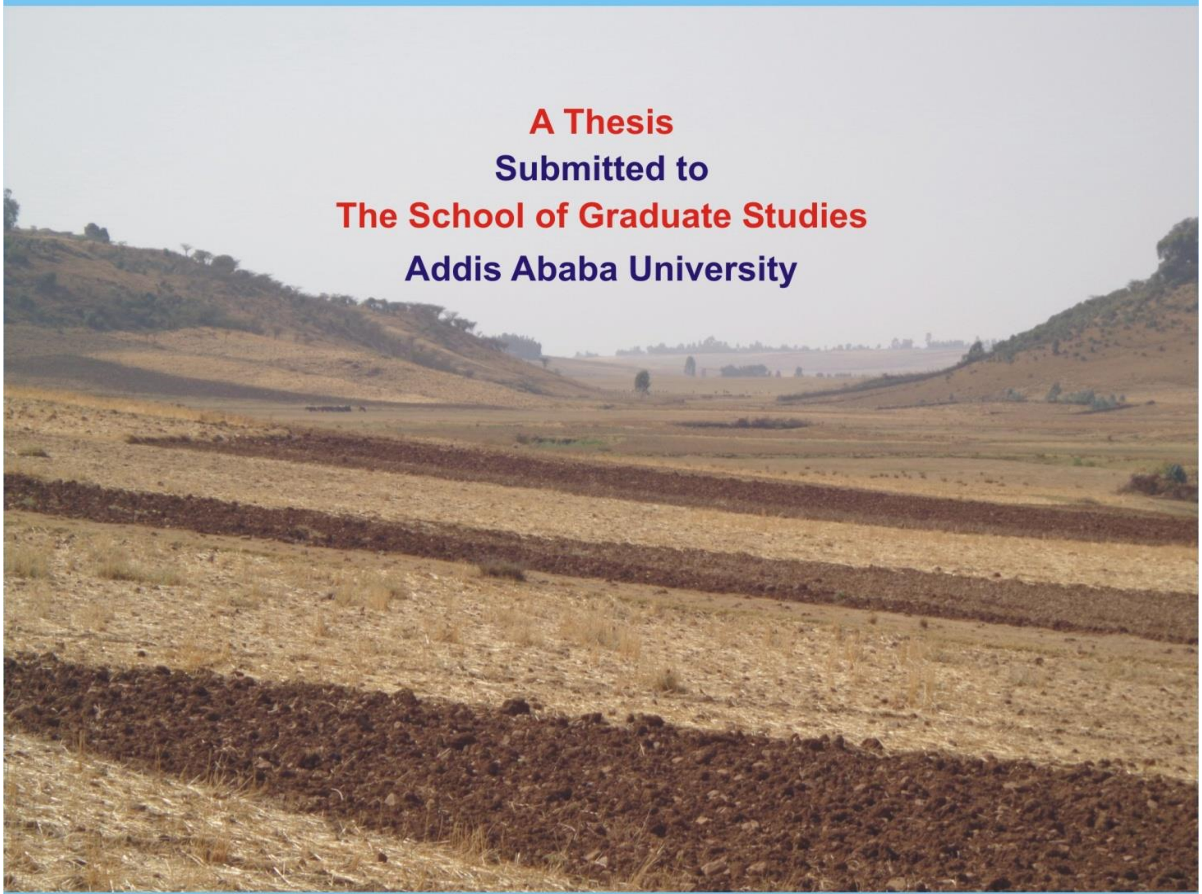
ADDIS ABABA UNIVERSITY
School of Graduate Studies
College of Natural Science

Characterization and Suitability Analysis of Embankment Material for Dam on Berga River, Central 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

Tutan Negash

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DEPARTMRNT OF EARTH SCIENCE

CHARACTERIZATION AND SUITABILITY ANALYSIS OF EMBANKMENT MATERIAL FOR DAM ON BERGA RIVER, CENTRAL ETHIOPIA

By
TUTAN NEGASH
ENGINEERING GEOLOGY PROGRAMM

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DECLARATION

I hereby declare that the thesis entitled "CHARACTERIZATION AND SUITABILITY ANALYSIS OF EMBANKMENT MATERIAL FOR DAM ON BERGA RIVER, CENTRAL ETHIOPIA" has been carried out by me under the supervision of Dr. Tarun Kumar Raghuvanshi, School of Earth sciences, Addis Ababa University during the year 2013 as part of Master of Science Program in Engineering Geology. I further declare that this work has not been submitted to any other University or institution for the award of any degree or diploma and all sources of materials used for the thesis have duly acknowledged.

TUTAN NEGASH

Signature______________________________

Place and date of submission: School of Graduate Studies, Addis Ababa University

May 2013

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*****

Abstract

Material quality and suitability is one of the main criteria for the success of embankment dams. Seepage through the embankment, foundation, and abutments must be controlled and collected to prevent excessive uplift pressures, piping, sloughing, dissolution and erosion of material. The embankment, foundation, and abutments must also be stable against slumping, sliding and sloughing during construction, reservoir operation and during events such as earthquake and floods. The present study was conducted on a proposed Berga earth dam which is located on river Berga in central Ethiopia, located approximately 11.5 km north of Holeta town. The main objective of the present study was to characterize and to assess the suitability of the proposed construction material for various zones of the embankment dam. Besides, it was also proposed to evaluate and characterize the dam foundation for its suitability for the dam.

For foundation characterization data pertaining to Rock mass classification (RMR) was collected from the left abutment. Further, stability analysis was made for both the abutments. The left abutment was found to be stable whereas, right abutment was analyzed for rotational mode of failure. The results revealed that right abutment will be stable only for static dry condition and for anticipated worst conditions the abutment slope would be unstable. Further, the soil samples were collected from the dam site and were analyzed. From the investigations conducted during present study it was found that the dam site possesses weakness in terms of seepage potential and abutment slope stability condition. However, in terms of bearing capacity, both the abutments have high capacity as they are composed of very strong basalt rock.

In order to assess the general suitability of the construction material for various zones of the embankment from the proposed borrow areas samples were collected and tested for grain size distribution, Atterburg limits, swelling potential, activity and permeability. Besides, data on soil properties from previous study was also reviewed and analyzed. Besides, mineralogical analysis was also made for core samples. In order to assess the general suitability of the filter material various filter criteria were applied. In general the proposed construction material for the dam is slightly plastic in nature. Thus, it is anticipated that this may result in to high compressibility which may lead to instability. Also, the proposed filter materials contain more fine materials than the desired percentage.

Also, the initial design was prepared by using Taylor’s stability numbers. Later, the stability of the embankment slopes was analyzed by using SLOPE/W and SARC software. The stability analysis results revealed that the dam design will be stable when there is no water in the reservoir. However, the factor of safety will decrease under anticipated adverse conditions which may possibly result in to the failure of the slopes.

Finally, in general it is found that proposed dam site is feasible with minimum engineering geological problems and construction material is available within the economic distance though it requires certain treatment for its improvement. Based on the findings of the present study certain remedial measures are forwarded.

Chapter 1 Introduction

1.1 General

Engineering geology is a discipline that conducts study for the safety of different civil engineering structures and one of these are embankment dams (Fig. 1.1) (Mukerjee, 1995). Depending on the predominant fill material used, the two principal types of embankment dams are earth dams and rock-fill dams. The structural safety of an embankment dam is dependent primarily on the absence of excessive deformations under all conditions of environment and operation, the ability to safely pass flood flows, and the control of seepage to prevent migration of materials and thus prohibit adverse effects on stability (USACE, 1982).

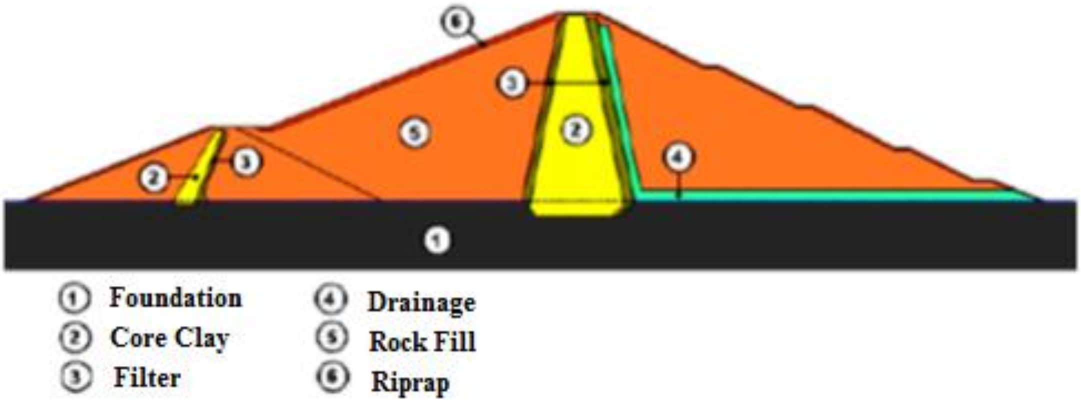
To properly evaluate the stability of an embankment dam, the following areas should be reviewed; Embankment zoning and cross section, seepage control measures and records, predicted or recorded deformation, erosion control measures and structural stability analyses (USACE, 1982).

For zoned embankments, the zoning geometry and properties of the materials placed in the zones should be reviewed to determine: (i) the structural design, and (ii) the types of internal features such as; chimney drains, blanket drains, toe drains, etc., that are proposed or were used to provide for and maintain the embankment stability. Embankment zoning is also established for economic reasons according to the availability of materials. The embankment zoning should provide an adequate impervious zone, transition zones between the core and the shells, and seepage control zones (USBR, 1987).

Desirable characteristics that these zones should have or provide are; the width of the core at the base of the cut-off should be equal to, or greater than, 25 % of the maximum difference between the maximum reservoir and minimum tail water elevations (USACE, 1952).

The coefficient of permeability of the core material should preferably be 10^{-4} cm/sec or less. More permeable core material may be acceptable if seepage is still adequately controlled and appropriate factors of safety are still met (USBR, 1987).

The transition zones should be sufficiently wide to ensure that they are continuous and constructible with a minimum of contamination at the contact (USBR, 1987; Terzaghi, 1967). The range of gradation of the transition zones should be limited to avoid segregation of materials during placement (USACE, 1982). Zoning of an embankment that places the more pervious material on each side of the core zone is preferable. This placement improves the stability of the embankment during rapid drawdown conditions and keeps the downstream slope drained for greater effective weight (USACE, 1952).



(*Source: Seyed et al., 2012*)

Fig 1.1 Cross Section of Zoned Embankment Dam

Most embankments are designed to utilize the economically available on-site materials for the bulk of construction. Special zones such as; filters, drains and riprap, may come from off-site sources. Soil materials used in embankment dams commonly are obtained by mass production from local borrow pits, and from required excavations where suitable (USSD, 2011).

In order to determine the suitability of these materials characterization based on different engineering criteria are applied. These include the filter criteria which determine that the drainage filters must be designed in such a way that neither the embankment nor the foundation material can penetrate and clog the filters (Garg, 2005).

Failure to apply the criteria will result in different kind of failures of the embankment material which includes slogging, piping, the development of pore water pressure and in the worst case the total failure of the dam. Detailed and systematic investigation must be carried out in order to avoid all these problems (Garg, 2005).

Berga earth dam is proposed to be constructed across the river Berga, a tributary of Awash River. It is located nearly 11.5 km north of Holeta town. The Berga dam could be earth dam or a rock fill dam. Up to 40 m high dam with 250 m crest length could be constructed at Berga dam site. The Berga proposed dam site is an ideal site to build a dam, it has flat reservoir and a good topographic closure at the dam site (Yohannes Belete et al., 2009).

The present study is conducted on a proposed dam site in central Ethiopia by giving especial emphasis on characterization of selected material for suitability of construction of dam on Berga River. It is necessary to know the general suitability of construction material for the safety and stability of the dam. Thus, the present study will provide characterization of material suitable for various zones of the dam.

In countries like Ethiopia, where agriculture is the main contributor to the GDP, there must be large production of crops in order to feed its people and also to sale excess production. To do this, there must be agricultural activities throughout the year. One of the possible ways to carry out agricultural activities is to build dams for irrigation and water supply.

This research was mainly conducted by considering the above mentioned point and by selecting suitable area to perform study for suitability of available material for the construction of the dam which is recommended for the use of either irrigation and/or water supply with a command area of about 3000 hectares (Yohannes Belete et al., 2009).

In order to conduct the present study systematic methodology has been followed. Relevant Literatures on the subject matter have been reviewed; which includes previous reports on the proposed dam and the similar studies on other dams, various standards for the characterization of construction material for dams etc. Primary data was collected from the field for further analysis of the materials and to characterize the abutments of the proposed dam site. GPS reading and base maps were used during field work to gather all the necessary data for the proposed research work.

This study is limited on characterization of material for various zones of the dam and their stability on different site conditions. The present study can be continued on further evaluation of the foundation area by conducting water pressure tests so that the dam can be constructed on the safe foundation without any problems of seepage, piping and blowouts.

1.2 Problem statement

The Geotechnical investigation, conducted by Geological Survey of Ethiopia (Yohannes Belete et al., 2009), acquired during the study of the proposed dam site provides only baseline information. Therefore, to come up with detailed and tangible outcome assisting future actions, it would be necessary to conduct further study especially on the proposed construction materials for the dam.

This dam site can be used for water supply and / or for irrigation purpose. In order to do that there should be study on the ability of the material to minimize seepage problems and also piping in the different parts of the dam. Therefore, the research problem evolved for the present study was entitled; “Characterization and Suitability Analysis of Embankment Material for Dam on Berga River, Central Ethiopia.**”**

1.3 Location and Accessibility

The study area is located in the northwestern plateau of Ethiopia. Geographically it is bounded between coordinates of 421000-443000m E and 996000-1020000m N of UTM 37 Zone. The study area can be accessed through Addis Ababa-Holeta-Ambo asphalt road and later by Holeta – Mugher asphalt road (Fig. 1.2 and Fig.1.3).

1.4 Objectives

The following objectives were formulated for the present study;

General objective

The general objective of the present research was to investigate the suitability of the construction material for the embankment dam.

Specific objectives

The specific objectives are:

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

(ii) To characterize construction material for its engineering behavior under different site conditions.

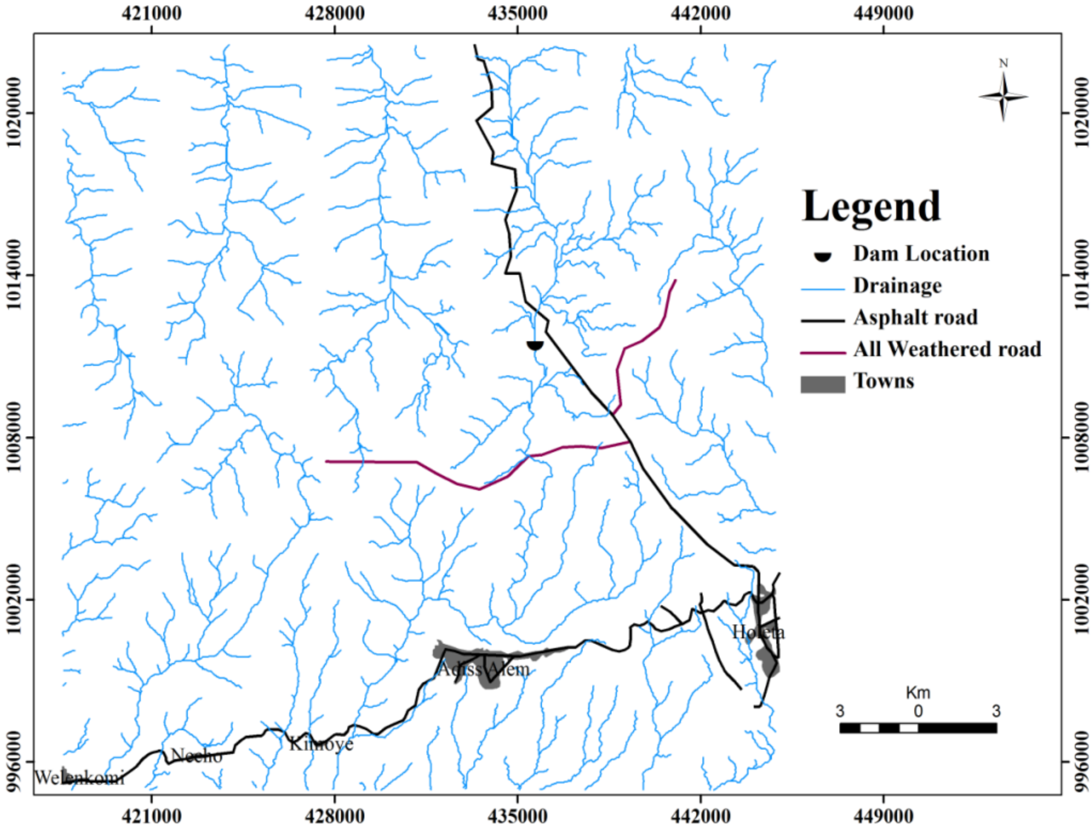


Fig. 1.2 Location of the study area

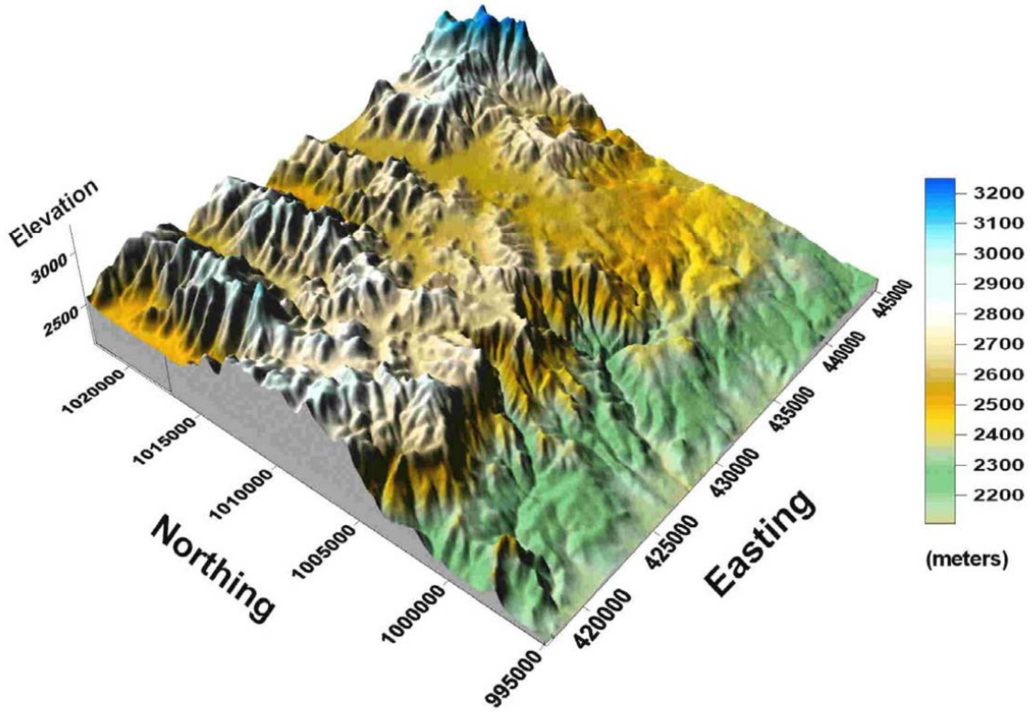


Fig. 1.3 Physiography of the study area

(iii) To develop suitable remedial measures to improve unsuitable material for core and filter.

(iv) Determine the slope stability of the embankment during and after construction.

(v) Based on the research findings to forward recommendations on general suitability of construction material for the proposed dam project.

1.5 Methodology

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

(i) Literature review to have an overview of geological, geomorphologic, hydro-geological and engineering geological condition of the dam site and the surrounding areas.
(ii) Geological Mapping of the dam site.
(iii) Collection of samples from dam foundation area, borrow areas and quarry sites for laboratory testing and analysis to determine various index properties.
(iv) Determination of the suitability of the construction material for shell, core, filters and transition zones based on standard specifications.
(v) Embankment Slope Stability analysis
(vi) Thesis Writing
(vi) Presentation and defense

1.6 Importance of the study

Embankment dams are one of the civil engineering structures that are constructed to be used for different purposes which includes; irrigation, power supply etc. During their construction, proper material must be identified at economic distance with sufficient amount. In comparison of concrete dams, embankment dams can be constructed in less suitable foundation which makes them preferable for construction in areas prone to devastating natural phenomena like earth quakes (USACE, 1982).

Even if all these points make embankment dams easier to build, they are exposed to different factors that lead to their failure. One of these factors is piping as a result of poor construction or improper material. Piping will lead a loss of large amount of water and in worst case total failure of the dam. The other factor is the development of pore water pressure inside the main dam and slogging around the toe which will contribute for failure (USBR, 1983).

In light of the above mentioned points it is mandatory to identify and use suitable material to construct the various zones in an embankment dam. Thus, through the present study attempt is being made to characterize the construction material as per the standard specifications. Besides, the general suitability of the construction material was assessed. The present study may provide very vital information on the suitability of the construction material which may be adopted by the project authority for safe design of the embankment. Further, the present study may also provide a useful guide on general methodology to be followed for material characterization for embankment dam.

1.7 Limitation and the Scope of the study

This study is limited on characterization of materials for different parts of the zoned dam and analysis of the stability of the slope of the dam for different site conditions. The study can be continued on further evaluation of foundation conditions by conducting water pressure tests so that the dam can be constructed on the safe foundation without any possible problems of seepage, piping and blowouts.

All efforts were made to conduct the present study in a systematic manner with actual data and information gathered from both secondary and primary sources. However, all these efforts were made under the constraints of resources, finance and time.

1.8 Scheme of Presentation

The scheme of presented of the research study is as follows;

Chapter 1: This chapter gives the general definition of embankment dams by explaining the different parts of the zoned dam which includes the core, filter and shell. The general and specific objectives with the methodology followed are listed. The importance and the limitation of the study are also presented.

Chapter 2: In this chapter, the literatures that were reviewed to understand the general geological and structural setup of the study area and the methodology that was followed are presented. The different parts of embankment dams are described with the criteria that are followed to have a safe dam structure. The geomorphological setup is also presented.

Chapter 3: This chapter describes the detailed regional and local geology by listing the different lithologies that are available in the vicinity. The detailed hydro-geological setup of the area with different test results is also presented. The seismicity of the study area is presented with reference to the Seismic Hazard map of Ethiopia. Besides, the geophysical investigation that was carried out at the dam site, prior to the present study, is also presented with the final interpretation.

Chapter 4: In this chapter, the engineering geological investigation of the dam site is presented. The rock mass classification was carried out following RMR classification system, the data collected is presented with tables with the interpretation. The deformation of the rock mass of the left abutment is presented which is determined by using Bieniawiski's (1979) empirical relation. Hoek and Brown Failure Criteria was also used to determine the failure potential of the left abutment. The permeability of the dam site is determined by using the soil mass classification which is also presented in this chapter. Slope analysis of the left abutment is also presented in the chapter. At last the engineering geological characterization of both the soil and rocks at the dam site are presented. Also, the regional and dam site engineering geological maps are presented.

Chapter 5: This chapter presents the detailed investigation of the embankment material of the borrow areas. It presents the analysis of the material by using different standards and filter criteria. The results of the different laboratory tests and field investigations of the borrow areas are presented with plates, tables and figures. The seepage control mechanism that can be used in the dam structure is also presented.

