these soils forms the part of the foundation therefore these soils will be subjected to the maximum hydraulic pressures after the construction of the dam. As per the design proposal it is proposed to excavate these soils to its full depth below the core of the dam. Also in the shell area it is proposed to excavate these soils to a depth upto which it attains a relative density more than 55%. However, the proposed method of removing the soil from the foundation area may not be effectively reducing the seepage problem. Therefore, in order to reduce the hydraulic pressure at the base of the core and the shell section measures must be adopted to reduce the seepage force. The most economical and effective method of reducing seepage force in the base area of the embankment is by providing upstream impervious blanketing.

In order to improve the physical properties of the foundation rocks consolidation grouting is necessary. It helps in closing all the openings, fishers, joints in the foundation, which not only weakens the foundation but also prevents the seepage through the open joints. Mistry (1983) proposed that consolidation grouting should be extended up to b/4 in the heel portion and b/3 towards the toe, where b is the base width of the dam along river section perpendicular to dam axis. In the river bed section consolidation grouting is done upto a depth of 10-15 m whereas, in abutment section it is done upto a depth of 5-15m. However, at Tendaho Dam as per the design the Project Authorities proposed a consolidation grouting upto a depth of 8 m in the abutment and the river section of the foundation. At Tendaho dam, the foundation rocks are affected by open joints and the overall RQD result of the boreholes indicate that the rocks are of poor strength. Therefore, consolidation grouting is an effective remedial measure not only to increase the strength of the foundation rocks but also to control the seepage through the foundation. It would be advisable to follow the recommendations made by Mistry to have maximum effect of the consolidation grouting in improving the foundation conditions.

For embankment it is important to control the seepage. Two approaches are followed to control the seepage through an embankment dam. The first approach is preventive whereas, the other approach is curative.In preventive approach efforts are made in keeping the water out in so far as possible while in the curative approach a safe outlet is provided to water, which has entered in spite of the preventive measures.

As per the Tendaho Dam design the seepage control through the embankment has been provided through impervious core, upstream transition zone, inclined chimney, filter downstream of core, horizontal filter, rock toe and toe drains. In addition, 2m thick transition zone on the upstream face of clay core has been provided to guard against internal piping.

Downstream shell to act as a drain in zoned section depends on ratio of permeability of the pervious material to impervious material used in the core section. The material suitable for a pervious shell is SW and GW. Thus, the downstream shell will be serving as a free drainage zone and will be effective in reducing the seepage forces developed within the dam body.

The downstream toe of an embankment dam is the most critical region in respect of seepage instability. The entire seepage tends to concentrate around downstream toe, if internal drainage is not provided. The soil mass in this region is subjected to excessive seepage forces. This may cause heaving and sloughing of the toe if not properly protected. Rock toes, drainage blanket and filter drains are provided on the downstream of the dam to give a safe outlet to the seepage. At Tendaho Dam site there is no scarcity of suitable rock for the construction of the rock toe. As per the design a 12 m high rock toe has already been proposed by the Project Authorities.

Toe drains are provided along the downstream toe of a dam in conjunction with a rock toe or horizontal drainage blanket to drain off the seepage water emerging from the body. Toe drains are also useful in collecting foundation seepage. For dams of low to moderate height horizontal drainage blankets are used to drain the embankment as well as the downstream portion of foundation. The length of horizontal drainage depends on the flow- net; however, U.S.B.R recommended that the length of the blanket be equal to three times the height of the dam. Moreover, it must be sufficient cross-section to convey the maximum quantity of seepage estimated to come through the dam section since it is supplemented by the chimney drains on the upstream side and with toe drains on the downstream side.

The foundation material at Tendaho Dam site has been classified as SP and SW, therefore there is a fair possibility of migration of fine particles from the foundation into the horizontal drainage. To protect the migration of the particles, provision of a filter in between the dam base and the horizontal drainage is desirable.

Chimney drains are the most important seepage controlling methods especially in the zoned embankment. The water from a chimney drain has to be taken out through horizontal drains and a toe drain. The chimney drain has to be protected on all sides by filter layers. The advantage of providing chimney drains in the earth dam is to keep the downstream portion free from seepage when the reservoirs is full. The Chimney drains also helps in reducing the drawdown and construction pore pressure. For Tendaho Dam Chimney drains are already provided as per the design of the dam by the Project Authorities.