Chapter 6: The stability analysis of the embankment is presented in this chapter. The initial design of the dam is presented. By considering different site conditions the stability of the embankment material is presented. SLOPE/W and SARC software were used to determine the critical failure surface of both upstream and downstream slopes of the embankment by calculating the factor of safety for anticipated conditions.

Chapter 7: Presents the summary and conclusion of the works performed and the recommendations made through the present study.

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Chapter 2 Literature Review

2.0 Preamble

Before the initiation of the present study a systematic literature review was carried out. Attempt was made to get a general conceptual framework on embankments dams. For this various literatures were reviewed to understand, types of embankment dams, their design considerations, characterization of construction material, possible causes responsible for failures, various methods of analyzing its stability, review of various standards for design etc. Thus, based on the systematic literature review a feasible methodology was evolved/ followed for the present study. A summary of literature review relevant to the present study is presented in the following paragraphs.

2.1 General

Dams are structures that are built on rivers to hinder the flow of water so that it can be used for different purposes like irrigation, water supply and power generation. In Ethiopia there are many dams that are built on different rivers like Awash, Tekeze, Omo, Abay and others which are either concrete or embankment dams. Table 2.1 shows some of the embankment dams that are constructed or under construction in Ethiopia (Ali Aman, 2008).

Table 2.1 Summary of construction material used in few major embankment dams in Ethiopia

No.	Name	Type	Height	Construction material			Year of construction	Purpose
				Core	Filter	Shell		
1.	Tendaho	Earth dam	45 m	CH clay blended with sand	Natural sand	Alluvial gravelly material	Currently under construction	Irrigation
2.	Dire	Zoned Earth dam	43 m	Clay (CH)	Crushed sand	Alluvial	1998-2000	Water supply
3.	Midmar	Zoned Earth dam	32 m	CL,ML, MH and CH	Natural sand	Gravely material (GC)	1993	Water supply

(Source: Ali Aman, 2008)

In order to prevent any problems related to the safety of embankment dams, detailed study must be conducted in every aspect. Embankment dam studies must include, water tightness of the reservoir, runoff characteristics of the river, availability of sufficient material near the dam site, history of seismic activity in and around the dam site and quality of the water for different uses (USACE, 1982).

At around 30 km from the proposed dam site, the Gefersa dam is located. This dam is constructed for water supply (Plate 2.1). It is one of the main dams that supply drinking water to Addis Ababa.

2.2 Previous works

The following literatures and reports were reviewed to have a thorough conceptual framework about the dams and to develop the general methodology for the present study. Besides, the literature review also helped to understand the area for the present proposed study;

Plate 2.1 Gefersa dam - one of the main dams that supply drinking water to Addis Ababa

Yohannes Belete et al. (2009) carried out investigation to collect baseline data on the suitability of project area for different civil construction purposes including identification and mapping of sites suitable for construction of multipurpose dams as well as occurrences of quality of raw construction material. To achieve these objectives, engineering geological mapping and electrical resistivity method were implemented. The results were compiled on 1:50,000 scale map and geo-electric sections were prepared. Further, one dam site (Berga dam) with potential catchment and command area is proposed to be used either for irrigation or for water supply. Inventory of existing and proposed quarry sites has been done. The result of this investigation requires feasibility study to clearly determine the dam's suitability for proposed purposes.

Assiged Getahun (2007) in his report presented the geological map of Addis Ababa city. The area comprises of volcanic rocks. The major structures in the area are lineaments, which trend in NW and NE directions, normal faults trending in NW direction and sedimentary bedding. The area has economic potential i.e. occurrence of rocks which can be used for cement production and construction. The methodology used in this investigation includes: using base map on 1:50,000 scale and satellite imageries, traverses were selected along which representative samples were collected. GPS readings were also taken and data were collected to produce preliminary geological map of the area.

Zanettin et al. (1977) divided the tertiary volcanic of central Ethiopia in three stages of volcanism and tectonism: transitional (thiolettic) flood basalt followed by alkali rhyolite and ending with alkali basalt.Silicic and basaltic fissural magmas erupted from the end of the Oligocene until the Upper-middle Miocene (25-15 Ma). The rest of the basalts are the result of this silicic and basaltic magma. Therefore, after the eruption of the Middle basalt there was uplifting and local tectonics in the upper parts of the study area. This was also followed by the formation of a local basin and deposition of Tertiary sediment. After the eruption of the Upper basalt there may also have been another uplifting with accompanying tectonics which formed a local basin in the central eastern parts of the study area. This activity was followed by the deposition of Tertiary sediments. The deposition of the later Tertiary sediment was followed by another cyclical fissural eruption which began and ended with basaltic and silicic magmas, respectively. This eruption produced the ignimbrites and basalts. After this stage the fissural volcanism may have been radically changed to a central type and gave the basalts whose centre could be Mount Megezez. Another small centre is found around Maset. After a certain time the young Quaternary ignimbrites erupted from centres probably located around Addis Ababa, and were underlain non-conformably by younger basalts.

Kebede Tsehayu et al. (1990) conducted study for the preparation of engineering geological map of Addis Ababa area. According to this work, basalt, rhyolite and trachyte of different ages are found in the area and a number of faults most of which have sub parallel trend with the main Ethiopian rift were identified. This study is multipurpose since it deals with different aspects of engineering geology. The soils and rocks are classified as alluvial, alluvial fun, residual, colluvial and lacustrine. There are four rock mass with different strength units; Very high rock mass strength for basalts, Very high rock mass strength for trachy, basalt, rhyolite and trachyte, medium rock mass strength for ignimbrite and low rock mass strength for tuff and agglomerate. Ignimbrite and basalts are widely used for construction material in Addis Ababa. Residual soil in Gulele area is raw material for brick making. Black cotton soil (lacustrine in origin) around Bole, Lideta and Mekanissa is found to be problematic for light houses due to swell and shrink property. In order to know the thickness of black cotton soil geophysical and drilling were carried out. From the boreholes information the thickness of black cotton soil vary from 2-10m in Bole area. According to geophysical investigation, black cotton soils thickness reaches up to 10 m around Lideta (Gebriel Meda) and average thickness of 5 m for Beklo Bet North of Nifas Silk. It was recommended that more samples and tests needs to be taken and performed in order to know detailed geophysical behaviors and over burden thickness of soils so that isopach maps can be prepared for designing and engineering works.

Mulatu Tumoro (2010) conducted investigation to characterize the available construction material with especial consideration to core and filter materials in Dendo dam. In order to achieve the objectives of the research work systematic methodology was adopted. This includes; literature review from both published and unpublished reports on construction material used for embankment dams and their case studies for dam failure due to improper use of embankment material was assessed. Based on the laboratory test results of soil samples generated during the study; the identified core material in Dendo borrow area was classified as CH, MI or OI, CI, MH or OH as per USCS. Proximity to the dam site was the main advantages of using this material for the core of the dam. When grain size distribution of Dendo dam core material was concerned addition of coarser material to the fine material is necessary as it locks the propagation of crack of different size through the core of the dam depending on the size of the coarser material.

Ali Aman (2008) has performed multi- dimensional characterization and suitability analysis of embankment material for Kesem dam. In order to achieve the objectives of the study systematic methodology was followed which include; Review of literature and secondary data analysis, field investigation, laboratory analysis and interpretations of the test results. During the study a number of field and laboratory tests were made to identify the means by which the engineering properties of these soils could be improved. Further, attempts were made during the study to perform blending of naturally available clay and gravelly material, which were available at reasonable and economic distance, as an option to improve the quality of the core material. For this various tests were conducted on normal clay and on different blending proportions of the normal clay (from Iselo clay borrow area) and the gravelly material. These tests include; classification, Procter compaction, consolidation, free swell, volumetric shrinkage, direct shear, and triaxial (CU) tests.

Agerie Genetu (2007) conducted study for engineering geological appraisal of dam foundation for Gumara dam. In order to achieve the objectives the following systematic methodology was adopted; Preparation of the base map of the study area from existing topographical maps, Literature review to have an overview of geological, geomorphologic, hydro-geological and engineering geological condition of the dam site and the surrounding areas, geological mapping of the dam site, sub-surface exploration through borehole logs, collection of soil and rock samples from dam foundation area for laboratory testing and analysis to determine various index properties. Based on the above methodology suitable remedial measures to improve the foundation condition has been suggested. The soil testing conducted during the study reveals that the soils present at shallow depth of up to 2.5m at dam foundation area are classified as ‘Inorganic clays of high plasticity’ (CH), inorganic silt of low plasticity (ML) and inorganic clay of high plasticity (MH). Further, from the secondary data analysis, all the soil samples falls below 'A-line' on the plasticity chart. Also, the samples show organic content in a range of 14.26 to 23%. Thus, these soils falls in organic silts of low plasticity 'OL' and organic clays of medium to high plasticity 'OH' groups. The rock mass exposed on the left abutment falls into Class-II and Class-III as per the Bieniawiski’s Rock mass rating system. Thus the rock mass is of Fair to good quality. The overall average cohesion (C) is 3.1 MPa and angle of friction for the rock mass is 36.6°, as determined from Hoek and Brown failure criteria. The left abutment slope is not kinametically unstable, as it does not satisfy the condition set by Markland. Thus, it may safely be presumed that the left abutment slope is stable for present geometric configuration.

Minwuyelet Mengistie (2009) conducted study for Seepage assessment and remedial measures for Gilgel Gibe III hydroelectric power project. The following systematic methodologies were preferred to adopt; literature review to have an overview of geological, geomorphologic, hydrological, and engineering geological conditions of the dam site and its surrounding, preparation of base map of the study area from existing topographical maps using Arc GIS, geological mapping of the foundation site, sub surface exploration through borehole logs, assessment of permeability of the dam foundation rocks through existing water pressure test data (using packer test), and making use of software for analysis of structural and seepage assessment.In this study it was concluded that the permeability condition in the foundation area is not suitable as far as seepage potential is concerned. From the water pressure tests a general increase in the amount of water take is observed from the abutments down to the valley floor. The permeable horizons in the dam foundation area show two depth ranges with lugeon values medium to very high absorption permeability conditions. In between these two sections there is impermeable section which is about 50m in thickness. Therefore, as a remedial measure a curtain grout for the foundation was suggested to improve the permeability condition in the foundation area. It was also recommended that grout holes have to be drilled to a depth 150-180m where these can cross the upper and lower pervious zones.

Nigatu Fikadu (2006) performed engineering geological studies for suitability of construction material and foundation condition evaluation on Tendaho dam. In this study, the following systematic methodologies were used; preparation of the base map of the study area, detailed literature review, geological mapping of the dam site and the borrow areas, collection of engineering rock mass classification data to work out the strength and deformability characteristic of the rock mass present at the dam foundation area and 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. In this study, the core material was classified as CH, MH, and CL. The samples of the area have plastic index values greater than 20 but the blended material has about 17. In general it was recommended that the blending proportion must be kept between 30 – 40% of coarse material. The filter material was classified as SW and it has a maximum dry density of 2.24 gm/cc and optimum moisture content of 8.0%. Riprap and shell materials were also identified and studied in detail. In this study it was found that seepage is the main problem in the foundation area and different seepage control mechanism were suggested as solution.

2.3 Embankment Material

2.3.1 Overview of Embankment material

Embankment dams are one of the major types of dam and they can be built with materials with or without concrete (Fig. 2.1).While most soils can be used for earth-fill construction as long as they are insoluble and substantially inorganic, typical rock flours and clays with liquid limits above 80 should generally be avoided. If a fine-grained soil can be brought readily within the range of water contents suitable for compaction and for operation of construction equipment, it can be used for embankment construction. Some slow-drying impervious soils may be unusable as embankment fill because of excessive moisture, and the reduction of moisture content would be impracticable in some climatic areas because of anticipated rainfall during construction.

Sound rock is ideal for compacted rock-fill, and some weathered or weak rocks may be suitable, including sandstones and cemented shales (but not clay shales). Rocks that break down to fine sizes during excavation, placement, or compaction are unsuitable as rock-fill, and such materials should be treated as soils. Processing by passing rock-fill materials over a grizzly may be required to remove excess fine sizes or oversize material (Gedeon, 2004).

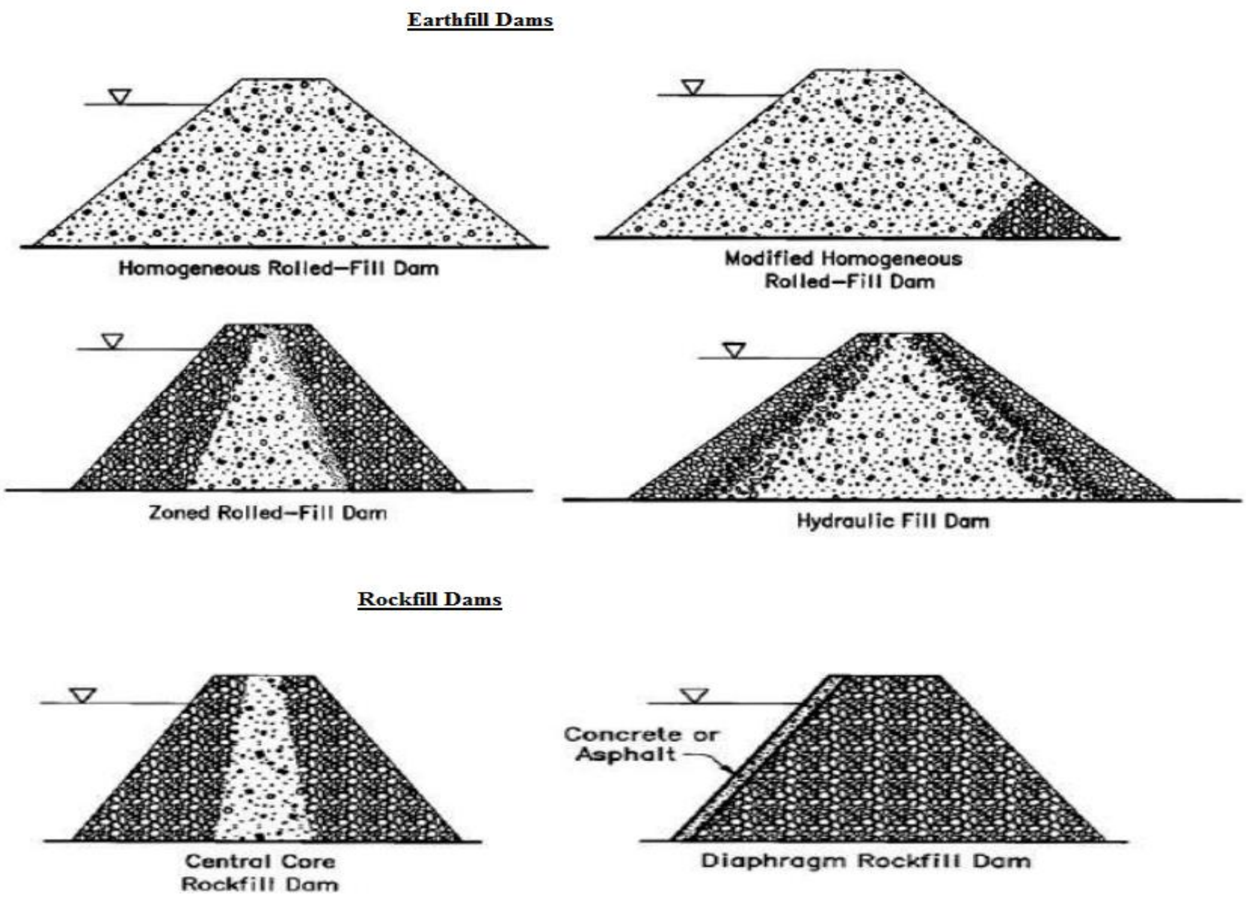

Fig. 2.1 The Major Types of Embankment Dams

2.3.2 Core Material

The core is an impervious barrier built within an embankment dam. In designing a core three aspects has to be considered; Selection of suitable material, determination of the core thickness and determination of core position within dam section.

a) Selection of Core Material

The first essential requirement for the core material is the availability of suitable material at an economic distance from 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.

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 materials cannot retain open cracks but such materials 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. In general the core material should have a high resistance to erosion (Arulanandan, 1983).

b) Core Thickness

The minimum core thickness is governed by the safety against piping or leakage. The minimum thickness for tolerable seepage loses and for proper compaction, the core thickness should thus depend on the type of material available and the design of transition or filter zones. If the available materials have a high erosion resistance as well as good flexibility, smaller thickness of the core can be used.

For a given type of material, the thickness can be kept less if the filter or transition zone material fully meets the specifications and of adequate thickness. Larger thickness has to be used in seismic areas where there are greater chances of cracking. In general, impervious zones, whether inclined or central, should have sufficient thickness to control through seepage, permit efficient placement with normal moving and compacting equipment, and minimize effect of differential settlement and possible cracking (Gedeon, 2004).

According to Bharat Singh (1981) a guide to core thickness may be obtained as;

$$\frac{L}{H} = \frac{\gamma_w . t}{2\,T1}$$

Where, ‘L’ is core thickness, ‘H’ is head, ‘γw’ unit weight of water and ‘T_1 ‘ is the limiting shear strength, may be taken as 3 to 5 times the critical value at which erosion begins.

c) Location of Core

The core can be located in one of the following three positions: (i) Central, (ii) moderately slanting, (iii) Slanting (Fig. 2.2). The central location need not be exactly symmetrical. When the downstream face of the core has an upstream slant of 0.5 H: 1V or more, the core may be considered as moderately slanting. A truly slanting core would be such that the downstream zone has a self-supporting slope, i.e. 1.25 H: 1V or so, such a core is almost always associated with a rock fill dam in which the main mass of rock fill downstream of the core can be placed independently by dumping or in thick layers and the placement of filter zones, core and upstream pervious zone take up later. Since impervious materials are generally weaker than the more pervious and less cohesive soils used in other zones, their location in a central core bordered by stronger material permits steeper embankment slopes than would be possible with an upstream sloping impervious zone. An inclined core near the upstream face may permit construction of pervious downstream zones during wet weather with later construction of the sloping impervious zone during dry weather. This location often ensures a better seepage pattern within the downstream portion of the embankment and permits a steeper downstream slope than would a central core (Gedeon, 2004).

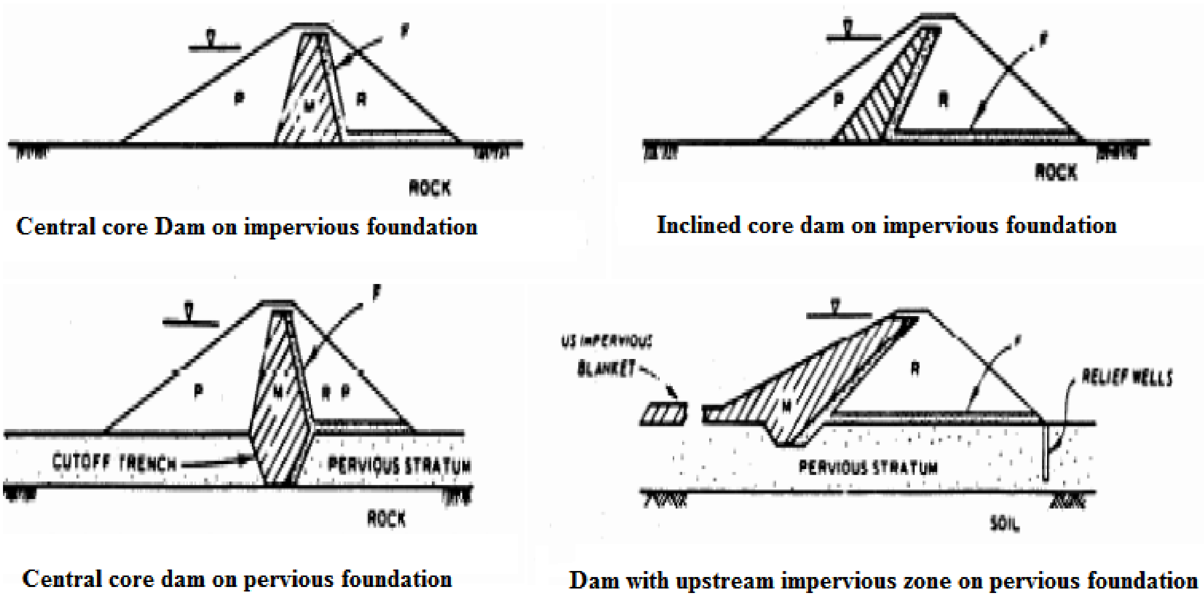

Fig. 2.2 Different locations of core

2.3.3 Filter

The filter design for the drainage layers and internal zoning of a dam is a critical part of the embankment design. It is essential that the individual particles in the foundation and embankment are held in place and do not move as a result of seepage forces. This is accomplished by ensuring that the zones of material meet filter criteria with respect to adjacent materials. In a zoned embankment the coarseness between the fine and coarse zones may be such that an intermediate or transition section is required. Drainage layers should also meet these criteria to ensure free passage of water. All drainage or pervious zones should be well compacted. Where a large carrying capacity is required, a multilayer drain should be provided. The design of filter has to satisfy the following criteria (Gedeon, 2004);

(i) The soil particles from the protected zone should not pass through the pores of the filter material. This places an upper limit on the size of the filter material.

(ii) The filter should be much more pervious than the protected low permeability zone as to provide effective relief to hydraulic pressure inside that zone. This determines the lower limit for filter material grading.

In general, filters must retain the protected soil and have permeability greater than the protected soil but do not need to have a particular flow or drainage capacity since flow will be perpendicular to the interface between the protected soil and filter.

The following are the criteria that can be used during filter designing;

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.

$$\frac{D_{15}\ of\ filter}{D_{85}\ of\ protected\ layer} < 4\ to\ 5 \qquad \text{..........eq. 2.1}$$

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,

$$\frac{D_{15}\ of\ filter}{D_{15}\ of\ protected\ layer} > 4\ to\ 5 \qquad \text{..........eq. 2.2}$$

The first criteria ensure that the fraction of the protected soil of size D_{85} or larger cannot pass through the pores of the filter.

Initially a finer fraction will penetrate a small thickness of the filter layer but subsequently the coarser fraction of the soil itself will prevent further migration. The permeability of a soil is approximately proportional to the square of its D_{15} size. The second criteria thus ensure that the filter is 16 to 25 times more pervious than the protected soil and the hydraulic gradient correspondingly smaller.

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% passing75 μ), the base material should be analysed on the basis of gradation of fraction smaller than 4.75 mm.

USBR Criteria for Filter Selection

The USBR filter design criterion is presented in Table 2.2.

Table 2.2 USBR Filter Design Criteria

S.No.	Filter Material Characteristics	Ratio $R_{50} = \frac{D_{50} \text{ of Filter}}{D_{50} \text{ of Base}}$	Ratio $R_{50} = \frac{D_{15} \text{ of Filter}}{D_{15} \text{ of Base}}$
1.	Uniform grain size distribution uniformity coefficient $Cu = \frac{D_{60}}{D_{10}} = 3to4$	5 to 10	-
2.	Well graded to poorly graded (non uniform) sub rounded grains	12 to 58	12 to 40
3.	Well graded to poorly graded (non uniform) angular particles	4 to 30	6 to 18

Indian Standard Code

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

i) $\frac{D_{15}\ of\ filter}{D_{85}\ of\ Base} < 5$ ....eq. 2.3

ii) $\frac{D_{15}\ of\ filter}{D_{15}\ of\ Base} > 4\ and < 20$ ....eq. 2.4

iii) $\frac{D_{50}\ of\ filter}{D_{50}\ of\ Base} < 25$ ....eq. 2.5

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

Sherard's Recommendations for Filter Design

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

$\frac{D_{15}\ of\ filter}{D_{85}\ of\ Base} < 9$ ....eq. 2.6

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 coefficient of permeability of dense filters is generally in the range of K = 0.2 to 0.6 D^2_{15} with average = 0.35 D^2_{15} (K in cm/s & D_{15} in mm).

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

e) Using the same criterion, either angular particles of crushed rock or rounded alluvial particles may be used in filter.

US Army corps

US army corps suggested the following filter criteria;

i) $\frac{D_{15}\ of\ filter}{D_{85}\ of\ Base} \leq 5$ ....eq. 2.7

ii) $\frac{D_{50}\ of\ filter}{D_{50}\ of\ Base} \leq 25$ ....eq. 2.8

Filter Thickness

Theoretically, a layer only a few grains diameter thick is adequate to serve as a filter layer. In practice the minimum thickness of a filter layer is that which can be placed without danger of gaps of discontinuity. Because horizontal filters are easily placed they are placed in thinner layers. The minimum practical thickness for a horizontal filter is considered to be 15 cm for sands and 30 cm for gravels and the minimum total thickness of filters is 1m (Garg, 2005).

2.3.4 Shell Material

Downstream shell: It has the primary function of supporting the core and downstream filter under all conditions including full reservoir and during an earthquake. It may consist of inherently strong, stable material such as gravel or rock fill, or of weaker materials with flatter downstream slopes. Poorer quality shell materials, i.e., materials containing more fines or which may break down on placement or exposure to elements and end up less pervious, may be used in downstream sections if adequately filtered internal drainage systems are provided (Jansen, 1988).

Upstream shell: Like that of the downstream shell, its function is to support the core. Unlike the downstream shell, however, the upstream shell is submerged by the reservoir and subject to its fluctuations. The success of a downstream shell to act as a drain in a zoned section depends on the ratio of permeability of pervious material to impervious material used in the core. If the pervious zone has a permeability ratio 1000:1 with the impervious core, then only a very small portion of the downstream shell near the bed will be saturated. However, if the permeability ratio goes on decreasing and an anisotropic condition also exist; the seepage line in the shell will go on rising. Hence, in situation wherein material of high permeability is not available for use in shell at an acceptable cost, it is advisable to provide another drainage system as well. If the shell material contains enough sand a small toe drain of selected coarse material should be provided to a height more than the anticipated saturation level in the shell so as to provide a safe outlet for water. Upstream shells should be as free draining as possible to ensure stability during rapid reservoir drawdown and under earthquake loadings (Jansen, 1988).

2.3.5 Slope Protection

Upper Stream Slope: the upstream slope of the earth dam is protected against the erosive action of waves by stone pitching or by stone dumping. The thickness of the dumped rock should be about 1 meter and should be placed over a gravel filter of about 0.3 m thickness. The filter prevents the washing of fines from the dam into the rip-rap. The provision of such a dumped rip-rap has been found to be most effective, and has been found to fail hardly in 5% cases. The stone pitching, i.e. the hand packed rip-rap requires a lesser thickness and may prove more economical if suitable rock is available on limited quantity. However, when provided in smaller thickness (i.e. single layer); it is more susceptible to damage and has been found to fail in about 30% of cases (Grag, 2005).

Riprap

The main purpose of riprap on embankment dams is to prevent erosion and damage from wave action. Rock fragment dumped riprap is the most common type of slope protection used for embankment dams. Quarried rock is the common source of rock fragments. Riprap must contain a high proportion of near maximum size fragments required by design considerations to resist wave attack and should contain enough smaller rock fragments to fill the voids and lock the larger stone in place. The riprap should be composed of dense, sound, durable rock fragments with near cubical shape as possible. Specification for construction frequently requires the ratio of the maximum to minimum dimension of the rock fragments shall not exceed 3.

This is important because the higher the unit weight, or relative density of the placed riprap, the better it will be to resist wave damage. It should be placed without segregation, and in as dense and interlocked state as possible. This requires machine or hand manipulation of individual rock fragments. The provision of a bedding layer, or layers, is essential to the successful performance of the riprap. Only in rare instances can bedding be eliminated where the underlying material meets filter criteria or the riprap layer is very thick and little wave action energy is left to erode the underlying embankment (USSD, 2011).

Down Stream Slope: the downstream slope of the earthen dam is protected against the erosive action of water and waves, up to and slightly above the tail water depth. Moreover, the downstream slope should be protected against the erosive action of rain and its runoff by providing horizontal berms at suitable intervals, say about 15m or so, so as to intercept the rain water and discharge it safely. Attempts should also be made to grass and plant the downstream slope soon after construction with proper selection of the plant type (Grag, 2005).

2.3.6 Dynamic property of the embankment material in seismically active area

An embankment dam should be capable of retaining the reservoir under conditions induced by the maximum magnitude earthquake where failure would cause loss of life. The following investigations should be accomplished for all proposed and existing embankments (USACE, 1983);

- A seismic stability investigation using a dynamic analysis for proposed and existing dams located in Seismic Zones.

- An evaluation of the liquefaction potential for all dams that have or will have liquefiable materials either in the embankment or foundation.
- A geological and seismological review of existing dams in Seismic Zones to locate faults and ascertain the seismic history of the region around the dam and reservoir.
- A seismic stability investigation of existing dams by dynamic analyses, regardless of the seismic zone in which the dam is located where capable faults or recent earthquake epicentres are discovered within a distance where an earthquake could cause significant structural damage.

2.3.7 Seepage Control Measures

All embankment dams are subject to some seepage passing through, under, and around them. If uncontrolled, seepage may be damaging to the stability of the structure as a result of excessive internal pore water pressures or by piping.

Seepage should be effectively controlled to prevent structural damage or interference with normal operations. In the evaluation of seepage reduction or seepage control measures as they pertain to dam safety, one should review and evaluate the following (USACE, 1982):

- Protective control measures such as relief wells, weighted graded filters, horizontal drains, or chimney drains which prevent seepage forces from endangering the stability of the downstream slope.

- Filters and transition zones designed to prevent movement of soil particles that could clog drains or result in piping (Terzaghi, 1967; USBR, 1987).

- Drainage blankets, chimney drains, and toe drains designed to ensure that they control and safely discharge seepage for all conditions. The design of these features must also provide sufficient flow capacity to safely control seepage through potential cracks in the embankment impervious zone (USACE, 1982).

- Contacts of seepage control features with the foundation, abutments, embedded structures, etc., designed to prevent the occurrence of piping and/or hydro-fracturing of embankment and/or foundation materials (USBR, 1987). If conduits or pipes exist through the embankment, they should be inspected to ensure that they are functional or have been properly sealed.

- Grouting, cut-off trenches, and impervious blankets. Construction records for foundation shaping, treatment and grouting at the contact between the impervious core and foundation.

- Measures such as compaction requirements, seepage collars, placement of special materials, or other similar features to prevent internal erosion from seepage at the interface with concrete structures (USACE, 1982). If seepage collars are present, special attention should be given to compaction requirements around them.

- For existing embankments, all seepage records compiled during the existence of the structure should be reviewed for significant trends or abnormal changes. The causes of any abnormalities should be determined as accurately as possible.

In the following paragraphs the few of seepage control measures (Fig. 2.3) are described in detail (IIT, 2006).

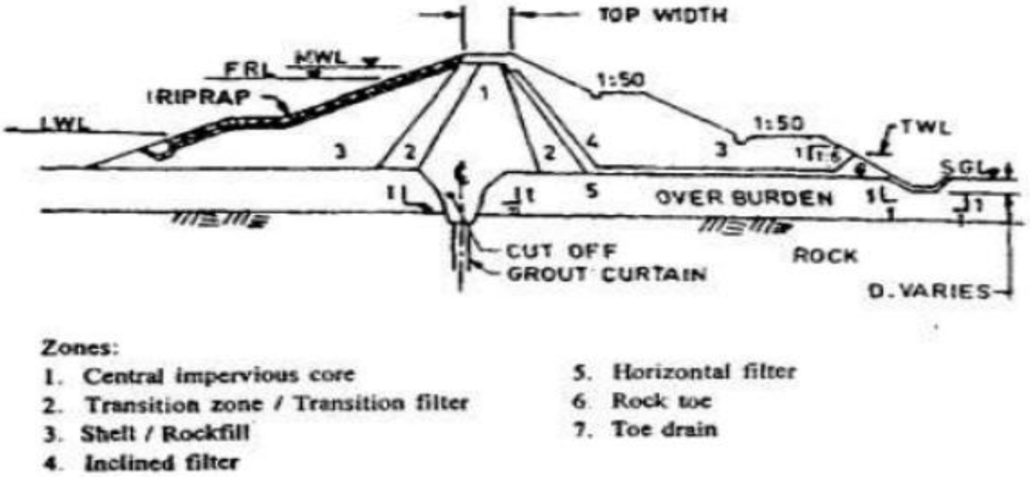

Fig. 2.3 Section of zoned dam showing seepage control features

Transition Zones and Transition Filters

Transition zones/filters in earth and rock fill dams in the upstream and downstream shells are necessary, when the specified gradation criterion is not satisfied between two adjacent zones. When such zones/filters are placed on either side of the impervious core, they help to minimize failure by internal piping, cracking, etc., that may develop in the core or by migration of fines from the core material. The filter material used for drainage system shall satisfy the following criteria:

a) Filter materials shall be more pervious than the base materials;

b) Filter materials shall be of such gradation that particles of base material do not totally migrate through to clog the voids in filter material; and

c) Filter material should help in formation of natural graded layers in the zone of base soil adjacent to the filter by readjustment of particles.

Horizontal filter layers at intermediate levels are sometimes provided in upstream and downstream shells, to reduce pore pressures during construction and sudden drawdown condition and also after prolonged rainfall.The filter layers should be extended up to the outer slopes of the embankment so as to drain out the collected water. These filter layers should not be connected with inclined or vertical filters. A minimum space of 2.0 m or more should be kept between the face of inclined/vertical filter and downstream intermediate filter.

Rock Toe

The principal function of the rock toe (Fig. 2.4) is to provide drainage. It also protects the lower part of the downstream slope of an earth dam from tail water erosion. Rock available from compulsory excavation may be used in construction of the rock toe. Where this is not possible and transportation of rock is prohibitively costly, conventional pitching should be used for protecting the downstream toe of the dam. The top level of the rock toe/pitching should be kept above the maximum tail water level (TWL). In the reach where the ground level at the dam toe is above the maximum tail water level, only conventional pitching should be adopted. The top of such pitching should be kept 1.0 m above the top of horizontal filter, or stripped level, whichever is higher. A zone of coarse filter should be introduced between the rock fill/ pitching and the fine filter. A combination of partial rock toe and pitching may also be considered to effect economy.

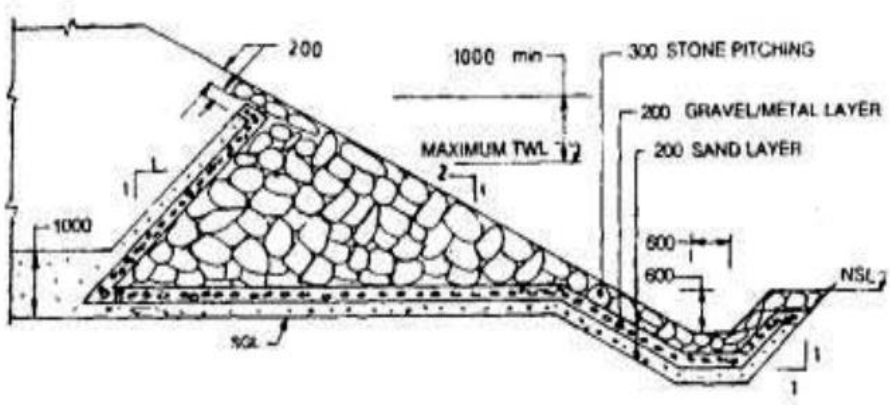

Fig. 2.4 Rock Toe

Toe Drain

Toe drain is provided at the downstream toe of the earth/rock fill dam to collect seepage from the horizontal filter or inner cross drains, through the foundation as well as the rain water falling on the face of the dam, by suitable means according to the site conditions. Additional longitudinal drain and cross drains connected with the toe drain are sometimes provided where outfall conditions are poor. It is preferable to provide the toe drain outside the toe of rock toe, to facilitate visual inspection. The section of the toe drain should be adequate for carrying total seepage from the dam, the foundation and the expected rain water.

Cut-off Trench

The cut-off trench (Fig. 2.5) consists of an impervious fill placed in a trench formed by open excavation into an impervious stratum. Grouting of the contact zone of the fill and the underlying strata constitutes an integral part of the positive cut-off. Pockets of such size, that compaction equipment cannot be operated and pot holes with overhangs, should be filled with concrete.

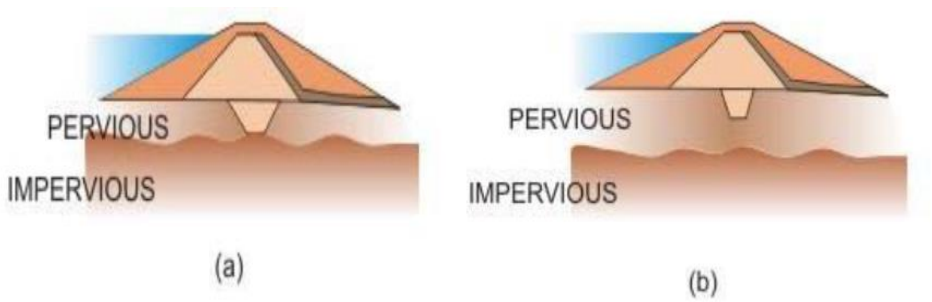

Fig. 2.5 Cut off trenches; (a) Positive cut-off (b) Partial cut-off

Concrete Diaphragm

A single diaphragm or a double diaphragm may also be used for seepage control (Fig. 2.6). Concrete cut-off walls placed in slurry trench are not subject to visual inspection during construction; therefore require special knowledge, equipment and skilled workmen to achieve a satisfactory construction.

Grout Curtain

Grout Curtain in Pervious Soils: Grouted cut-offs are produced by injection, within the zone assigned to the cut-off, of the voids of the sediments with cement, clay, chemicals, or a combination of these materials (Fig. 2.7). An essential feature of all grouting procedures is successive injection, of progressively finer pockets in the deposit. Inasmuch as grout cannot be made to penetrate the finer materials as long as more pervious pockets are available, the coarser materials are treated first, usually with the less expensive and thicker grouts, whereupon the finer portions are penetrated with less viscous fluids.

Fig. 2.6 Concrete diaphragm; (a) Complete (b) Partial

Grout Curtain in Rock: Grout curtain in rock admit of routinized treatment if the purpose is only to block the most pervious zones. These can be treated by cement grout with suitable admixtures. Concentrated seepage would generally develop at the base of the positive cut-off. This zone is particularly vulnerable when a narrow base width is used for the cut-off trench in relation to the height of the dam. The depth of the grouted zone would be dependent on the nature of the substrata and their vulnerability to subsurface erosion.

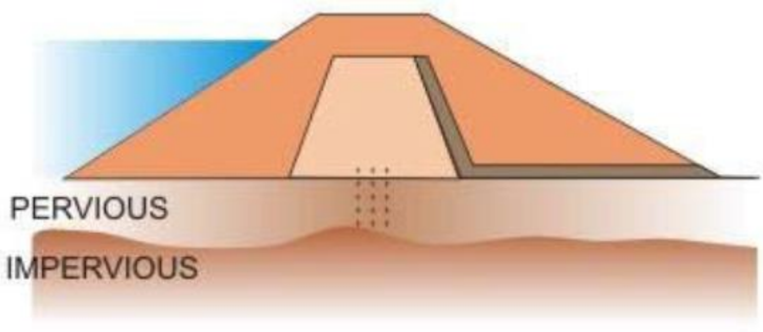

Fig. 2.7 Grout curtain

Steel Sheet Piles

Sheet piles are useful as barrier to arrest internal erosion. However, they have proved to be rather ineffective as a positive means of controlling seepage through pervious deposits. Even if sheet pile cut-offs are intact they are not water-tight because of leakage across the interlocks. In addition the locks may break because of defects in the steel or when a pile hits an obstacle. Once the lock is split, the width of the gap increases rapidly with increasing depth and may assume dimensions of a few meters.

Upstream Impervious Blanket

If a positive cut-off is not required, or is too costly, an upstream impervious blanket combined with relief wells in the downstream section may be used. Filter trenches supplement relief wells in heterogeneous deposits and in zones of seepage concentrations. An upstream blanket may result in major project economies, particularly if the only alternative consists of deep grout curtains or concrete cut-off walls. Since alluvial deposits in river valleys are often overlain by a surface layer of relatively impervious soils, it is advantageous if this natural impervious blanket can be incorporated into the overall scheme of seepage control.

Relief Wells

Relief wells are important aide to most of the preceding basic schemes for seepage control. They are used not only in nearly all cases with upstream impervious blankets, but also along with other schemes, to provide additional assurance that excess hydrostatic pressures do not develop in the downstream portion of the dam, which could lead to piping. They also reduce the quantity of uncontrolled seepage flowing downstream of the dam and, hence, they control to some extent the occurrence and/or discharge of springs. Relief wells should be extended deep enough into the foundation so that the effects of minor geological details on performance are minimized. It is necessary to note the importance of continuous observation and maintenance of relief wells, if they are essential to the overall system of seepage control.

Chimney drains

Chimney drains are an attempt to prevent horizontal flow along relatively impervious stratified layers, and to intercept seepage water before it reaches the downstream slope (Fig. 2.8).

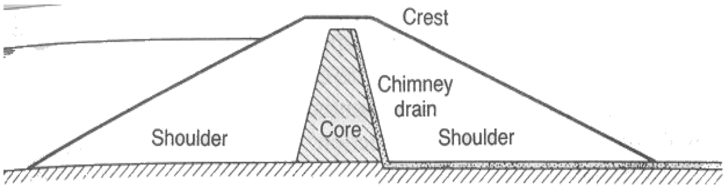

Fig. 2.8 Chimney drain

Chimney drains are often incorporated in high homogeneous dams which have been constructed with inclined or vertical chimney drains. In some major dam projects, chimney drains have been inclined at a considerable slope, both upstream and sometimes downstream. An upstream inclined drain can act as a relatively thin core. In addition to controlling seepage through the dam and increasing the stability of the downstream slope, the chimney drain is also useful in reducing pore water pressures both during construction and following rapid reservoir drawdown.

Horizontal drainage blanket

Horizontal drainage blankets are often used for dams of moderate height. Drainage blankets are frequently used over the downstream one-half or one-third of the foundation area. Where pervious material is scarce, the internal strip drains can be placed instead since these give the same general effect.

2.4 Evolution of methodology for the present study

Based on the systematic literature review general background knowledge about the embankment dams was developed. With a thorough conceptual framework a comprehensive methodology was developed. Below is the summary of the methodology followed for the present study;

Engineering geological mapping of the dam site was carried out in order to understand the general condition of the site including the abutments. This has been performed by collecting rock mass rating data (RMR) on the left abutment. During the preparation of the map, discontinuity orientation data was also collected to determine the preferred orientation so that kinematic check can be performed whether the abutment slope is susceptible to either plane or wedge mode of failure. Collection of samples from dam foundation area, borrow areas and quarry sites for laboratory testing and analysis to determine various index properties was also carried out. These index properties have helped to determine and characterize the proposed embankment dam material for their suitability. Besides, embankment slope stability analysis was carried out. This analysis may help to understand the behavior of the embankment dam material in different adverse site conditions. The methodology that was followed for the present study helped to forward recommendations on the general suitability of the proposed embankment construction material. In evaluating the general suitability of the proposed dam site and embankment construction material in particular various standard specification were followed.

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Chapter 3 Geology and Hydro-Geological Setup

3.1 Geomorphologic Characteristics of the study area

3.1.1 Land form

The geomorphology of the study area can be classified in to four land form units. These landform units are assumed to be derived from structural origin, volcanic origin, denudational (residual origin) and alluvial origin (Fig.3.1) (Yohannes Belete et al., 2009).

1. Alluvial land form

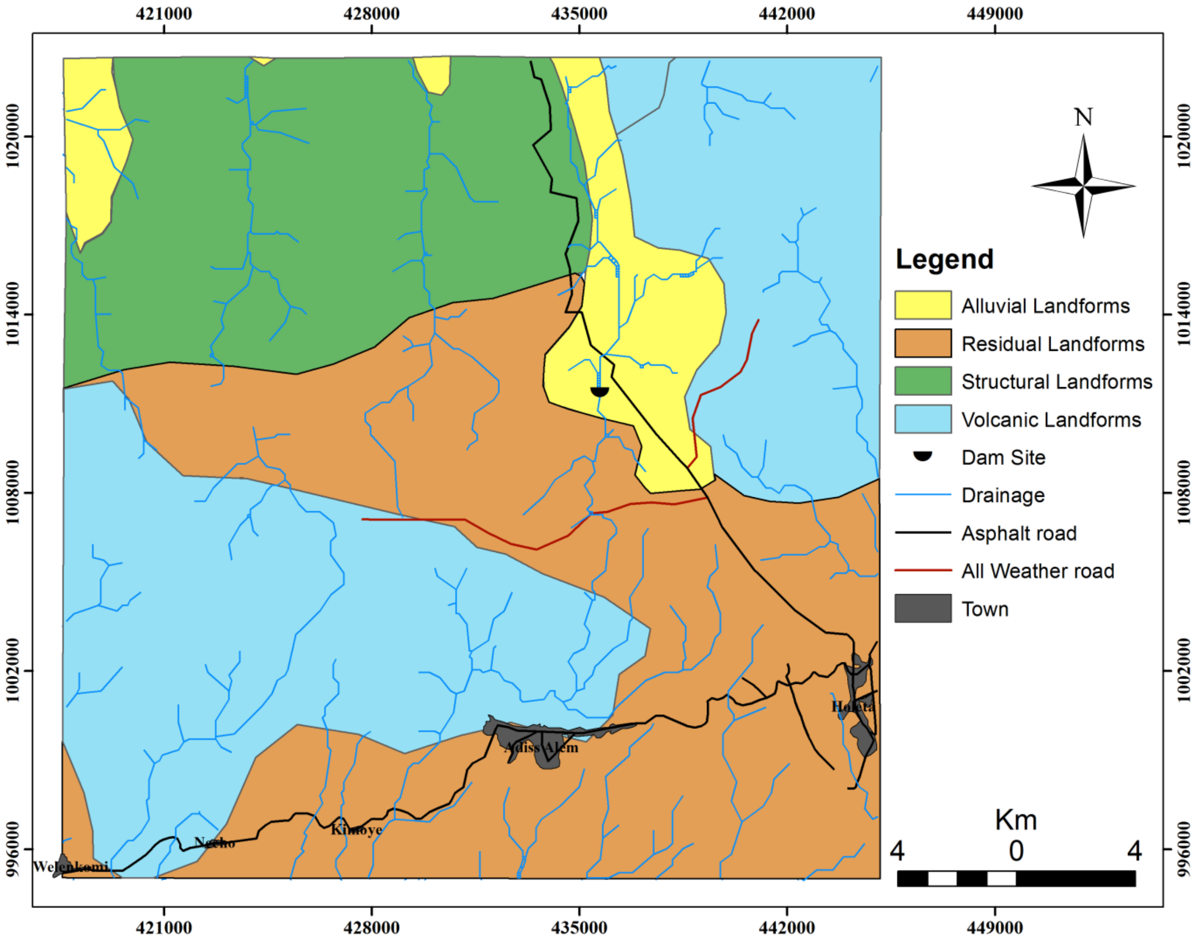
This land form is found at the north and north western part of the study area forming flat plain and with composition of thick brown silty clay and black cotton clay soil at the upstream part of Berga River. This part of the area is observed to be marshy and un-cultivated. The area can be recognized on the satellite images by its flat bottom with the absence of pronounced runoff channels, no considerable erosion and transportation of soil and rock fragments to the downstream can be expected.