8.2 Recommendations

For Tendaho Dam most of the construction material is available in good quantity within the economic distance from the dam site. However, the quality of the construction material for the dam embankment needs to be improved by adopting following measures.

The core material, proposed to be used, for the construction is lack of sufficient coarse fraction. Thus there is a need of blending. The blending proportion for coarse material may be kept in a range of 30-40%. However, to work out the exact proportion of the coarse material further testing of the blended material would be required.

The core and the shell material contain carbonate material. The presence of carbonate in the shell and core material may result into solution action by the impounded reservoir water. The blending of core material with a coarse fraction and inclusion of stone chips upto 10 – 20% may improve the erosion resistance of the core material. Further, for the shell material the piping effect due to solution action of carbonate may be reduced if the permeability and the flexibility of the material are reduced. This may be achieved if compaction of the material is done at moisture content wet of optimum.

The suitable quarry site identified during the present study for the riprap material is about 14 km upstream from the proposed dam site along the highway to Addis Ababa. This site may be utilized for the dumped riprap material.

On the left abutment a paliosoil zone is present which has to be treated so that no excessive seepage takes place through this zone. The possible effective treatment method is to provide a diaphragm wall, backfilling with a mixture of clay material with betonite or to provide a radiating curtain grout along the periphery of the ridge.

There is a need to provide a curtain grouting upto a depth of 34 m in the river section. However, this depth may be reduced on the abutment section as the seepage is a direct function of hydraulic head.

In order to improve the physical properties of the foundation rocks consolidation grouting is necessary. Consolidation grouting should be extended up to b/4 in the heel portion and b/3 towards the toe of the dam.

The fine filter to be provided between the core and the coarse filter on the downstream shell side should be extended down to the base of the core.

In order to reduce the hydraulic pressure at the base of the core and the shell section upstream impervious blanketing may be provided, this may help to reduce the chances of seepage and erosion in the base area of the dam.

Thus, finally it may be concluded that the dam foundation of proposed Tendaho Dam has some seepage problems, which needs proper attention. By adopting suitable measures, as highlighted through the present study the seepage problem in the foundation area may be taken care off. There are certain problems in the construction material to be used in the embankment. These construction materials must be treated as per the recommendations made through the present study.

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Annex –2
Procedure for compaction test

Sample preparation

Take about 18kg air dried soil sample for 100cc mould(40 kg for 2250 cc mould) . Sieve the soil through 20mm and 4.75mm ASTM. Sieves and calculate the ratio of fraction passing 20mm ASTM sieve and retained on 4.75 ASTM. sieve. Use 100mm dia mould if percentage of soil retained on 4.75mm ASTM.,sieve is less than 20, and 150mm dia mould if percentage of soil retained on 4.75mm ASTM . sieve. Mix the soil retained on 4.75 ASTM. sieve and passing through 4.75mm I.S. sieve thoroughly and uniformly in its original ratio.

Testing of the specimen

Take about 2.5kg of the soil for 100mm dia mould (or 5.6 kg for 150mm dia mould) for light compaction. For heavy compaction take about 2.8kg of the soil for 100mm dia mould (6 kg for 150 mm dia mould) . Add water to the soil specimen to bring the moisture content to about 4-5 percent in sandy soiln and 8-10 percent in clayey soil Mix the water thoroughly. 18-20 hours for clayey soils. Prepare similar six to seven equal parts(each of about 2.5 kg) from the oven dried (18kg) soil sample at different moisture content.

For light compaction

Compact one part of the wet soil in three equal layers using the Hammer of mass 2.5 kg and free fall 310mm with 25 evenly distributed bows in each layer for 100 mm diameter mould and for 150mm diameter mould free fall of 56 blows.

After eject the soil out of the mould, take a representative sample from centre of the compacted specimen and keep it in an oven for its water content determination. Repeat the procedure four to five times using a fresh soil specimen after adding a higher water content then the preceding one till there is either a decrease or no change in the mass of the wet compacted soil in the mould.

Observation and calculation

Determination	Trial -1	Trial -2	Trial -3	Trial -4	Trial –5
Weight of mould + compacted soil(g)					
Weight of compacted soil (g)					
Wet density					
Container No					
Weight of Container + wet soil (g)					
Weight of Container + dry soil (g)					
Weight of water					
Weight of Container					
Weight dry soil (g)					
Weight content (w)					
Dry density					

Maximum dry density =
Optimum moisture content =