2. Residual land form

This land form type is found in the central and southern part of the study area. The area can be characterized by gently sloping and undulating topography and covered by up to 5 m thickness of silty clay soil. In this area, the main process which is responsible for the formation of this land form is degradation, that is, the disintegration of rock (weathering) and the stripping of loose, weathered material from the land surface by the processes like erosion and mass wasting. The area is intensively cultivated producing teff and cereals.

3. Volcanic land form

This area is covered by Tertiary volcanic (upper and lower basalt) which is found at central, western and north eastern part of the study area. This land form is characterized by volcanic eruptions scattered over wide areas. Mainly, the volcanic landscape forms steep slope to cliff morphology (14 - 42°), it occurs mostly forming ridge. At places it occurs as small and elongated hills. It can be differentiated from the rest of the landscapes, which forms numerous discontinuous ridges. The area is moderately cultivated with thin layer of top soil.



(Source: Yohannes Belete et al., 2009)

Fig. 3.1 Geomorphological map of the study area

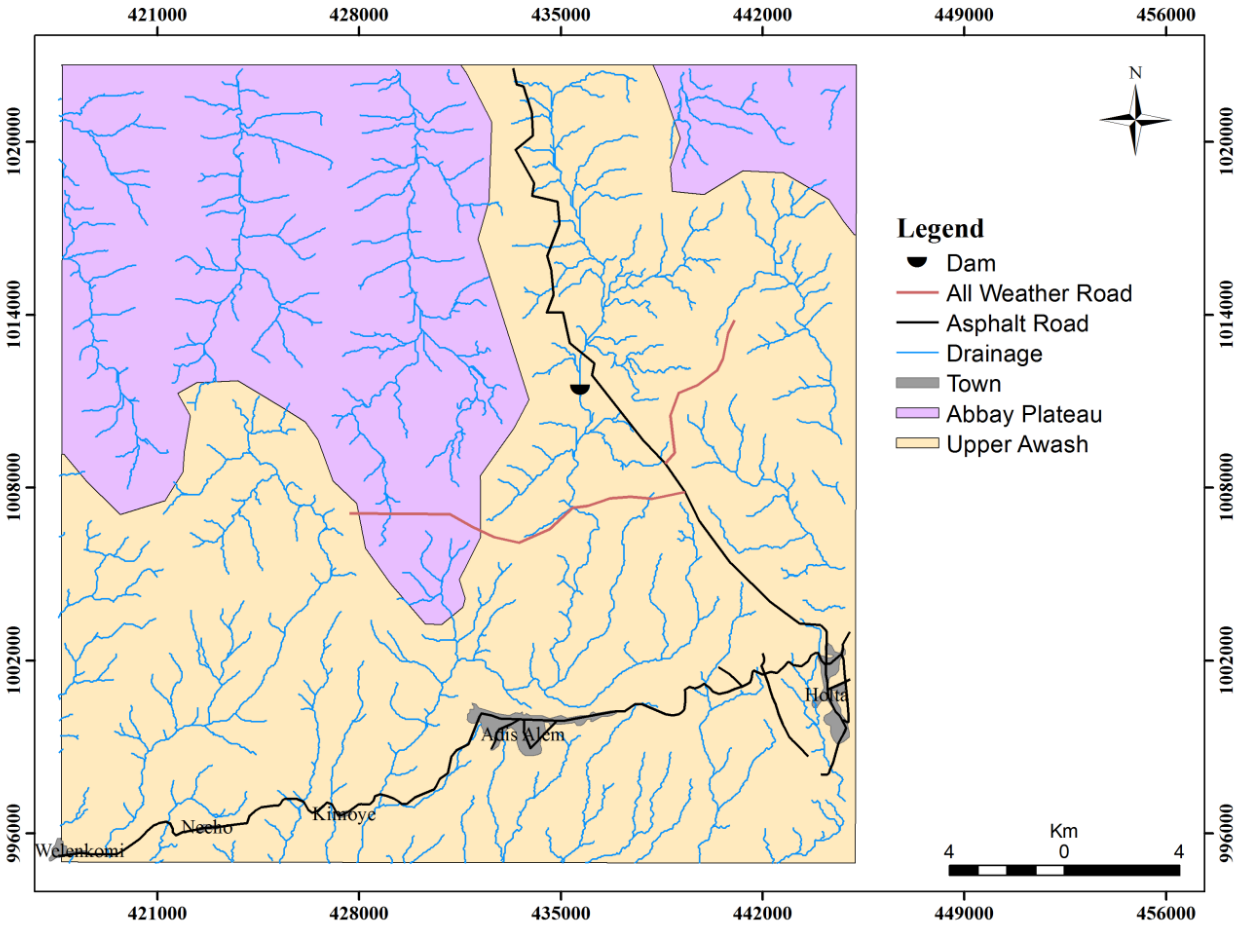
4. Structural land form

This landscape is found on the north and north western part of the area, as it is clearly emerged from the hill shade view, the development and appearance of the landform is largely influenced by an east west running geological structures.

The influence of such geologic structure can range up to large features which exert a dominance influence on the form of an entire landscape. The north-south running volcanic ridges which are separated by north south structures and crossed by east-west fractures shows the clear control of the structure over the landform. The area is generally less to moderately cultivated and is covered by thin layer of top soil.

3.1.2 Drainage

Most of the streams flow from south to north to Abay Basin except Berga and Kela perennial streams which flow from north to south to Awash Basin. The drainage system is controlled by lineaments that run generally north south (Fig. 3.2) (Yohannes Belete et al., 2010).



(Source: Yohannes Belete et al., 2009)

Fig. 3.2 Drainage map of the study area

The type of the drainage pattern in the area is sub-dendritic and trellis which are controlled by type of lithology, topography and structure. Drainage density can be influenced by many factors like rock type, fracture, soil type, relief, vegetation, rainfall amount and intensity.

Some generalization can be made concerning drainage density in the area, flat plain, residual soil, and structurally controlled places shows low drainage densities on the other hand, alluvial soils with homogeneous hydraulic property and valleys reveals high drainage densities.

3.2 Regional geology

3.2.1 General

The regional geology of the area (Fig. 3.3) consists of complete succession of Precambrian basement, Paleozoic and Mesozoic sediments, Tertiary and Quaternary volcanics and

Quaternary sediments. In general the dominant lithological units from the oldest to the youngest are Biotite gneiss, Biotite hornblende gneiss, Meta-gabro Granite, Paleozoic sandstone, Lower sandstone (Adigrat sandstone), Gypsum (Gohatsion formation), Antalo limestone, Muger mudstone formation, Upper sandstone (Debere libanose sandstone), Aiba basalt, Tarmaber basalt, lower ignimbrite Entoto rhyolite, trachyte and ignimbrite, Wechecha-Yere-Furi ignimbrite/trachyte & tarchy basalt, Quaternary plateau basalts of central Ethiopia and Quaternary supper facial deposits (Bereket Fentaw et al., 2010).

3.2.2 Stratigraphy

The brief description of stratigraphy of the area is as follows;

Precambrian Metamorphic rocks

(i) Biotite Gneiss

The biotite gneiss is dark grey, medium to coarse grained and shows gneissosity, which is defined by the alignment of biotite, feldspar and quartz crystals. Strike of foliation is NNW and NNE and it dips gently or at moderate angle towards east or west. It is highly granitized, pegmatized and at places injected by quartz veins (Ilfios, 2008).

(ii) Biotite hornblende gneiss

This unit is observed to intrude the biotite gneiss. There is weakly developed foliation and magmatization. It has medium to coarse grained texture. This rock develops slight degree of weathering and fracture, while outcrops along trails show high degree of weathering. Whenever the biotite gneiss is exposed, the unit is observed to outcrop sporadically as lenses within it. It is mainly composed of hornblende, biotite, plagioclase and quartz (Ilfios, 2008).

(iii) Granite

Its color varies from pink to light grey. Mostly, they are fresh, weakly to moderately foliated with visible biotite, k-feldspar and quartz grains alignment (Ilfios, 2008; Assegid Getahun, 2006).

It is observed to intrude the biotite gneiss and the Mesozoic sandstone. When it is intensively weathered, it is altered to reddish brown soil. Jointing and fracturing perpendicular to the foliation is also commonly observed phenomena. Generally rugged topography is formed by this unit and usually it is discontinuous patch (Bereket Fentaw et al., 2010).

4. Meta Gabbro

On fresh surfaces, they are dark grey, medium to coarse grained. Generally, they show weak to moderate foliation and they are injected by pegmatite and quartz veins. Often, they form domes and they are discontinuous ridges. Jointing is commonly observed structural feature. They are slightly to moderately weathered (Bereket Fentaw et al., 2010).

Paleozoic to Mesozoic Sediments

(i) Paleozoic sediments

Lithologically, they are represented by varying proportions of sandstone, siltstone, mudstone, shale and some paleo soil. The sand stone dominates in the top part, while the shale is abundant in the lower most parts.

Silt stone usually occupies the middle part and the mud stone occurs as intercalation within all the units. There is a development of paleo soil with thickness 2-3 m between the lower parts of Mesozoic sandstone and the upper parts of Paleozoic sandstone. The thickness of Paleozoic sediment reaches up to 200 m in some exposure (Ilfious, 2008; Assegid Getahun, 2007).

(ii) Mesozoic Sandstone (lower sandstone)

It has a maximum thickness of 1131 m (Ilfious, 2008). The succession mostly consists of sandstone with very thin intercalations of silt stone, mud stone and some paleo soil. In the top parts it is, conglomeratic and fine to medium grained, reddish brown to light gray in color. In most cases it develops primary structures like lamination and cross bedding. The degree of weathering and fracturing is high in the top part.

(iii) Gypsum (Gohatsion Formation)

This formation is mainly composed of gypsum and mudstone with variegated color gray at the top, pink at the middle and white at the bottom. There is an intercalation of yellow limestone at the base and shale towards the top (Assiged Getahun, 2006). It is slightly to moderately weathered and compact (Bereket Fentaw et al., 2010).

(iv) Limestone (Antalo limestone)

This formation is characterized by alternating beds of marl. There are also shale intercalations which are frequent towards the bottom. Most of the time the limestone appears as light gray and yellow; when weathered its color changes to dark, white and sometimes to deep yellow.

At places, higher degree of weathering is observed, the precipitation of the secondary minerals such as; calcite and silica are observed along fractures and weak zones. Structures such as karsts, chert nodules and stylelites are observed at the bottom of the limestone.

The petrographic study indicated that this limestone has a range of texture from mudstone to wackstone and pack stone (Assiged Getahun, 2006).

(v) Mugher mudstone –siltstone Formation

The dominant types of rocks in this formation are mudstone, siltstone and shale. However, there are multiple beds of different intercalations. This formation has variegated red colored mudstone, light green to gray for shale and yellow to white for siltstone. It exhibits high degree of weathering. The main structures are laminations, cross laminations, ripple marks and bedding (Bereket Fentaw et.al. 2010).

(vi) Debrelibanose Sandstone (Upper Sandstone)

The sandstone unit exhibits wide range of compositional variation ranging from top part yellow color, well sorted, medium grained to red color, conglomeratic cross bedded sandstone at the middle.

The bottom part is dominated by jointed, fine grained white sandstone. This unit is slightly weathered at the top and highly weathered at the bottom. In general it exhibits coarsening upward sequence (Bereket Fentaw et.al. 2010).

Tertiary Volcanic Rocks

(i) Aiba Basalt

This basalt has a dark grey color on fresh outcrops and it has dark-brown, gray and reddish brown colored weathered surfaces. In this unit there is a vertical compositional variation. The top part is composed of vesicular basalt. At the middle coarse grained basalt is noticed. The bottom of this unit is made up of columnar joint, cliff forming and relatively fresh aphanitic basalt. In general this basalt is characterized by well-developed columnar joints with hexagonal faces, and cliff forming. The maximum thickness measured is about 356m (Assiged Getahun, 2006).

Twenty two radiometric analyses from samples scattered in the whole outcropping area and from different stratigraphic levels give an age ranging from 36 to 18 ma (Giovanni et.al., 1979).

(ii) Olivine - plagioclase phyric basalt (Tarmaber Megzeze basalt)

The Tarmaber unit is made up of often lenticular basalts with large amount of tuffs, scorriaceous lava flows and typical red paleo soils. Sometimes, the basalt flows fill ancient erosion channels cut in the paleo soils. The thickness of the Tarmaber basalts is about 1,000 m. From the 15 isotopic dating its age ranges from 27 to 5 ma. In the Blue Nile area it directly overlies the Ashangi or Blue Nilebasalt (Giovanni et.al., 1979).

(iii) Lower Ignimbrite

This unit consists of inter layers of ignimbrite, ash and tuff. It is grey and black in color and shows columnar jointing having medium to fine grains. It generally consists of two layers coarse at the top and fine at the lower layer. It is highly affected by joints which are vertical and horizontal plunging in N34°E and N15°Edirection (Assiged Getahun, 2007). It is intensively weathered and fractured (Bereket Fentaw et al., 2010).

(iv) Inter layers of ignimbrite, Welded tuff and Ash

This unit is characterized by grey color, containing fragments of ignimbrite, tuff, pumice and some rhayolites. It is fine to medium grained. In most places, it is overlain by aphanitic to medium grained vesicular trachyte (Wechecha – Yere Furi trachyte and trachy basalt) and overlies the lower ignimbrite. It is slightly to moderately weathered and rarely fractured with massive surface (Bereket Fentawet al., 2010).

(v) Entoto Rhyolite and Trachyte

The Entoto mixed rocks constitutes ryholite, trachyte, ignimbrite pyroclastic rocks and sediments. All the rocks are highly weathered and jointed with few layers of agglomerate at some places. There is a red soil development at the contact with underlying basalt. It shows a variegated color of weathering mainly pink, yellow, white and grey and sometimes light green and reddish brown.

This lithologic unit is highly affected by joints trending E-W and N 29°E. It forms high mountain chain called ‘Entoto’ trending E-W (Assiged Getahun, 2007).

(vi) Wechecha- Furi –Yerer Trachyte and Trachy basalt

It has an aphanitic to medium grained texture with vesicular varieties mostly at its lower parts. The characteristic color is light grey to dark grey often to greenish grey. Mostly the trachyte and the trachy basalt are found alternatively layered with the trachyte being dominant. In its lower parts it shows columnar jointing and is affected by two sets of joints (Assiged Getahun, 2007).

(vii) Aphanitic to phorphyritic trachyte (Entoto trachyte)

This unit is generally coarse grained porphyritic and highly weathered. This makes it to have weathering color of light pink to white. It also develops slight fracturing. This unit is affected by EW and SE-NE joints. The joints are filled with dark brown clay. This unit is covered by patches of Quaternary olivine basalt (Bereket Fentaw et al., 2010).

Quaternary volcanic rock and sediments

(i) Quaternary Plateau basalts of Central Ethiopia

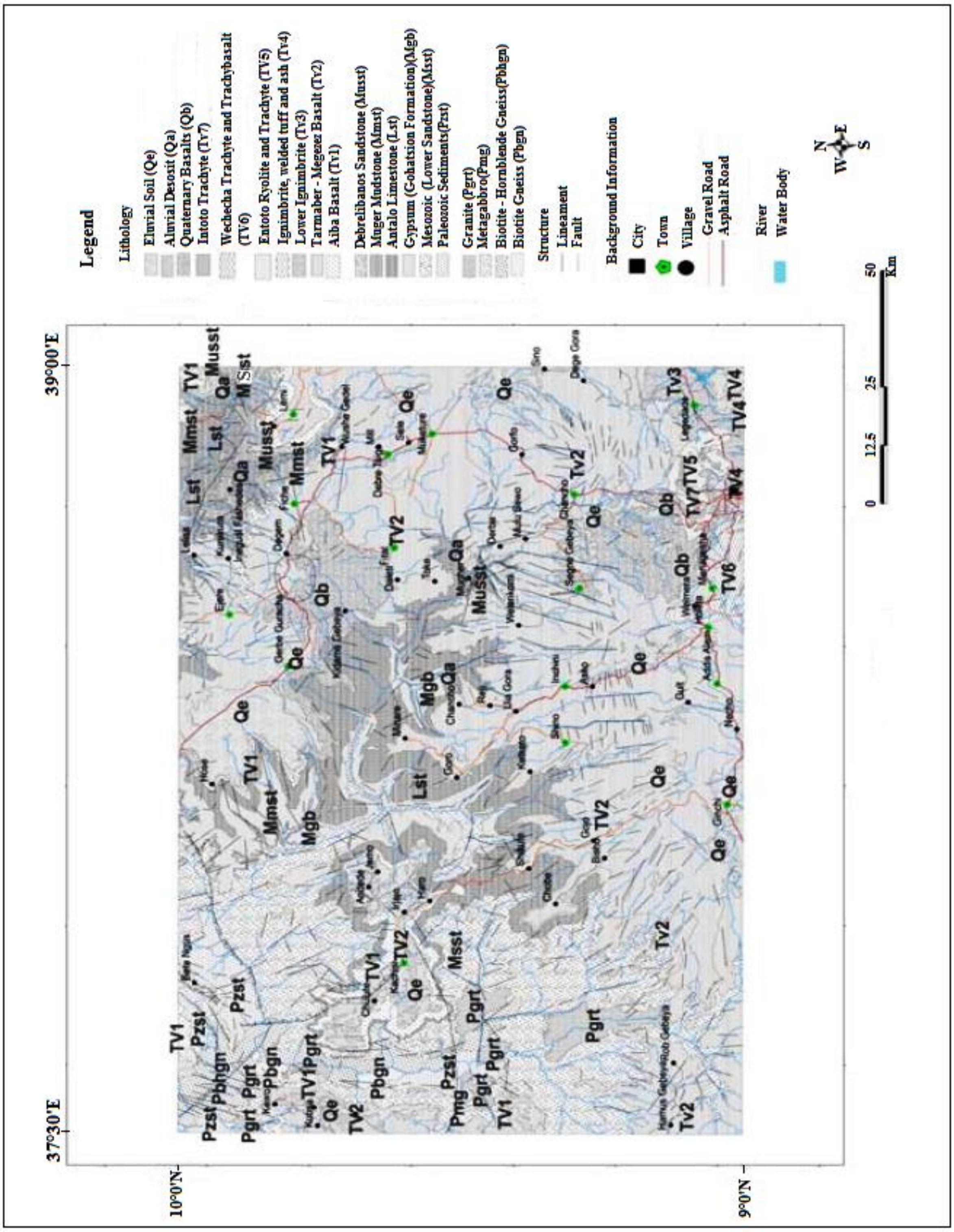
This unit is dominantly olivine basalt with a characteristic grey color on fresh outcrop while after weathering it becomes reddish brown. As observed in most of the cases it outcrops in boulder form and generally vesicles are filled by secondary materials. It forms mainly ridges with a maximum thickness measured to be about 50m.

The northern part is dominantly trachyte and trachy basalt with dark green color having aphanitic and porphyritic texture. In this unit, sheeted and layered flow structure is observed at the top of the dome, and it is oriented in E-W direction with shallow dip angle (22° – 35°) (Assiged Getahun, 2007).

(ii) Quaternary superficial deposits

This unit comprises mainly of eluvial soil and very small position of alluvial soil. The eluvial soil is deposited in eastern south-eastern and central parts of the area. It is dark grey dark brown and black in color, its thickness ranges from 1-5m (Matebie Metebe and Yonas H, 2006).

Its texture varies from sand to Silt size. The basalt, limestone and quartz grain fragment association indicate the probable parent rocks from which this alluvial was derived (Assiged Getahun, 2007).



(Source: Bereket Fentaw et al., 2010)

Fig. 3.3 Simplified Regional Geological Map

3.2.3 Regional Geological Structures

Structures play significant role in any developmental activities. For safe design, construction and performance of the structures in its engineering life time, geological structures may pose problems. The presence of these structures may propagate and amplify earth quake occurrences and therefore, it will be very important to identify such structures during the investigation so that proper remedial measures can be worked out. In the area, major lineaments are traced from satellite images trending NS, NE and EW. The lineaments are manifested by straight streams, valleys and ridges (Fig. 3.4). The northern part of the area is structurally simple and occasionally tectonized whereas on contrary the southern part is highly tectonized and is complex in structure since it is near to the Main Ethiopia Rift margin. The main structures encountered in the area are lineaments, faults, joints and dykes (Bereket Fentaw et al., 2010).

1. Lineament

The major lineaments in the study area trend NE-SW, NW-SE, N-S and E-W directions. The NE-SW lineaments being dominant in the area and they are parallel to the structures of the rift or rift margin. Most of the lineaments can be observed in all units. In addition there are some NWW and NEE trending lineaments which are more concentrated in sedimentary rocks. Most of the lineaments follow trends of liner of ridges, mountains and mainly river valleys and streams. The length of lineaments varies from few meters to about 12 km (Assegid Getahun, 2006; Matebie Metebe and Yonas H., 2006).

2. Faults

There are few normal faults in the area. They are mainly found in north eastern, north western and southern parts of the area. The dominate faults are those associated with the MER trending NE-SW faults. There are also N-S and E-W trending faults. They cut Mesozoic sedimentary and tertiary basalts. The NE- SW and N-S trending faults are act as good ground water conduits in the northwestern and southern parts of the study area (Bereket Fentaw et al., 2010).

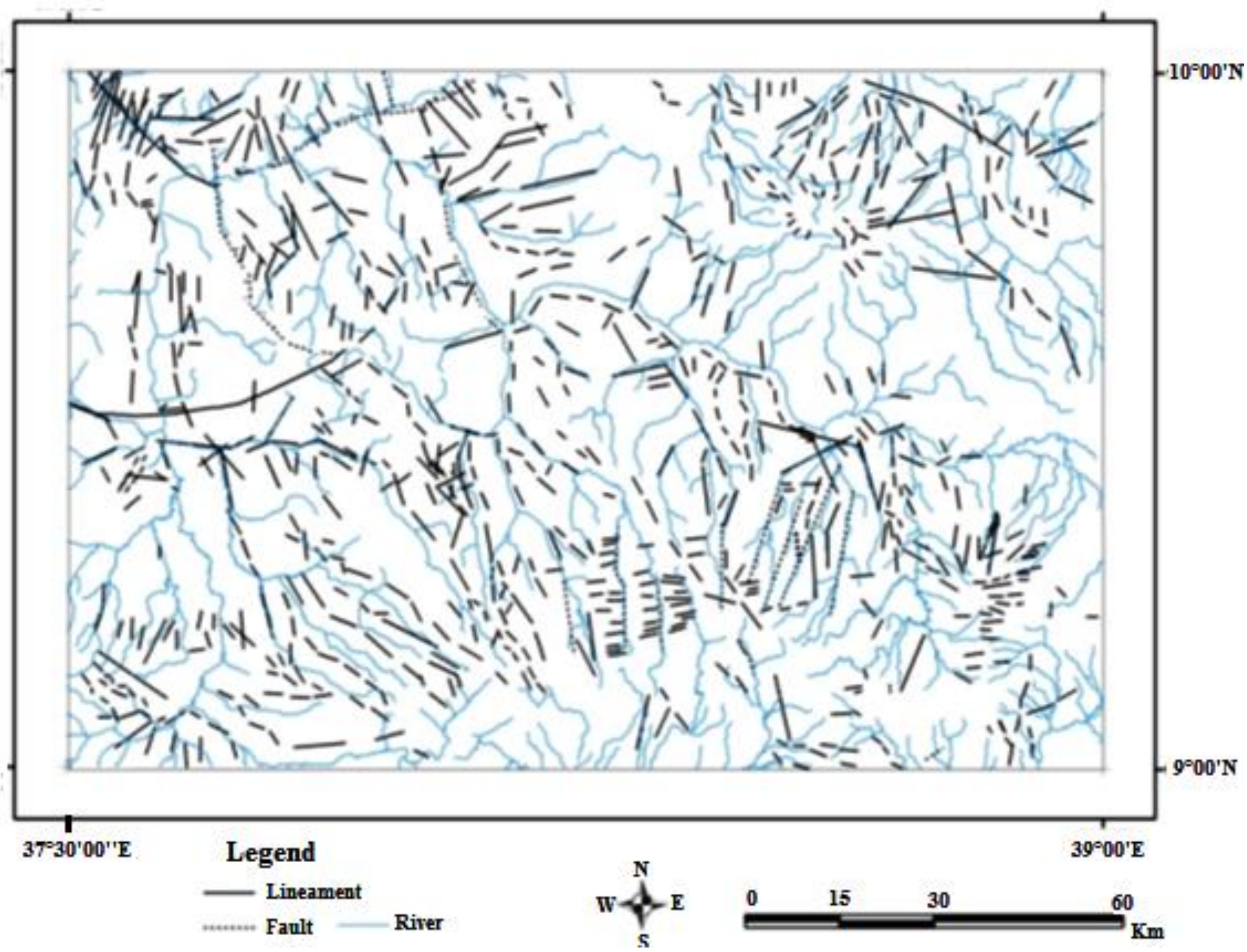
3. Joints

Joints are widely observed in tertiary basalts and upper sandstone. The E-W and NW trending joints are more common on the upper sandstone. Most of the joints are filled by secondary material such as calcite, iron oxide, silica & feldspar (Bereket Fentaw et al., 2010).

4. Dykes

These features are more observable in northern and central part of the area. They are parallel and oriented in the NE direction with maximum width about 2 m (Assegid Getahun, 2006).

Its composition varies from pyroxene porphyric to aphanitic and vesicular basalt.



(Source: Bereket Fentaw et al., 2010)

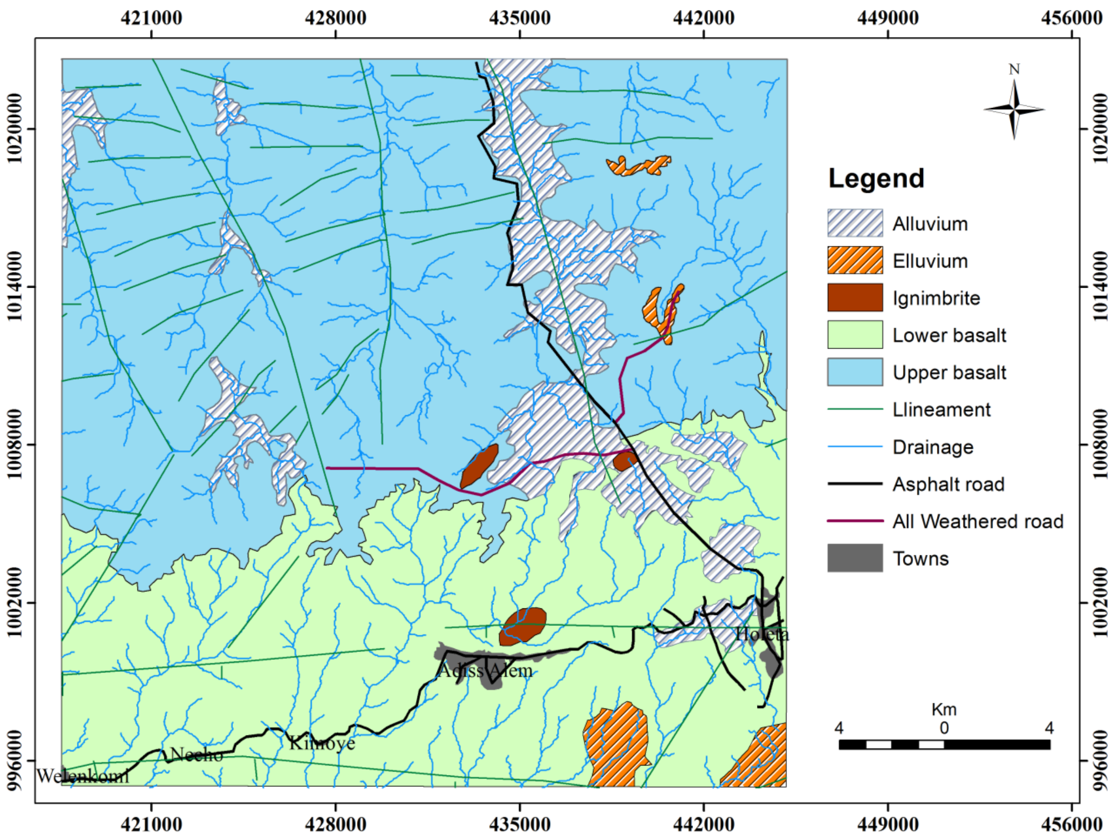
Fig. 3.4 Simplified Regional Structural map

3.3 Geology of the study area

Generally the study area consists of four units; Lower basalt, upper basalt, quaternary alluvium, quaternary residual soil and ignimbrite (Fig. 3.5) (Yohannes Belete et al., 2009).

3.3.1 Lower basalt

The lower basalt covers almost the southern part of Addis-Alem sub sheet. The lower basalt at most of the places out crops as boulders, it has high strength and black in colour. It contains no vesicles, mostly slightly weathered. In some places the lower basalt consists of two major joint sets horizontal and vertical. The rock is aphanitic. From previous petrographic studies the aphanitic basalt is composed of on an average 90% ground mass, 4% plagioclase, 3% olivine, 2% opaque, and 1% sericite.



(*Source: After Regional Geology Department, Geological Survey of Ethiopia,2008*)

Fig.3.5 Geological map of Addis-Alem sub-sheet

3.3.2 Upper basalt

The upper basalt is exposed mainly in the northern part of the sub sheet. The upper basalt is black in colour and it is fresh to slightly weathered. It has high strength and it forms numerous discontinuous ridges. It can be differentiated from the lower basalts by its morphology which usually occurs as ridges.

The upper basalt is sometimes aphanitic and sometimes slightly porphyritic. Previous petrographic study shows 37-65% ground mass, 25-35% plagioclase, 19% olivine, 10% opaque, 5% sericite, 4% iddingsite.

3.3.3 Quaternary alluvium

Quaternary alluvium is deposited mainly in stream channels, terraces and on flat plains. The composition of alluvium as observed from test pits is mainly brown clay soil with some black cotton soil. Generally, along river banks the thickness of brown clay soil is up to 5m. Close to Kimoye village in the river channel of Berga River there is a small quantity of gravel deposit.

The alluvium soil is deposited along Berga, Bora and Boreche River plains.

3.3.4 Quaternary residual soil

Residual soils are in situ developed soils by the decomposition of rocks on which it lies due to physical and chemical weathering (Garg, 2005). The residual soils in Addis- Alem sub sheet are mainly brown clay and black cotton soils. Residual soils are mainly exposed towards south of Addis-Alem and Holeta towns as well as in Telecha locality. As observed from the test pits sunk on residual soil, the thickness of the brown clay soil is over 3 m. The composition of alluvial and residual soils is similar except residual soils occur relatively on higher grounds.

3.3.5 Ignimbrite

Ignimbrite out crops is found in the central part of the study area. The ignimbrite is columnar jointed with joint spacing from 30 cm to 60cm.

3.3.6 Local Geological Structures

In the area, major lineaments are traced from satellite images trending NS, NE and EW. The lineaments are manifested by straight streams, valleys and ridges. These traced lineaments are presented in Fig. 3.6 (Yohannes Belete et al., 2009). These lineaments have size which ranges from 7 km to 18 km. According to Wells and Coppersmith (1994), the magnitude of the earthquakes that occurred in areas is a function of surface rupture length (SRL) in km which is expressed by the formula;

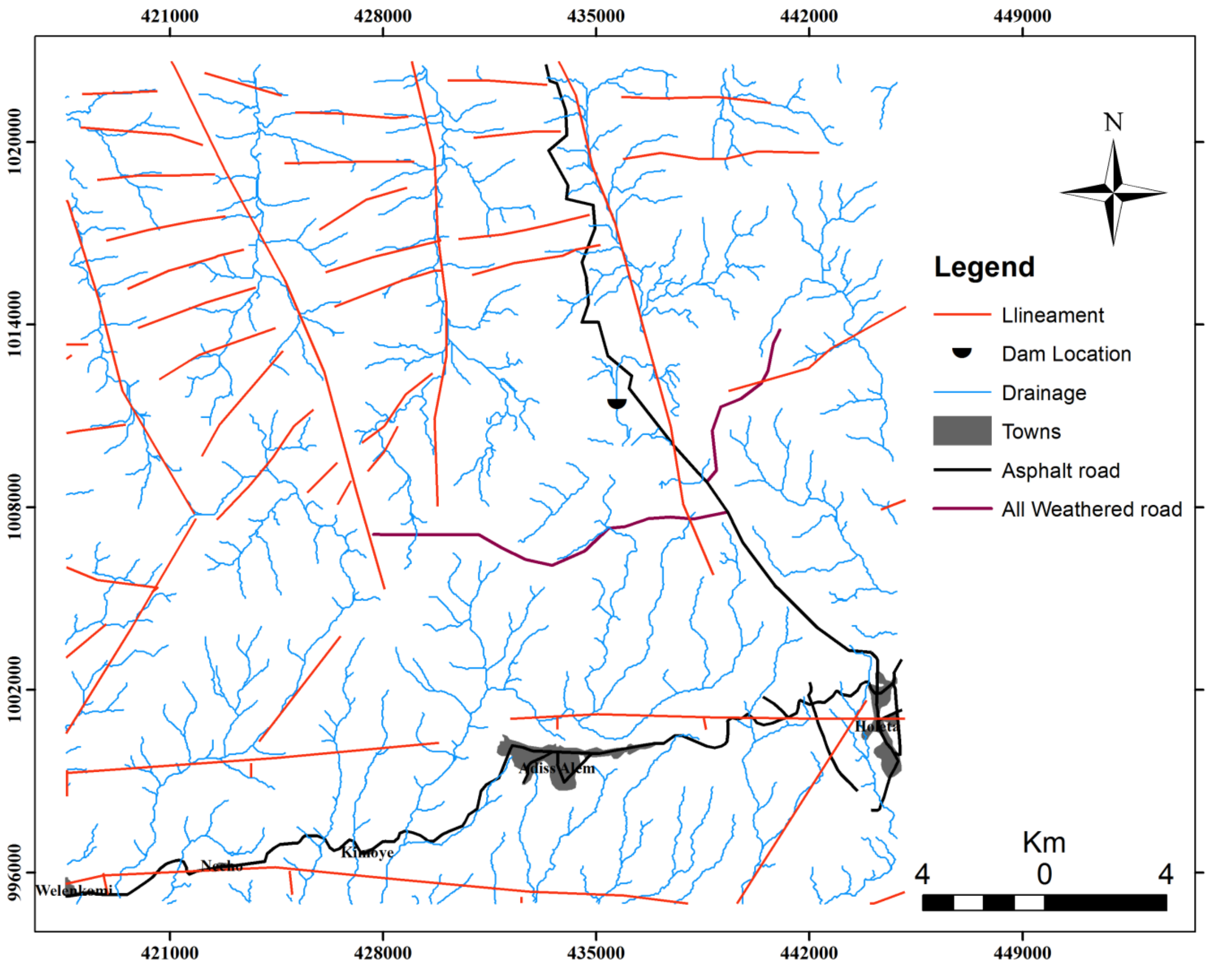
$M = 5.08 + 1.16 * log(SRL)$ ……………*eq. 3.1*

Where; ‘M’ is Moment Magnitude.

Based on the above equation the study area has experienced a moment magnitude up to 6.5 and there is a probability of the occurrence of this much magnitude earthquake in future. However, as the study area is near to the main Ethiopian rift, it can be affected by the magnitude as high as 8.

3.3.7 Geology of the area around the dam Site

The dam site is located in the central plateau of Ethiopia. It is located near the active Main Ethiopian rift which is characterized by active extensional tectonics that has produced series of horst and graben structures (Yohannes Belete et al., 2009).



(Source: Yohannes Belete et al., 2009)

Fig. 3.6 Structural map of the study area

The area is composed of basalts as the only rock types with both alluvial and residual soils (Fig. 3.7). Large part of the area is covered by soils. The basalt is exposed on both the proposed left and right abutments. The basalt is either aphanitic or porphric. It is dark in color and light brown where it is weathered. It forms a gentle slope and its angle increases from 10° to 15° as the height increases. It is highly fractured and forms hills of a height up to 40 m. The alluvial deposits are located in the central area which is found in between the abutments. It is Light to dark reddish brown in color. The residual soil is found at the eastern part of the dam site. It is dark brown in color. Based on the river cut exposure and the pits dug in the area, the soils have a thickness of more than 3 m. Both soils are composed of fines (silt and clay), which occupies large percentage, with little sand content.

The spill way is located on the left bank and is composed entirely of basalt with very thin soil cover. Based on the work by Yohannes Belete et al. (2009), there is one major fault alignment that is found at the eastern part of the area which has the orientation of N-S.

Generally, the dam site geology favors the construction of the area as it is flat land, which makes it easy for accessibility, contains abutments with gentle sloping of 10° - 15°. There is a little possibility of intensive weathering in the abutments as they are formed from the very strong basalts.

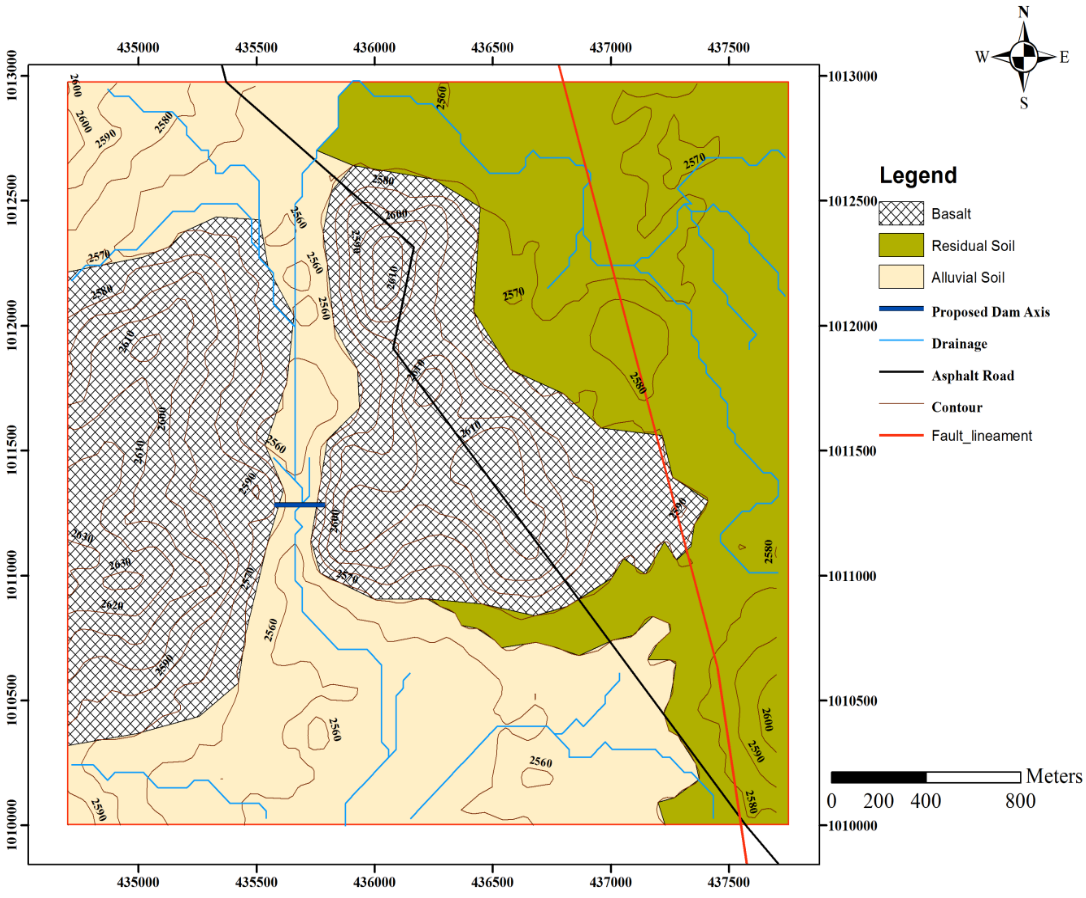


Fig. 3.7 Geological Map of the Dam Site

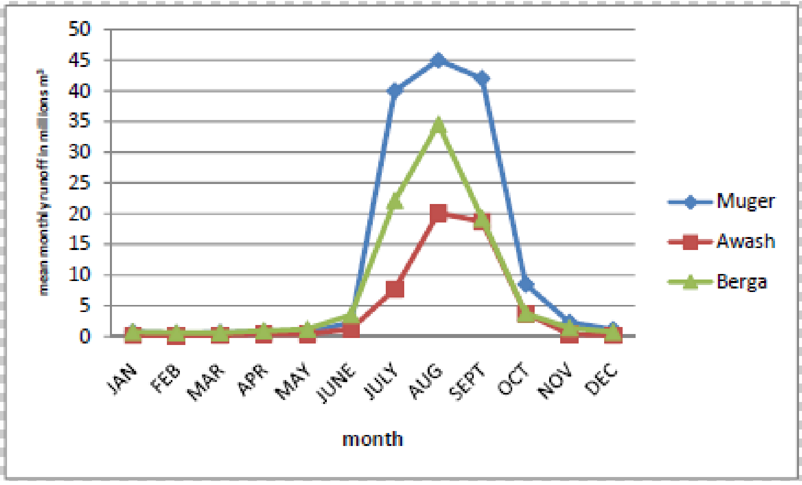
3.4 Hydrogeology

3.4.1 Hydrography

The surface water hydrographs of almost all river gages are of similar type with one peak discharge which is in the rainy season, where there is surplus of water. The runoff at Awash, Mugher and Berga rivers shows a high fluctuation. Awash River has a relatively lower river discharge and runoff than Mugher and Berga Rivers. This seems due to data taken from Awash is at its upstream course as shown in Fig.3.8 (Bereket Fentaw et al., 2010).

3.4.2 Water quality

Different water quality standards were used for evaluating the suitability of water for drinking, irrigation and industrial uses (Bereket Fentaw et al., 2010).



(*Source: After Bereket Fentaw et al., 2010*)

Fig. 3.8 Mean monthly runoff Awash River near Ginchi, Mugher and Berga River (1993-2004)

(i) Domestic Use

For the domestic purpose the groundwater drinking quality standards is evaluated based on the standard given by Ministry of Water Resources (MOWR) and Ethiopia standard. The quality of water for domestic use can be limited to the high concentration of the cations and anions (Table 3.1).

(ii) Irrigation Use

Bereket Fentaw et al. (2010) analyzed the groundwater and surface water quality of the area for Irrigation purpose based on US salinity criteria. The US salinity criteria are made based on the EC values. The EC value of the area ranges from39-2340µs/cm. The field EC measurement and laboratory results shows the water of the study area (ground water) is good for irrigation except two samples which are characterized by high EC value of 2340 and 3350 µs/cm respectively, which could be hard for irrigation purpose. However, it may be satisfactory for salty tolerant crops or plants on soils of high permeability with special leaching. Groundwater quality for irrigation purpose based on US salinity criteria is summarized on Table 3.2 below (Bereket Fentaw et al., 2010). The sodium concentration with the exception of one sample is below the maximum desirable and permissible level of the standards.

Boron

Boron concentration above the recommended limit can cause toxicity. Therefore, the boron concentration in water quality analysis for irrigation purpose should be considered. The water quality of the study area for irrigation purpose is summarized on Table 3.3 (Bereket Fentaw et al., 2010).

Table 3.1 Quality criteria of water samples compared to drinking water standards and guidelines

(*Source: After Bereket Fentawet al.,2010*)

Property	Ethiopian Standard (1) and MOWR Guidelines (2)mg/l		Addis Ababa Map Sheet including present study area (mg/l)	
	Highest Desirable level	Maximum permissible level	Range	Number of Exceeding Value
Na(2)	358	358	1-919	2/2
Ca(1)	75	200	2-124	16/none
Cl(1)	200	600	1-411	1/none
Cl(2)	533	533		None/none
B(2)	0.3	0.3	0.15-4.31	26/26
Mg(2)	50	150	0.6-39	None
SO_4(1)	200	400	1-202	1/none
SO_4(2)	483	483	1-202	None/none
TDS	500	1500	57.23-3527.7	28/2
Total Hardness $CaCO_3$(1)	100	500	7.47-470.28	95/none
Total Hardness $CaCO_3$(2)	392	392		2/2
PH(1)	7-8.5	6.5-9.2	5.01-9.19	76/10
PH(2)	6.5-8.5	6.5-8.5		11/11
NO_3(1)	10	45	0.2-98	58/2
NO_3(2)	50	50		2/2
F(1)	1	1.5	0.02-28.6	10/7
F(2)	3	3		4/4

Table 3.2 The US salinity criteria for irrigation purpose *(Source; After Bereket Fentaw et al., 2010)*

EC (μs/cm)	Class	Exceeding no. of sample	Remark
<250	Low salinity(C1)	94	Good
250-750	Moderate(C2)	111	Good for soils of medium permeability for most plant
750-2250	Medium to high(C3)	10	Satisfactory for plants having moderate salt tolerance
2250-4000	High(C4)	2	Satisfactory for salty tolerant crops on soils of good permeability with special leaching
>4000	Very high(C5)	Nil	Not fit for irrigation

Chloride

The sensitivity of crops to chloride concentration is an important criterion to put standard for irrigation. Based on the Canadian water quality guideline most water samples are fit for irrigation of sensitive crops having less than 1000 mg/l of chloride (Bereket Fentaw et al., 2010).

Table 3.3 Quality for irrigation based on boron *(Source; After Bereket Fentaw et al., 2010)*

	Quality criteria based on Boron					Sample number and characteristics				
	E	G	P	D	U	E	G	P	D	U
Sensitive crops	<0.33	0.33-0.67	0.67-1	1-1.25	>1.25	43	14	2	2	8
Semi-tolerant	<0.67	0.67-1.33	1.33-2	2-2.5	>2.5	57	5	3	1	3
Tolerant	<	1-2	2-3	3-3.7	>3.75	59	6	2	1	1
Note E = Excellent G = Good P = Permissible D = Doubt full U = Unfit										

Sodium Adsorption Ratio (SAR)

The sodium hazard of irrigation can be evaluated by the sodium adsorption ratio indexes. Based on the sodium adsorption ratio (SAR) method most water samples are good for irrigation and have SAR value less than 10 except few which show SAR value >10 (See Table 3.4) (Bereket Fentaw et al., 2010).

Table 3.4 Sodium Classification *(Source; After Bereket Fentaw et al., 2010)*

SAR Value	Class	Remark	No. of studied case
<10(s1)	Low	Good	163
10-18(s2)	Medium	Good for coarse-grained permeable soil unsatisfactory for highly clayey soil with low leaching	Nil
18-26(s3)	High	Suitable only with good drainage, high leaching organic addition	1
>26	Very high	Unsatisfactory	2

Total dissolved solids (TDS)

For irrigation purpose a criteria is used to evaluate the water sample based on TDS values. Only few samples exceed the criteria having TDS values of 2143.96 and 3527.7 mg/l respectively, which are unsuitable for irrigation purpose as shown in Table 3.5 (Bereket Fentaw et al., 2010).

Table 3.5 Irrigation water quality criteria based on Total dissolved solids (TDS)

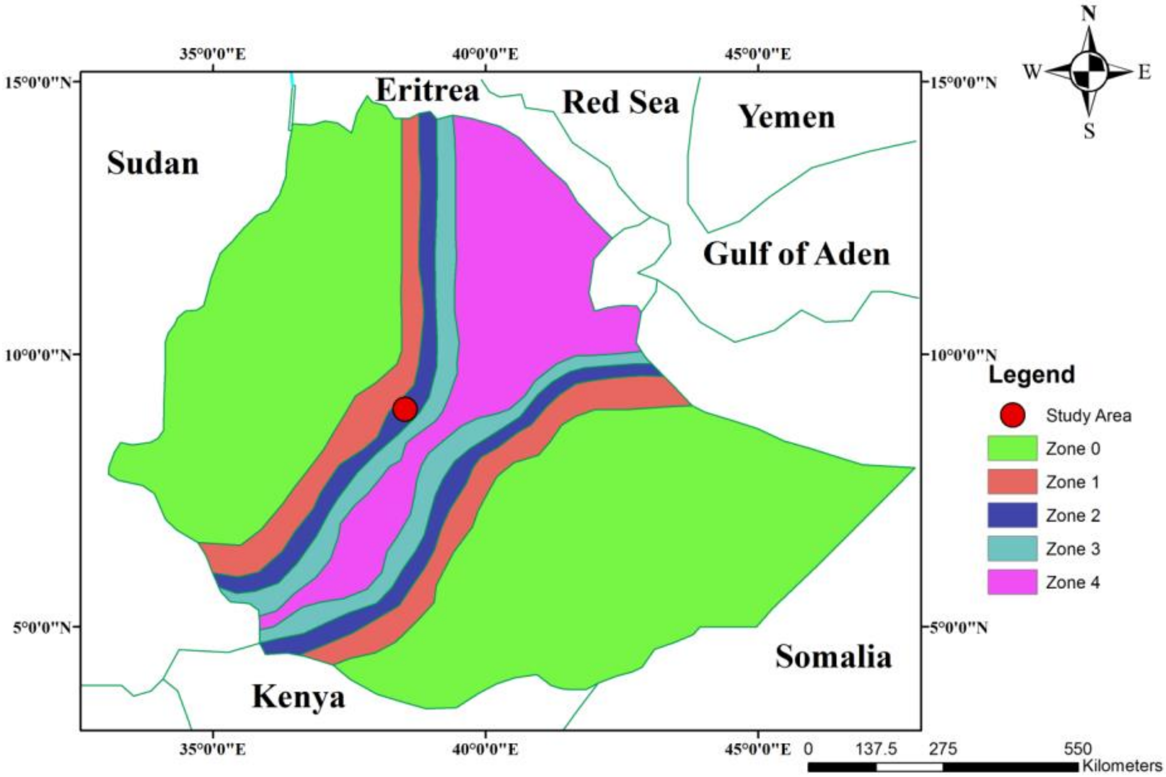
(Source; After Bereket Fentaw et al., 2010)

Water class	TDS (mg/l)	Remark	No. of samples in the range
Excellent	<1000		164
Suitable	1000-1700	If Ca≥ 25%(Na+Ca)	1
Unsuitable	1000-1700	If Ca≤ 25%(Na+Ca)	Nil
Unsuitable	>1700		2

3.5 Seismicity of the Study Area

The surface of the earth is in a state of continuous disturbance due to vibration called earth quakes. Although noticeable and sever earthquakes occur only occasionally, the destruction they cause through loss of life and property is often alarming (Mukerjee, 1995). The study area is located near the margin of the main Ethiopia rift which is an active divergent boundary. Fig.3.9 shows the location of the study area within the seismic zone of Ethiopia.

A seismic hazard zoning released in 1995 by Ethiopian Building Code Standard (EBCS, 1995), classifies the country into five zones. The zoning simply is a buffer about the Ethiopian Rift which is a tectonically active continental rift. According to the Seismic hazard map of Ethiopia, Addis-Alem sub sheet and the proposed Berga dam site are located in Zone 2 and can be categorized under the seismic zone with a design ground acceleration of 0.05g, where 'g' stands for gravity in cm/sec^2. Details of the ground acceleration and seismic zones are presented in the Table 3.6.



(Source; adopted by the Ethiopian Building Code Standard (EBCS 8))

Fig.3.9 Seismic Hazard map of Ethiopia

Table 3.6 Bed rock Acceleration ratios

Zone	**4**	**3**	**2**	**1**
a	0.1	0.07	0.05	0.03

Even if the above classification is given, for the actual design of the dam detailed site ground motion investigation must be conducted so that the effect of earthquakes on ground can be identified.

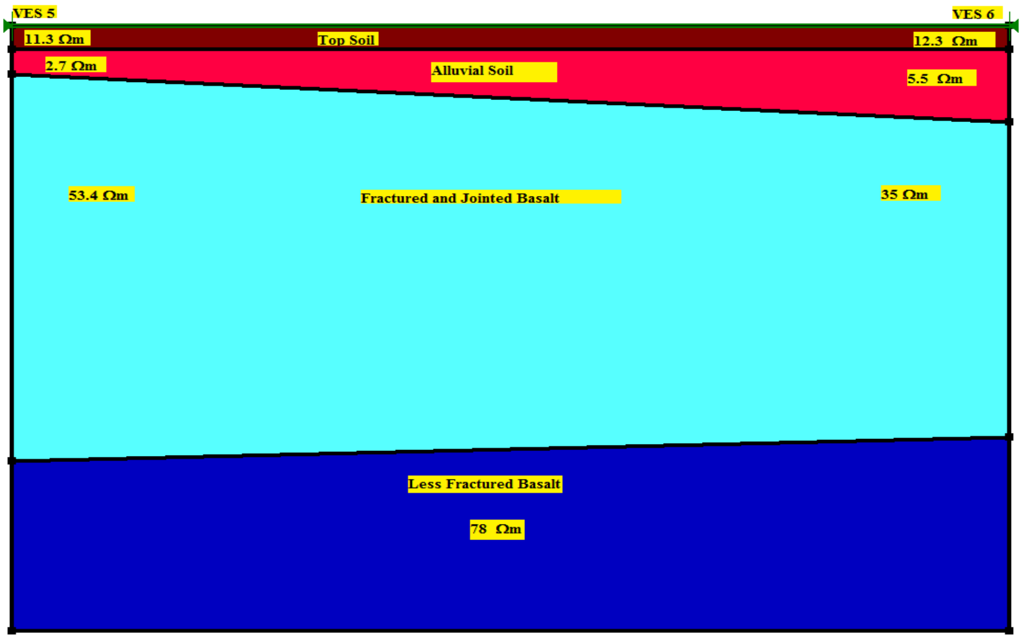
3.6 Geophysical Investigation conducted on the proposed dam site

3.6.1 General

Electrical resistivity method serves as a useful tool in determining the thicknesses of unconsolidated materials, depth to water table, depth to sound rock as well as mapping sub-surface geological structures. These are fundamental components that are required in any civil engineering constructions. With this consideration Yohannes Belete et al., (2009) carried out electrical resistivity survey (along with engineering geological mapping) by applying Vertical Sounding (VES) on the Berga dam site.

3.6.2 Interpretation and results

VES-5 & VES-6: these surveys were conducted across the proposed dam axis in order to acquire information about the overburden thickness and hydro-geological characteristics of the reservoir site. At both places the top soil with brown color is characterized by similar thickness (0.9 m) and resistivity value (11.3 and 12.3 Ω-m resistivity, respectively). Similarly, the second layer has also revealed 1 and 3.1 m thickness and slightly different resistivity responses (2.7 and 5.5 Ω-m). It is attributed to alluvial soil of variable grain sizes.



(Source: Yohannes Belete et al., 2009)

Fig. 3.10 Geo-electric section using the data across the proposed dam site axis

The third layer at VES-5 appears more resistive (53.4 Ω) and slightly thicker (16.4 m) than the same bed at VES-6 (35 Ω-m and 13.7 m). This enhanced response is associated with the effect of fractured and jointed columnar basalt. In both cases the bottom layer (compositionally to be similar to the overlying bed, but with different degree of fracturing) shows more enhanced resistivity (above 78 Ω-m) and its occurrence is inferred starting from a depth of about 18 m from the surface. From the hydro-geological point of view, the third and fourth layers with thickness over 33 m represent the aquifer bed. At these stations the depth to the water table ranges from 2 to 4 m. Near the proposed dam axis the total thickness of the overburden is estimated to be about 9 m, out of which about 2 m brown clay and 7 m superficial jointed basalt as shown in Fig. 3.10 (Yohannes Belete et al., 2009).

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Chapter 4 Engineering Geological Appraisal of Dam Site

4.1 Introduction

For the success of dams and their associated reservoirs, five geologic conditions are important which are; watertight reservoir basin in which the rate of accumulation of silt is not likely to exceed the admissible limits, a narrow river-channel which can be blocked with a relatively small dam, safe foundation, provision for disposal of surplus water through a suitable spillway and availability of the required materials for construction (Mukerjee, 1995).

In order to determine these conditions for any dam site, a detailed engineering geological investigation and characterization of the dam site is very important. This includes investigation on rock and soil physical and mechanical properties which are important parameters for engineering geological design and construction (USBR, 1983).

The proposed Berga dam site is found in the Central Plateau of Ethiopia. This Central Plateau covers wide area. It is characterized by a flat plain with gentle slopes and undulating terrain build by Tertiary volcanic rocks (basalt and ignimbrite) and is covered by alluvial and residual deposits.

4.2 Classification of rock mass present at dam site

The rock mass classification concept has been applied extensively in engineering design and construction such as; tunnels, slopes and foundations for a long time. The main objective of rock mass classification is to provide quantitative data and guidelines for engineering purposes that can improve originally abstract descriptions of geological formation (Ya Ching et al., 2006). Until now for rock engineering, the most commonly used rock mass classification systems are the Rock Structure Rating, RSR (Wickham et al., 1972), the Rock Mass Rating, RMR (Bieniawski, 1973, 1975, 1979, 1989) and the NGIQ-system (Barton et al., 1974).

Based on the previous and present research works it has been found that the rock mass present at the proposed Berga dam site has a high rock mass strength. The main rock type present in the area is basalt which is either porphyritic or aphanitic. The term porphyritic refers to a texture in which a portion of the mineral grains is significantly larger than the rest of the rock. A rock of the latter category is said to be microcrystalline (aphanitic), if the constituent mineral grains can only be distinguished under a microscope (Mukerjee, 1995). Both the abutments consist of highly disintegrated rock mass (Plate 4.1 and Plate 4.2). Classifying the rock of the dam foundation may help in designing the outline of the embankment which will basically be based on the stability and bearing capacity of the foundation material.

Plate 4.1 Left Abutment

Plate 4.2 Right Abutment

To characterize the rocks of the proposed dam site the method used was rock mass rating (RMR) system. This system is the sum of five parameters and one correction factor (Bieniawski, 1989). The parameters used for RMR system were; strength of rock material, Rock Quality Designation (*RQD*), Spacing of discontinuities, Condition of discontinuities, Groundwater conditions and the correction factor i.e. Orientation of discontinuities.

In previous work on the study area (Yohannes Belete et al., 2009), engineering geological mapping was conducted. During this work the rocks, especially the basalt and ignimbrite, were classified to be either; rock mass with high strength or medium strength rock mass, respectively. Even though this classification is very important, additionally for the present work the rock mass at dam site was further characterised by utilizing Bieniawski's (1989) RMR system. Data was collected to compute the RMR and also previous work's data especially for the USC was used which was collected using point load test and laboratory test. As the right abutment contains only highly disintegrated rock mass with soil cover, it was analysed by using rotational mode of failure and the left abutment was analysed for plane and wedge modes of failures. However, it is recommended that during the construction of the dam, stripping to reasonable depths must be conducted to analyse both abutments in more detail so that a more tangible result can be obtained.

4.2.1 Strength of the rock material

Rocks are aggregates of minerals and they are the individual units constituting the crust of the earth (Mukerjee, 1995). From engineering point of view rocks can be classified either as; intact rock or as a rock mass. Intact rock is the term applied to rock containing no discontinuities such as; joints and bedding. Sometimes it is also known as “rock material”. A rock mass is a mass of rock containing discontinuity planes. The rock blocks within the discontinuity planes have intact rock properties. Strength of intact rock depends on component mineral strengths and the way they are bound together – by interlocking or cementation. Rock mass strength applies to a mass of fractured weaknesses. Hardness is not directly related to strength; normally hardness is only relevant for the ease of drilling. Strength of a rock mass largely depends on the density, nature and extent of the fractures within it. Rock mass strength also relates to rock strength, weathering and water conditions (Tony, 2010).

According to the results of point load test the upper basalt, which the dam foundation is composed of, has rock material strength that generally ranges from 6.3 – 8.2 MPa. The un-confined compressive strength calculated from point load strength value generally ranges from 151.2 to 196.8 MPa. From point load and un-confined compressive strength values, the upper basalt is classified as rock with high mass strength.

Upper Basalt rock unit is exposed in the abutments of the Berga dam and in many quarry sites where most of it have little or no top soil (Plate 3). Its density varies from 2.56 to 2.91 gm/cm^3, which indicates its high resistance to weathering. The water absorption result shows that most of the basalt samples have water absorption less than 1% which implies the rocks could be used as aggregate. According to laboratory results, the UCS result of the upper basalt ranges from 177 to 307 MPa which indicates high rock mass strength (Table 4.1(a) and Table 4.1(b)) (Yohannes Belete et.al. 2009).

4.2.2 Rock Quality Designation (RQD)

The concept of rock quality designation, *RQD*, was introduced by Deere (1964). It is based on the percentage core recovery when drilling rock with NX (57.2 mm) or larger-diameter diamond core drills. Assuming that a consistent standard of drilling can be maintained, the percentage of solid core obtained depends on the strength and number of discontinuities in the rock mass concerned. The *RQD* is the sum of the core sticks in excess of 100 mm, expressed as a percentage of the total length of core drilled. However, the *RQD* does not take account of the joint opening and condition, a further disadvantage being that with discontinuity spacing greater than 100 mm, the quality is excellent, irrespective of the actual spacing (Bell, 2007).

Plate 4.3 Basalt exposure at proposed quarry site

Table 4.1(a) Engineering parameters of rocks (*Source: Yohannes Belete et al., 2009*)

Sample	Easting	Northing	Elevation (m)	Water Absorption (%)	Porosity (%)	Bulk Density (gm/cm^3)	Formation	Texture
AR – 1	440824	1005884	2518	1.2	3.5	2.91	Lower Basalt	Vesicular Basalt
AR – 2	435142	1012190	2592	0.55	1.6	2.89	Upper Basalt	Porphyritic Basalt
AR – 3	434053	1021078	2596	0.8	2.27	2.84	Upper Basalt	Aphanitic Basalt
AR – 4	437209	1011198	2588	0.55	1.55	2.84	Upper Basalt	Porphyritic Basalt
AR – 6	431892	1006898	2656	0.78	2.24	2.86	Upper Basalt	Porphyritic Basalt
AR – 7	443016	1002176	2412	0.5	1.42	2.84	Lower Basalt	Aphanitic Basalt
AR – 8	419559	995400	2150	1.26	3.21	2.56	Lower Basalt	Porphyritic Basalt
AR – 9	442391	1001123	2410	0.64	1.77	2.77	Lower Basalt	Aphanitic Basalt
AR – 10	439417	1007500	2567	9.63	20.03	2.08	Ignimbrite	Ignimbrite
AR – 11	420658	1022701	2817	0.76	2.16	2.85	Upper Basalt	Porphyritic Basalt
AR – 12	434893	1001017	2383	10.65	21.44	2	Ignimbrite	Ignimbrite

Palmström (1982) suggested that, when no cores are available but discontinuity traces are visible in surface exposures or exploratory adits, the RQD may be estimated from the number of discontinuities per unit volume. The suggested relationship for clay-free rock mass is;

$RQD = 115 - 3.3\ Jv$ …………..*eq. 4.1*

Where; ‘Jv’ is the sum of the number of joints per unit length for all joint (discontinuity) sets known as the ‘volumetric joint count’.

To determine the quality of the rock mass of the left abutment, data pertaining to RQD was collected. During the present work, the RQD data was collected every 10 m along the slope height so that it represents the overall rock mass present on the abutment. As there was no drilling data in the area, volumetric count was used for which number of the discontinuities with a length equal or greater than 10 cm were counted within 1 x 1m exposed rock mass. Even though it was difficult to find such exposure because of the soil cover in the abutments, few good exposures along the slope were found from where data was collected. Generally, the rock mass exposed on abutment has RQD for which the ratings generally range from 13 to 17 according to the rock mass rating system.

Table 4.1(b) Engineering parameters of rocks *(Source: Yohannes Belete et al., 2009)*

Sample	Degree of weathering	W1 (cm)	W2 (cm)	Point Load test height (m)	Peak Point Load Value (KN)	Index strength (Mpa)	UCS (Mpa)	Schmidt Median (Kg/cm^2)
AR – 1	SW	8	13.5	5.5	48	7.3		
AR – 2	SW	9	10	6.8	49.23	7.62	225.8	367
AR – 3	Fresh	9	10	5.5	42.78	8.2	297.1	367
AR – 4	SW	9	15	6.5	54.74	7.02		
AR – 6	SW to MW	7	10.5	5	28.82	6.76		
AR – 7	SW	9	10	6.8	51.41	7.96	367.5	400
AR – 8	SW	8	11	7	36.06	5.42	177.8	681
AR – 9	Fresh	10	11	5.6	49.28	8.4		
AR – 10	SW	7	10	6.8	25.62	4.43	87.36	431
AR – 11	SW	10	11	5.2	41.16	6.3		
AR – 12	SW	6	8	7.6	12.33	2.32	68.03	
Note: SW – Slightly Weathered; MW – Moderately Weathered								

4.2.3 Spacing of discontinuities

The discontinuities can be the single most important factor governing the deformability, strength, and permeability of the rock mass. Moreover, a particularly large and persistent discontinuity could critically affect the stability of any excavation. For these reasons, it is necessary to develop a thorough understanding of the geomechanical properties of discontinuities and rock mass and understand the way in which discontinuities affects the engineering structures (Suping et al., 2007).

Spacing of the discontinuities of the left abutment was measured as a parameter for the rock mass classification and in general it is up to 4 cm wide. Even if the individual intact rocks have higher strength, the spacing of the discontinuities will affect the RMR result. During the construction stage, all these disintegrated rock mass needs to be removed to minimize the effect of a weak surface or to provide a way for the water circulation which will greatly affect the stability of the abutment.

4.2.4 Condition of discontinuity

The nature of the opposing joint surfaces also influences rock mass behaviour because the smoother they are, the more easily can movement take place along them. However, joint surfaces are usually rough and may be slickensided. Hence, the nature of a joint surface may be considered in relation to its waviness, roughness and the condition of the walls. Waviness and roughness differ in terms of scale and their effect on the shear strength of a joint. Waviness refers to first-order asperities that appear as undulations of the joint surface and are not likely to shear off during movement. On the other hand, roughness refers to second-order asperities that are sufficiently small to be sheared off during movement. Increased roughness of the discontinuity walls results in an increased effective friction angle along the joint surface. These effects diminish or disappear when infill is present (Bell, 2007).

At the present dam site the condition of the discontinuity in the rock mass of the left abutment in general is rough, with no infillings and weathering. This condition shows that the rocks are very strong and may not easily weather. The roughness and waviness of the rock mass will increase the shear strength between the adjacent rock blocks which eventually enhance the stability of the abutment slopes.

4.2.5 Ground water condition

Presence of water in rocks is one of the major factors that will affect the stability of the rock slopes. Water develops pore pressure and minimizes the shear strength of rocks in openings. Therefore, special attention must be given to the presence of water (Bell, 2007).At the time of investigation the Left abutment of the dam was completely dry and stability of the slope represents the dry condition however, during rainy season slope may be recharged with water and water forces may develop which may affect the slope stability condition.

4.2.6 Orientation of discontinuities

The orientation of discontinuities within the rock mass exposed over the slopes may indicate the possibilities for potential instability. If a discontinuity set is oriented towards the valley or dipping in the same direction as that of slope it may provide a condition favorable for instability. Such conditions may be verified by checking the orientation of the discontinuities with the slope direction. According to Markland, if kinematic conditions are satisfied the slope will have a chance to fail. The abutments do not satisfy these kinematic conditions and are more or less stable. The detailed characterization of the left abutment is presented in Table 4.2.

4.3 Rock mass Deformation

When a compressive load is applied to the rock mass it gets deformed. Deformation is change in shape. It may or may not be accompanied by change in volume. Deformation without the change in volume and which is restored on release of stress is called elastic deformation; whereas that which is not restored is called plastic deformation (Garg, 2005).

Table 4.2 RMR data collected from various locations on the Left Abutment

RMR data Points	**Parameters rating**									**RMR**	Rock mass class
	UCS	**Ra**	**RQD**			**SP**	**Con**	**GWC**	**Ori**		
			JV	**RQD**	**Ra**						
Left Abutment											
LA1	151.2	**12**	15	65.5	**13**	**8**	**20**	**15**	**-5**	**63**	Good
LA2	201.6	**12**	10	82	**17**	**8**	**25**	**15**	**-5**	**72**	Good
LA3	168	**12**	10	82	**17**	**10**	**20**	**15**	**-5**	**69**	Good
LA4	156	**12**	14	68.8	**13**	**15**	**20**	**15**	**-5**	**70**	Good

UCS – Unconfined Compressive Strength, **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

The RMR data that is presented in Table 4.2 is calculated by collecting the data from two to three observation points at each location (LA1 to LA4) and the values for various parameters presented in Table 4.2 are the average values for each respective locations.

If a dam is founded over various types of rocks having varying deformation properties, shear and diagonal tension stresses will develop due to the uneven deflection of the foundation. If such properties of the foundation rocks are known in advance, dam structure can be designed accordingly, to handle such deflections (USSD, 2011).

The empirical techniques to determine Modulus of Deformation ‘E_d’ are mainly based on rock mass classification system.

Bieniawiski (1979) proposed an empirical relation to determine Modulus of Deformation ‘E_d’ by using RMR. For the rock mass having RMR higher than 55 and UCS of intact rock ‘qc’ greater than 100Mpa the relation is in close agreement with the tested valves.

$E_d = 2RMR–100$ …………………*eq. 4.2*

Table 4.3 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' (Kg/cm^2)
			C (MPa)	Φ (°)	
Left Abutment					
LA1	2564	**63**	3.15	36.5	2.6×10^5
LA2	2574	**72**	3.6	41	4.4×10^5
LA3	2584	**69**	3.45	39.5	3.8×10^5
LA4	2596	**70**	3.5	40	4.0×10^5
Note ; C – Cohesion, Φ – angle of internal friction					

In Table 4.3 the shear strength parameters, cohesion and angle of internal friction were determined using the formula proposed by Bieniawiski (1979);

C = 0.05 RMR ……………………*eq. 4.3*

Φ = 0.5RMR + 5…………………*eq. 4.4*

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

As the abutments are formed from fragmented rocks, the failure criteria by Hoek and Brown (1981) can be applied to determine the strength of the jointed rock mass.

The following Hoek and Brown empirical formula is used to perform the analysis;

$\sigma_1' = \sigma_3' + \sigma_c\, [mb\ (\sigma_3/\ \sigma_c) + S]^{\,a}$…………………*eq. 4.5*

Where, `mb' 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 uni - axial compressive strength of the intact rock pieces and σ_1' & σ_3' are the axial and confining effective principal stresses, respectively.

The RMR data can be used to calculate the parameters 'm', 's' and 'a' as mentioned in eq.4.6, 4.7 and 4.8. The uni-axial compressive strength was calculated from the point load test. The value of material constant 'mi' has been directly adopted from the standard table proposed by Hoek and Brown (1981). The other material constant 'mb' and S were determined by using GSI (Geologic Strength Index) as;

Average RMR of Left Abutment = 68.5, mi = 17 (for Basalt),
Average GSI of the Left abutment = RMR_{89} – 5 = 68.5 – 5 = 63.5

$\frac{m_b}{m_i} = exp\left(\frac{GSI-100}{28-14D}\right)$ ....eq. 4.6

$S = exp\left(\frac{GSI-100}{9-3D}\right)$ ....eq. 4.7

a = 0.5 ....eq. 4.8

'D' is a factor which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses (Hoek et al., 2002).

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') is presented in Table 4.4 and Fig. 4.1.

In Table 4.4 the minor principal stress were presumed to calculation the major principal stress. The principal stresses that are plotted in Fig. 4.1 represent the average of the locations LA1, LA2, LA3 and LA4.

4.4 Permeability of the dam foundation

Permeability of rock and soil is the property by virtue of which it allows water to travel through its pore-spaces or other openings and is found to be proportional to the square of the diameter of the grains forming the rock. In gravels, permeability is high due to its large size. Sand is less permeable than gravel. Silt is still less permeable while clay, with its extremely fine grain-size, is practically impervious, although it may be porous to a considerable extent. A permeable rock is necessarily porous, but the converse may not always be true. A porous rock is permeable only if its pore spaces or other openings are interconnected and are of sufficiently large size to provide free movement of water (Mukerjee, 1995).

The permeability of the dam foundation is one of the most important conditions that must be studied so that the dam can function well without the substantial loss of water. The permeability of the proposed Berga dam foundation can be determined by using packer test.

Packer testing is carried out in either open boreholes or through the wire line drilling rods. The latter situation allows for the packer equipment to be used in unstable bore holes where unstable wall rock conditions would likely cause the tool to become jammed by falling rock or sand.

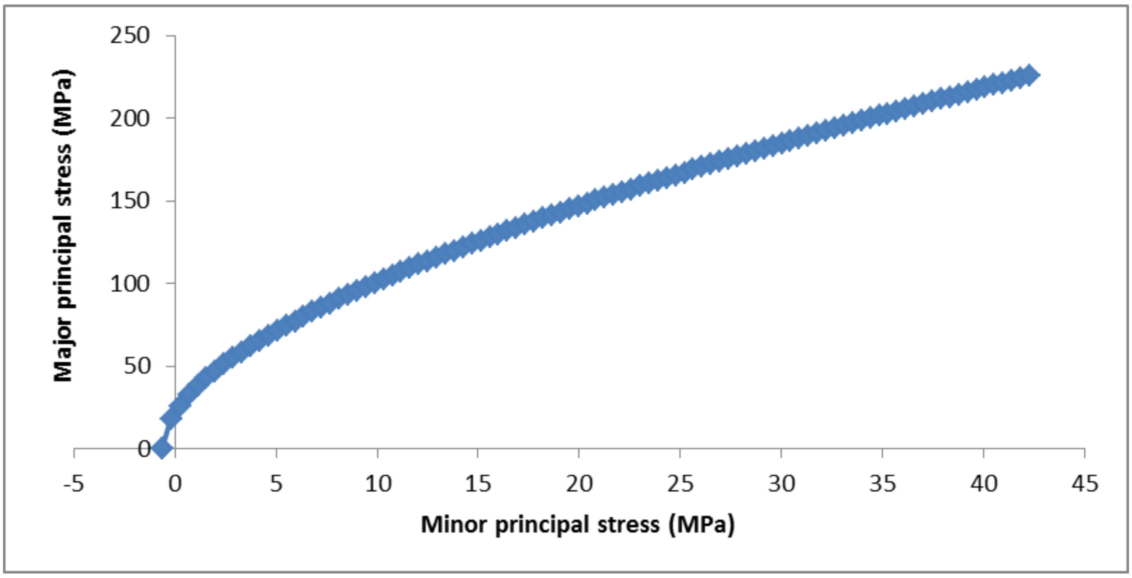


Fig.4.1 Average principal stresses for rocks exposed at left abutment of the dam site

Table 4.4 Major Effective Principal Stresses of Left Abutment as determined by Hoek and Brown Failure Criteria

RMR	UCS (Mpa)	Material Constants		Minor principal stress (σ_3)	Major principal stress (σ_1)	Shear strength parameter	
		mb	S			Cohesion (Mpa)	Angle of friction (°)
63	151.2	3.793	0.0094	1.0	30.76		
	151.2	3.793	0.0094	10.0	87.47	3.15	37.49
	151.2	3.793	0.0094	20.0	128.85		
72	201.6	2.231	0.0256	1.0	47.91		
	201.6	2.231	0.0256	5.0	86.51	3.94	40.16
	201.6	2.231	0.0256	12.0	129.48		
69	168	4.7	0.0183	1.0	37.92		
	168	4.7	0.0183	5.0	71.58	3.43	39.27
	168	4.7	0.0183	15.0	126.04		
70	156	4.871	0.0205	1.0	40.36	3.85	
	156	4.871	0.0205	5.0	70.39		39.57
	156	4.871	0.0205	10.0	101.18		
Average							
68.5	169.2	4.617	0.0173	1.11	37.86		
	169.2	4.617	0.0173	5.06	71.48	3.59	39.13
	169.2	4.617	0.0173	10.31	102.56		

It also allows for the drill rods to be used as the test water supply line, thus making it far easier to deal with the equipment involved in deep. By using packer test the permeability of the dam foundation can be determined so that it will help to determine whether the foundation needs grouting treatment or not (Royle, 2002).

As the proposed Berga dam foundation is covered by alluvial soil and fractured basalt, according to the geophysical exploration results (Yohannes Belete et al., 2009), there is a possibility of seepage even in small amount and by considering this condition it is recommended to perform the test at the dam foundation during the construction stage.

The other way is to conduct laboratory permeability test on samples from the foundation area. In general, in the present study area the soil at the dam foundation is classified as MH and according to the Indian Standard (IS) soil characteristics pertinent to embankments and foundations (Garg, 2005) it has a permeability of 10^{-4} to 10^{-6}cm/sec.

Based on the laboratory results, the soils in the foundation area were classified as moderate to highly plastic soils. These soils have less permeability because of their plastic nature. However, there is a small amount of sand. This sand content appears to be the cause for permeability of the soils even though the fine proportion is very large. The soil in the foundation area has a thickness of more than 3 m. This thickness may help to minimize the seepage through the dam foundation. According to the geophysical study conducted by Yohannes Belete et al. (2009), the soil in the foundation area is underlain by fractured basalt. This formation will be the major cause for the loss of water through the foundation area as a result of the fractures that it contains.

In the present study area both the dam abutments have thin soil cover and disintegrated rocks. Such condition may possibly help water to travel through the fractures within disintegrated rock mass and may result into development of pore water pressures. In general, based on the observations it is anticipated that the foundation of the proposed dam site may be susceptible to slight seepage. Thus, it is recommended to conduct more detailed studies pertaining to permeability of foundation and based on the results appropriate measures to improve the permeability may be carried out during construction stage.

4.4.1 Grouting of Rock Foundations

Grouting is a process by which fluid like materials, either in suspension or in solution form, is injected into the void spaces of the underground soil or rock, and is allowed to solidify. Such grouting will therefore help in reducing the void space, and hence increase the load carrying capacity of the soil, or to reduce the permeability, or both. A grout that primarily increases the load carrying capacity of the foundation soil is termed as consolidation grout; and that which reduces the permeability of the soil is called water-proofing grout (Garg, 2005).

According to Houlsby (1980), important factors in grouting are;

- The angle to the vertical at which the holes are drilled (to intersect the maximum number of open cracks).
- Spacing between adjacent grout holes.
- The pressure at which the grout is injected (too great a pressure may cause new cracks in the foundation rock).
- The thickness of the grout which is controlled by the ratio of water to cement in the grout.

The above mentioned techniques can be used during the construction stage so that the dam can have a suitable and less pervious dam foundation. The foundation of the proposed Berga dam, based on the geophysical investigation (Yohannes Belete et al., 2009), is composed of soil layer with fractured basalt layer. This sequence of lithology in the long run will provide an access to the impounded water to seep through the base of the dam and at the same time may develop pore water pressures which may result into foundation blowout. Therefore, conducting a controlled grouting will help to minimize this situation and possibly help to have a well stabilized dam structure.

Grouting technique, however, remains an expensive and highly specialized field. The injection technique and its control require a lot of experience and specialized skills (Garg, 2005). Trenching at the foundation area can also be used as a way to reduce the seepage of water.

4.5 Slope Stability of the Abutments

The abutments of a dam include that portion of the valley sides to which the ends of the dam joins and also those portions beyond the dam which might present seepage or stability problems affecting the dam. Right and left abutments are so designated looking in a downstream direction (Gedeon, 2004). The stability of the abutments is one of the basic criteria for the success of any dam structure. Detailed study must be carried on their stability, degree of weathering so that stripping can be done to rest the dam on a fresh and stable slope.

The design of a slope excavated in a rock mass requires as much information as possible on the character of the discontinuities within the rock mass, since its stability is frequently dependent on the nature of the discontinuities (Hoek and Bray, 1981). Information relating to the spatial relationships between discontinuities affords some indication of the modes of failure that may occur. Information relating to the shear strength of the rock mass or, more particularly, the shear strength along discontinuities, is required for use in a stability analysis

(Bye and Bell, 2001). The inclination of discontinuities is always the most important parameter for slopes of medium and large height (Bell, 2007).

During the present work both the left and right abutments of the proposed Berga dam were studied. However, the right abutment (Plate 4.4) was highly weathered and it was impossible to measure the orientation of the joints, as the exposed rocks were highly disintegrated and appeared to be separated blocks. Therefore, during the present study left abutment slope stability studies were carried out for plane and wedge mode of failure whereas, right abutment was considered for rotational mode of failure, as the rock mass on right abutment was highly disintegrated.

Plate 4.4 Highly fractured and disintegrated rock mass on right abutment

4.5.1 Discontinuity Analysis

Both the left and the right abutments contain highly disintegrated basalt but the left abutment has better exposure to collect data. The data pertaining to discontinuity orientations on the left abutment was collected to determine the type of the possible slope failure that may occur. The discontinuity data was stereographically analysed and checked for potential mode of failure (Table 4.5 and Fig. 4.2).

Table 4.5 Preferred orientations on the left abutment

Preferred orientation of discontinuity planes Dip direction/ Dip amount			
Slope	**J1**	**J2**	**J3**
N270°/10°	N290°/44°	N018°/66°	N206°/18°

Even if the joints and the discontinuities have preferred orientations, kinematic check must be performed. This is because different types of slope failure are associated with different geological structures and it is important to determine the potential stability problems during the early stage of project. According to Markland, rock failure only occurs if the following conditions are satisfied;

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

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

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 and' ϕ 'is the angle of internal friction of the two wedges forming plane.

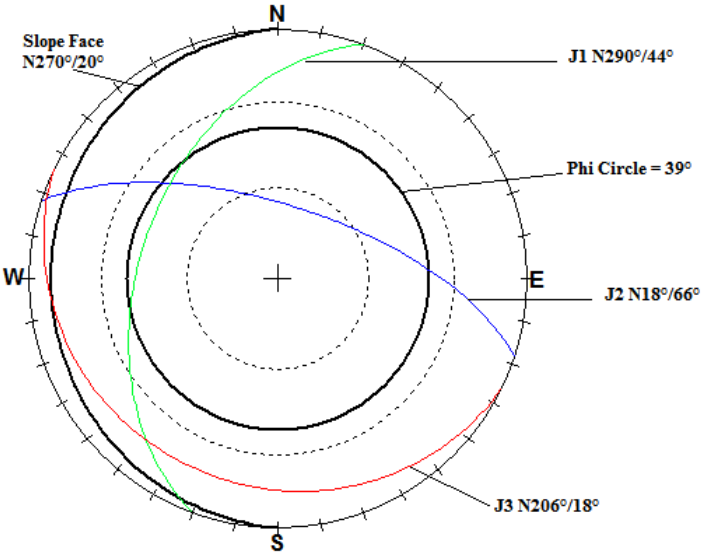


Fig.4.2 Kinematic check for potential mode of failure in right and left abutment slopes

By considering Markland's criteria, the left abutment does not satisfy the conditions for neither plane nor wedge mode of failure.

In the case of right abutment analysis for potential rotational mode of failure was performed by using SARC software. The input parameters used for this analysis and the results obtained are presented in Table 4.6 and Table 4.7, respectively.

For N, eight coordinates were used as it can be seen from Fig. 4.3 (right abutment with coordinate). The cohesion and angle of internal friction values are the average value of the soils in the study area. Unit weight of the soil was taken from the laboratory analysis conducted during the present study. The earthquake magnitude was assumed to be 8 in Richter scale (as the area is near the rift valley such magnitude earthquake have a greater chance to occur).

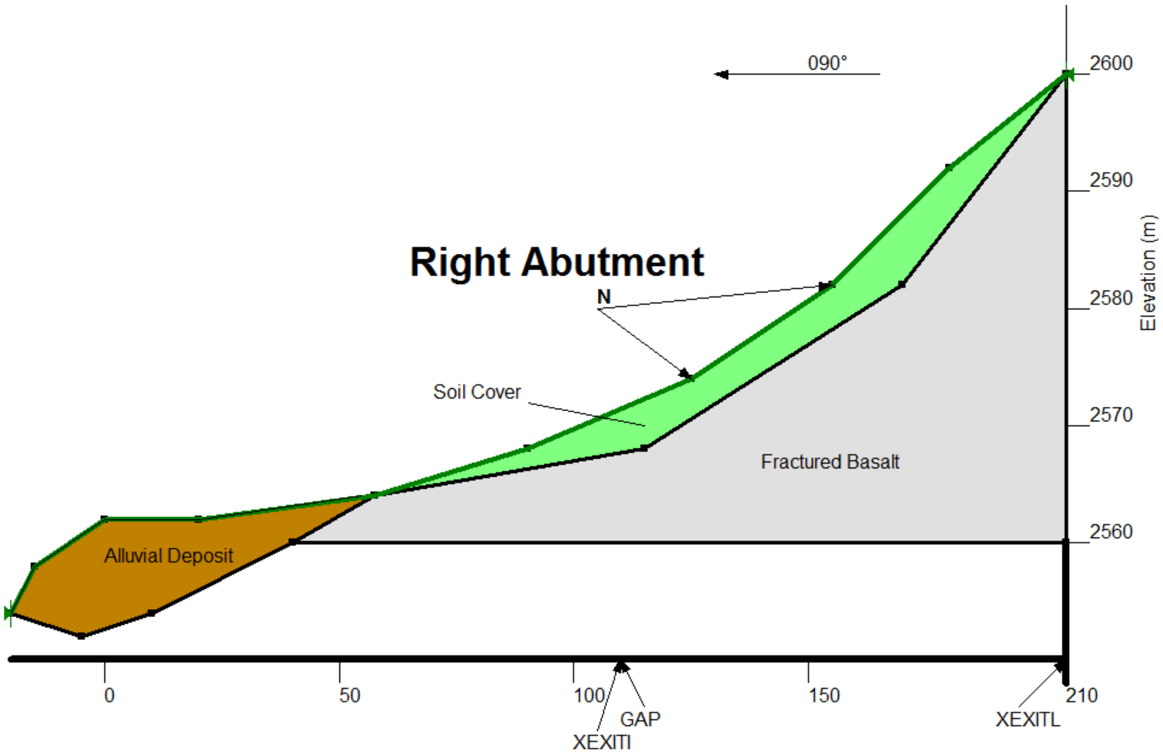


Fig. 4.3 Right Abutment with coordinates used in Analysis

Table 4.6 Input parameters used to calculate factor of safety for right abutment

Input Parameters		
Parameter	Value	Parameter description
N	8	N = Number of profile coordinates(< 50)
X(I),Z(I) I =	1 to N	(X,Z) = Coordinates of profile points (X(I) < X(I+1))
C =	11	C = Cohesion of soil/rock
PHI =	30	PHI = Angle of internal friction (Degree) of soil/ rock
GAMA =	1.96	GAMA = Unit weight of soil/rock
GAMAW =	1.0	GAMAW = Unit weight of pore water
BBAR =		BBAR = Pore water pressure/(GAMA * Average height of slices)
AH =	0.2	AH = Horizontal component of EQ. acceleration near crest of slope
AVR =	1.0	AVR = Vertical component of EQ. acceleration / AH
EQM =	8	EQM = Corresponding EQ. magnitude on Richter Scale
NENP = 1		NENP = Number of entry points of slip circles (<10)
ENTX (I), ENTY (I) =		ENTX = X-Coordinate of entry point of circle
(0.0,2.0)		ENTY = Y-Coordinate of entry point of circle
NEP = 0.0		NOPT = 0 ,When only minimum factor of safety is required
NOPT = 0.0		= 1,When all F.S. corresponding to all exit points are also required
XEXITI =	105	NEP = Number of exit points(<50)
XEXITL =	210	= 0 , When no individual point is given)
GAP =	105	XEXITI= X-Coordinate of first exit point of circle
		XEXITL= X-Coordinate of last exit point of circle
		GAP = Horizontal distance between consecutive exit points
		XEXIT = X-Coordinate of exit point of circle

Table 4.7 Result of analysis for right abutment

Anticipated Conditions	FOS	Weight	Coordinate of center	Exist point	Radius
Static dry	5.5905	$.94x10^4$	85.04,2691.28	210,2600	154.74
Static moderately Saturated	2.7309	$.34x10^5$	93.50,2644.56	210,2600	124.73
Static Fully Saturated	1.2401	$.68x10^4$	73.71,2753.93	210,2600	205.60
Dynamic Dry	1.8533	$.73x10^4$	79.61,2721.30	210,2600	178.09
Dynamic Moderately Saturated	0.8283	$.53x10^4$	68.85,2780.78	210,2600	229.35
Dynamic Fully Saturated	0.4976	$.32x10^4$	54.48,2860.19	210,2600	303.13
FOS = Factor of Safety,					

Based on the results (Table 4.7) the slope of the right abutment is stable during static condition in dry state and moderately saturated state. It is also stable in dynamic dry condition. For other anticipated conditions the slope will be unstable. This indicates the need for the removal of the soil cover from the slope so that the abutment will be stable during and after the construction of the dam for all anticipated conditions.

4.6 Engineering Geological Characterization and Properties of Soils

Soils are the earthy materials (weathered rocks), inorganic (such as sand, clay, silt etc.) or organic (such as peat), in nature. A soil is thus a natural aggregate of mineral grains, which are separable by gentle mechanical or chemical means. For engineering purpose, soils are classified based on certain criteria. Among these criteria, size of particles (grain size) and plasticity of soil are widely used (Garg, 2005). On the bases of their origin, soils can also be classified as either residual soils or transported soils (alluvial, lacustrine, marine, aeolin or glacial) (Garg, 2005). For the present study two soil samples were analyzed for engineering geological characterization of the foundation and one sample result was used from the previous study (Table 4.8).

Table 4.8 Summary of field and laboratory test results of soil samples of foundation area

Area	Sample No.	Grain Size				Atterberg Limit			Free Swell	UCS	Classification	
		Gravel	Sand	Silt	Clay	LL	PL	PI			USCS	Genesis
Dam Foundation	BD - 5	-	2.5	59.5	38	-	-	-	-	-	***MH**	**Alluvial**
	BD - 6	-	-	-	-	52	37.7	14.3	-	-	**MH**	
	ATP – 2A	-	0.7	67.5	31.8	48	34	14	40	5.5	ML	
Note: * By refereeing BD – 6ATP = Previous Work Data, BD – Present Work Data												

4.6.1 Grain size

Based on the results of seive and pipette analysis, the percentage of gravel, sand, silt and clay were determined. The areas that were selected for core material has more than 80% of the sample containing both silt and clay size particles. On the other hand the area selected for filter material (Area – 2) contains about 50% of sand and gravel size particles. The gradiation curve of the samples is presented in Fig. 4.4.

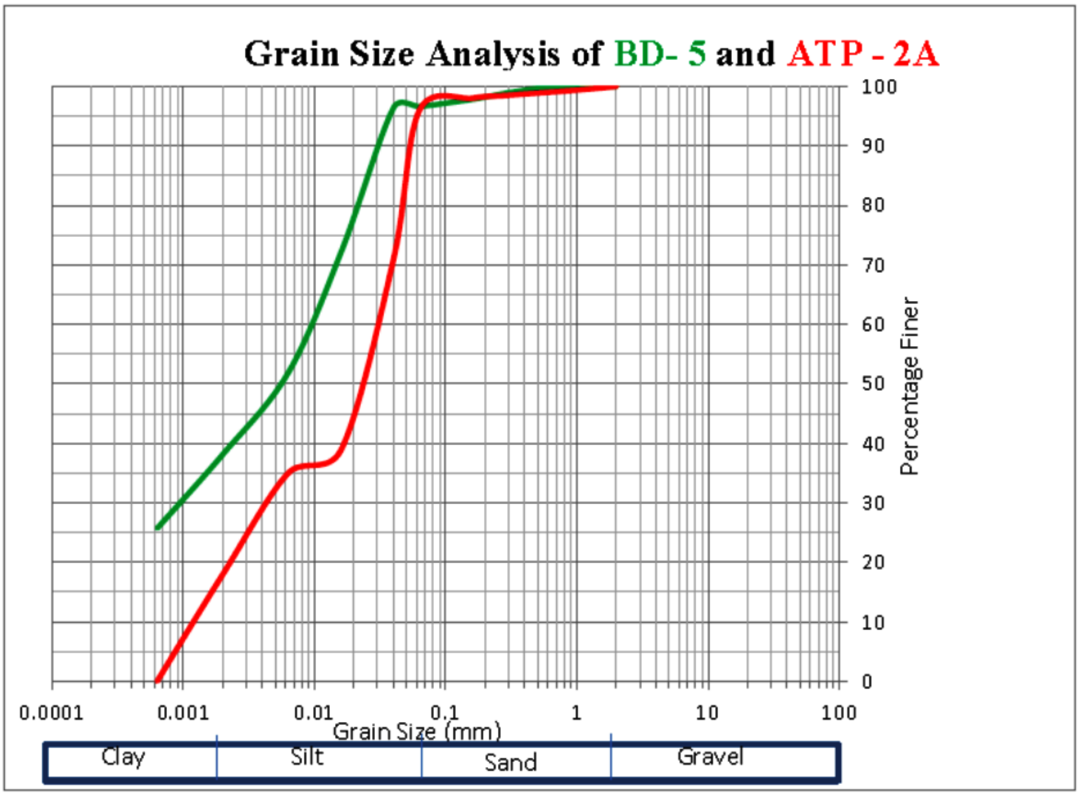


Fig. 4.4 Grain Size Analysis of the Dam foundation

4.6.2 Atterburg's Limits

The range of water content in which the soil behaves like a plastic material, that is Liquid limit (LL) – Plastic limit (PL) is known as the plasticity index (PI) of the soil. Soils with a high plasticity index are called plastic soils, because they behave like plastic material for a large range of water contents. These limits are generally called the Atterberg's limits. This classification is used for fine grained soils (Garg, 2005).

Soil Samples from the proposed dam foundation area were analysed to determine the Atterburg limits. The soils have liquid limit of about 50 and plastic index of 14 (Fig. 4.5). The soils of the study area have the free swell value of 40%.

In Table 4.8 the samples BD – 5 and BD – 6 were collected from the same area. Both werecollected to make different analysis. As it issummarized in Table 4.8 the soilsofthe study area were classified based on USCS (Unified Soil Classification System) as ML andMH. In order to classify the soils priveous data were used. In the present study additional samples were collected and submitted to Geological Survey of Ethiopia Laboratory for different analysis. According to Indian Standard forEmbankments and foundations, the soils, BD – 6 and ATP – 2A, have values of fair with no requirements for seepage control.

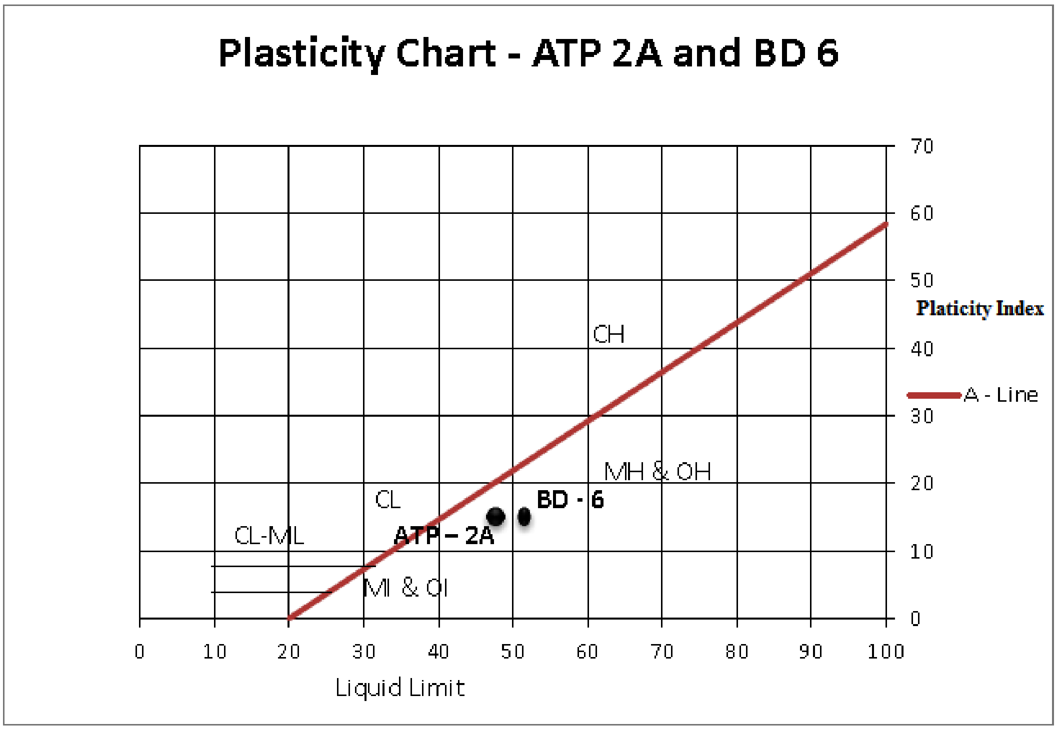


Fig.4.5 Plasticity Chart Plotting of Foundation Soil

4.7 Liquefaction of foundation soils

The term liquefaction is used to describe large deformations occurring in a soil mass caused by monotonic, transient disturbance of saturated cohesion less soil (Mogami, 1953). In the evaluation of the liquefaction hazard it is required to (a) determine the soil susceptibility to liquefaction, (b) determine the potential for liquefaction to be triggered, and (c) estimate the extent of damage (Fernandez, 2009).

Liquefaction may present itself in different forms and magnitudes and it requires a detailed investigation, from susceptibility to hazard potential, in order to elaborate an optimal remediation approach. Such investigation involves field activities, analysis, testing, and interaction with regulators and the community.Therefore, at the dam foundation, penetration resistance test must be conducted to know the ground acceleration by using Liquefaction potential versus saturated penetration resistance no (Seeds and Idress, 1971).

The Berga proposed dam site is located near the margins of the main Ethiopian rift. As this rift is very active there is a high potential for the dam to be exposed for different magnitude of earth quakes. The liquefaction potential of the foundation soils can be analysed by using Day (2006) criteria. According to this criterion for a soil to be susceptible to liquefaction, it has to have 0.005mm grain size <15% and liquid limit less than 35. The foundation soils of the dam site have 0% grain size of 0.005mm and the liquid limit is 43. Thus, according to Day (2006) criteria it may be concluded that foundation soils may not have likely liquefaction criteria. However, further in situ test may be performed such as Standard Penetration Test (SPT) to determine the relative density of foundation soils. Generally, soils with SPT-*N*-values of 0 to 10 may suffer severe damage due to liquefaction whilst those of 20 to 40 may not suffer and it is frequently found that the lower the relative density of the silt or fine sand the greater is its likelihood to liquefy (Seed and Idriss, 1971).

4.8 Engineering geological maps

4.8.1 Regional Engineering Geological Map

Engineering geological maps and plans provide engineers and planners with information that will assist them in the planning of land use and for the location and construction of engineering structures of all types. Such maps usually are produced on the scale of 1:10,000 or smaller, whereas engineering geological plans, being produced for a particular engineering purpose, have a larger scale. Engineering geological maps may serve a special purpose or a multipurpose (Anon, 1976). Special-purpose maps provide information on one specific aspect of engineering geology, for example, the engineering geological conditions at a dam site or along a route way or for zoning for land use in urban development. Multipurpose maps cover various aspects of engineering geology. Engineering geological maps should be accompanied by cross sections, and explanatory texts and legends. Detailed engineering geological information can be given, in tabular form, on the reverse side of the map (Bell, 2007).

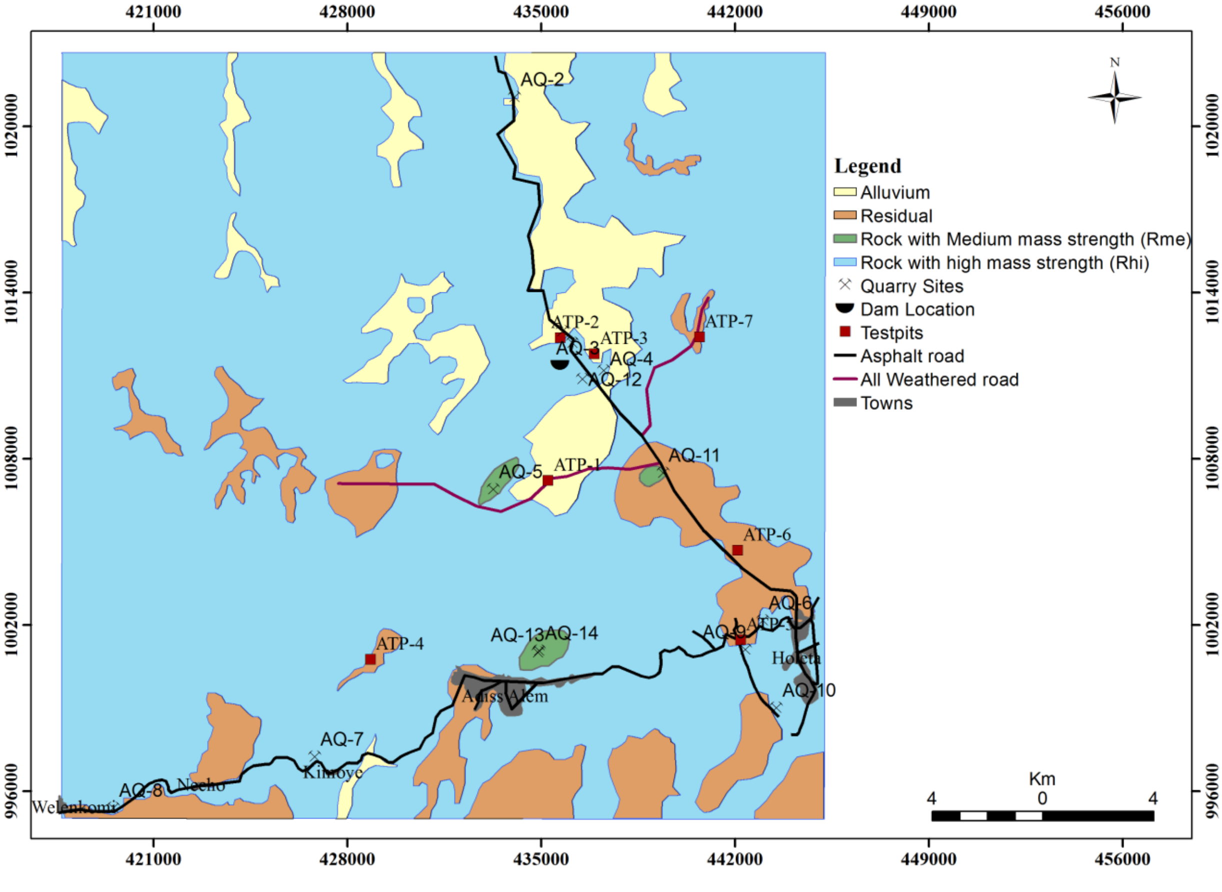
Based on field investigation coupled with laboratory testing the various lithological formations of the Addis-Alem sub sheet was prepared and presented (Fig. 4.6).The Addis-Alem sub sheet is classified in to the following engineering geological units (Yohannes Belete et al., 2009);

1. Engineering Geological Rock Units

<u>Rock with high rock mass strength (Rhi)</u> - There are two types of basalts out cropping in Addis-Alem sub-sheet, these are the lower basalt which covers the southern part of the sub sheet and the upper basalt which covers the northern part of the sub sheet. The lower basalt has aphanitic texture. The rock material strength of the lower basalt ranges from 4.44 -8.4 MPa and has a joint spacing of 0.5 to 1m.

Rock with medium rock mass strength (Rme) –this is composed of Ignimbrite outcrops of the area. The ignimbrite is columnar jointed with joint spacing from 30 cm to 60 cm. The point load test result of ignimbrite ranges from 1.8 to 4.3 MPa which indicates the rock has medium mass strength. The calculated un-confined compressive strength of ignimbrite ranges from 43.2 to 106.32 MPa and the laboratory result of UCS ranges from 68 to 87 MPa, therefore it is classified as medium rock mass strength.

The water absorption result of ignimbrite ranges from 6.02% to 10.65%, it means ignimbrite due to its high water absorption capacity it cannot be used as aggregate.



(Source: After Yohannes Belete et.al. 2009)

Fig.4.6 Engineering Geological Map of Addis Alem map Sheet

2. Engineering Geological Soil Units

Alluvial soil - Alluvial soils are deposited in the central parts of Addis-Alem sub-sheet along Berga, Borche and Bora streams. The soils in general can be described as, slightly to highly firm, moderately plastic and slightly wet.

Residual soil - In the study area residual soil occurs in the central upper flat plateaus to gently sloping and low relief areas of basalt that is mainly found south of Adiss-Alem and Holeta towns as well as in Telecha locality. As observed from the test pits, the residual soil is brown clay soil, firm, slightly to moderately plastic and dry to slightly wet with thickness over 3 m.

4.8.2 Engineering Geological Map of the Dam site

In the present work, the engineering geological map of the dam site (Fig. 4.8) was prepared by using the rock mass classification which includes rock quality designation (RQD), spacing and condition of discontinuities and degree of weathering. The soils of the area were classified based on their origin and soil mass classification system.

Soil Samples collected (in the previous and present studies) from the borrow areas were disturbed. They were taken from surface exposure, river cuts and pits that were dug inside the study area. The depth range of the samples taken was from 0.5 m to 3 m. All the samples were either alluvial or residual as per their origin. These samples were collected to determine different engineering properties (grain size, Atterberg limits, and free swell) of the soil deposits that are proposed for the construction of the Berga dam. The samples have colours from reddish light brown to dark brown. They were silty clay (the core materials), slightly firm, moderate to high plastic and slightly wet. The samples for the filter material were brown in colour, contains more sand sized particles and they were slightly wet. According to the measurement taken by pocket penetrometer (Yohannes Belete et al., 2009), the soils have unconfined compressive strength (UCS) between 3.5 kg/cm^2 to 13 kg/cm^2.

The samples that were collected for the present study are presented in Fig 4.7 and Table 4.9.The view and the cross section of the abutments are shown in Plate 4.5 and Fig. 4.9.

For mapping purpose the rocks of the dam site were classified and used accordingly. To determine the strength of the rocks point load data was used and this data was converted to the unconfined compressive strength. The weathering conditions and the ground water conditions were characterized visually. Finally, the RMR values of the rocks were determined which falls in good categories based on Bieniawiski's (1989) Geomechanics classification system.

The soils of the dam site were classified according to their origin and soil mass classification system and can be classified as alluvial soils and based on Soil classification system as MH.

The dam site is composed of three engineering geological units; fractured basalt, inorganic clay and inorganic silt.

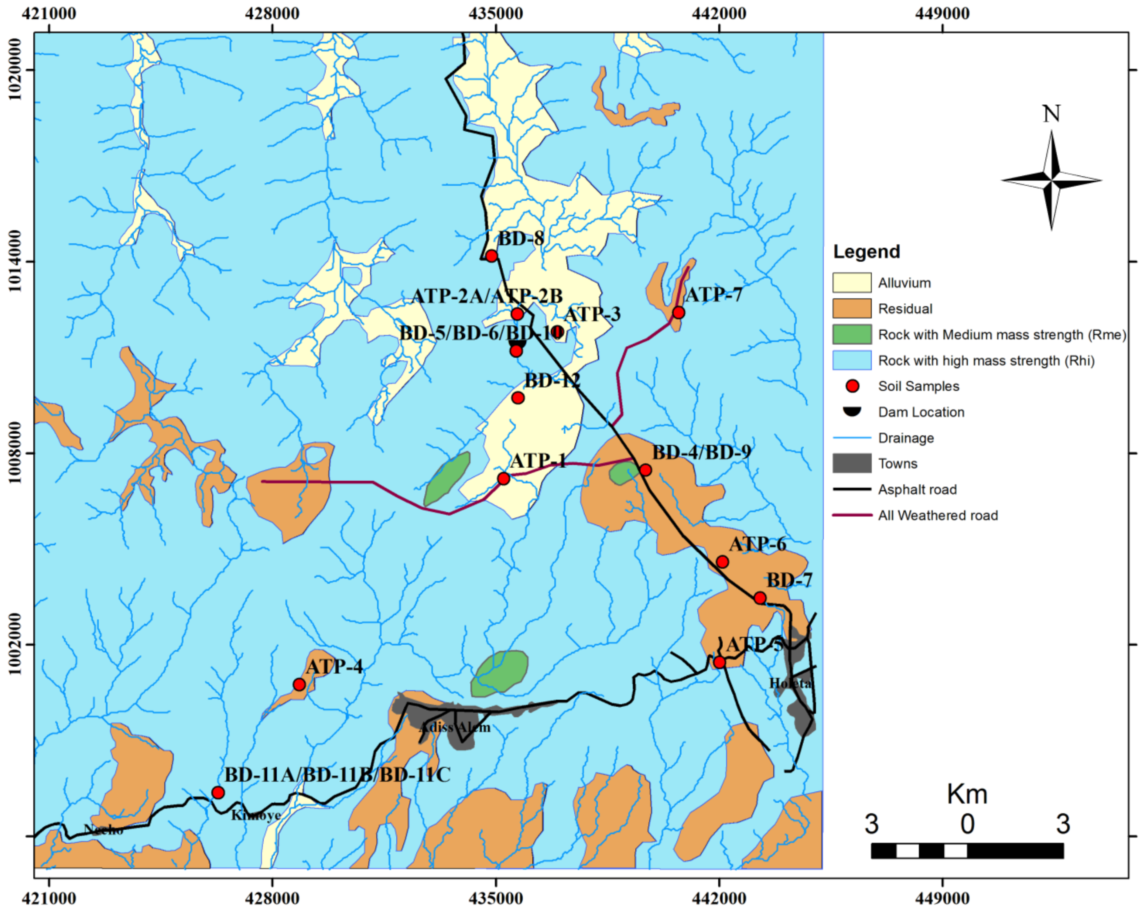


Fig. 4.7 Sample locations of the Study Area

Table 4.9 Sample Locations

No.	Samples Name	Location			UCS (kg/cm^2)	Soil Type (Genesis)	USCS
		Easting	Northing	Elevation			
1	ATP – 1	435254	1007215	2549	10.5	Alluvial	
2	ATP – 2	435691	1012372	2562	5.5	Alluvial	ML
3	ATP – 3	436925	1011795	2580	3.5	Alluvial	CH
4	ATP – 4	428837	1000762	2300	7	Residual	MH
5	ATP – 5	442009	1001460	2410	13	Residual	CH
6	ATP – 6	442098	1004597	2484	10	Residual	ML
7	ATP – 7	440734	1012404	2735	10	Residual	MH
8	BD – 4	439689	1007464	2565	-	Residual	ML
9	BD – 5	435628	1011210	2552	-	Alluvial	MH
10	BD – 6	435628	1011210	2552	-	Alluvial	MH
11	BD – 7	443279	1003469	2457	-	Residual	-
12	BD – 8	434864	1014173	2622	-	Residual	-
13	BD – 9	439689	1007464	2565	-	Alluvial	-
14	BD – 10	435628	1011210	2552	-	Alluvial	-
15	BD – 11	426289	997377	2166	-	Alluvial	SM
16	BD – 12	435686	1009726	2556	-	Alluvial	-
UCS = unconfined compressive strength. This value is taken from Yohannes Belete et al. (2009) which were measured by using pocket penetrometer.							

Plate 4.5 View from the Right to Left Abutment

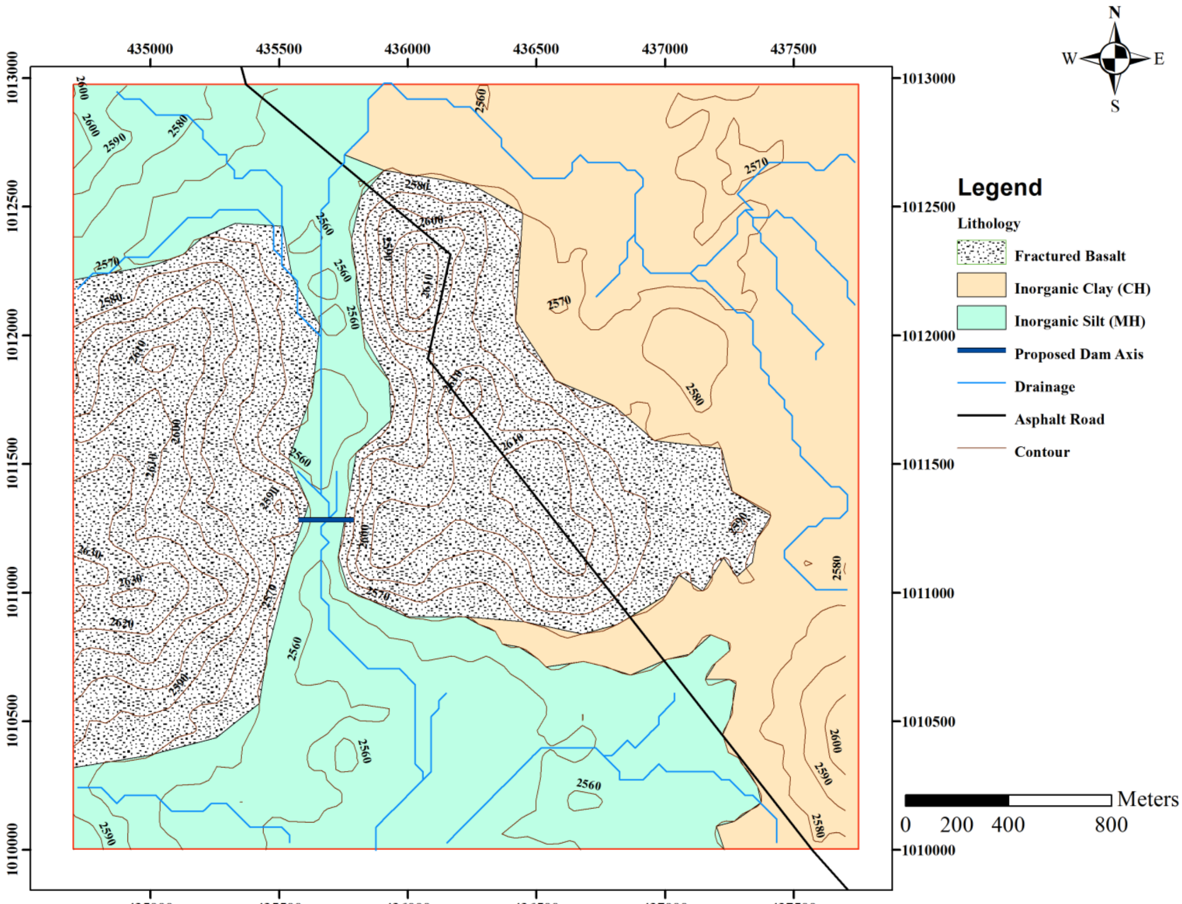


Fig.4.8 Engineering Geological map of the dam site.

1. Rock Unit

Fractured basalt - This unit is found in both left and right abutments. It is highly fractured and is covered with thin soil cover. Since, the rock unit is basalt it is expected that it may possess a high bearing capacity after removing the top soil and the disintegrated rock fragments. At present the exposed rock mass is highly disintegrated and manifest many fractures. It is most likely that if the proper treatment for this disintegrated rock mass is not undertaken it may provide most favourable condition for seepage to varied degree which ultimately may affect the stability of the structure. Therefore, it is required to strip off the soil cover and the disintegrated rock mass to the reasonable depth. Based on the exposed rock mass condition (appeared after stripping) proper remedial measures needs to be evolved. It is most likely that stripping of undesirable material from abutments will eventually make the slope gentler and the section for dam will become wider. Thus, it may require design to be addressed in a way that whatever considerations are made on this account should be techno-economically feasible.

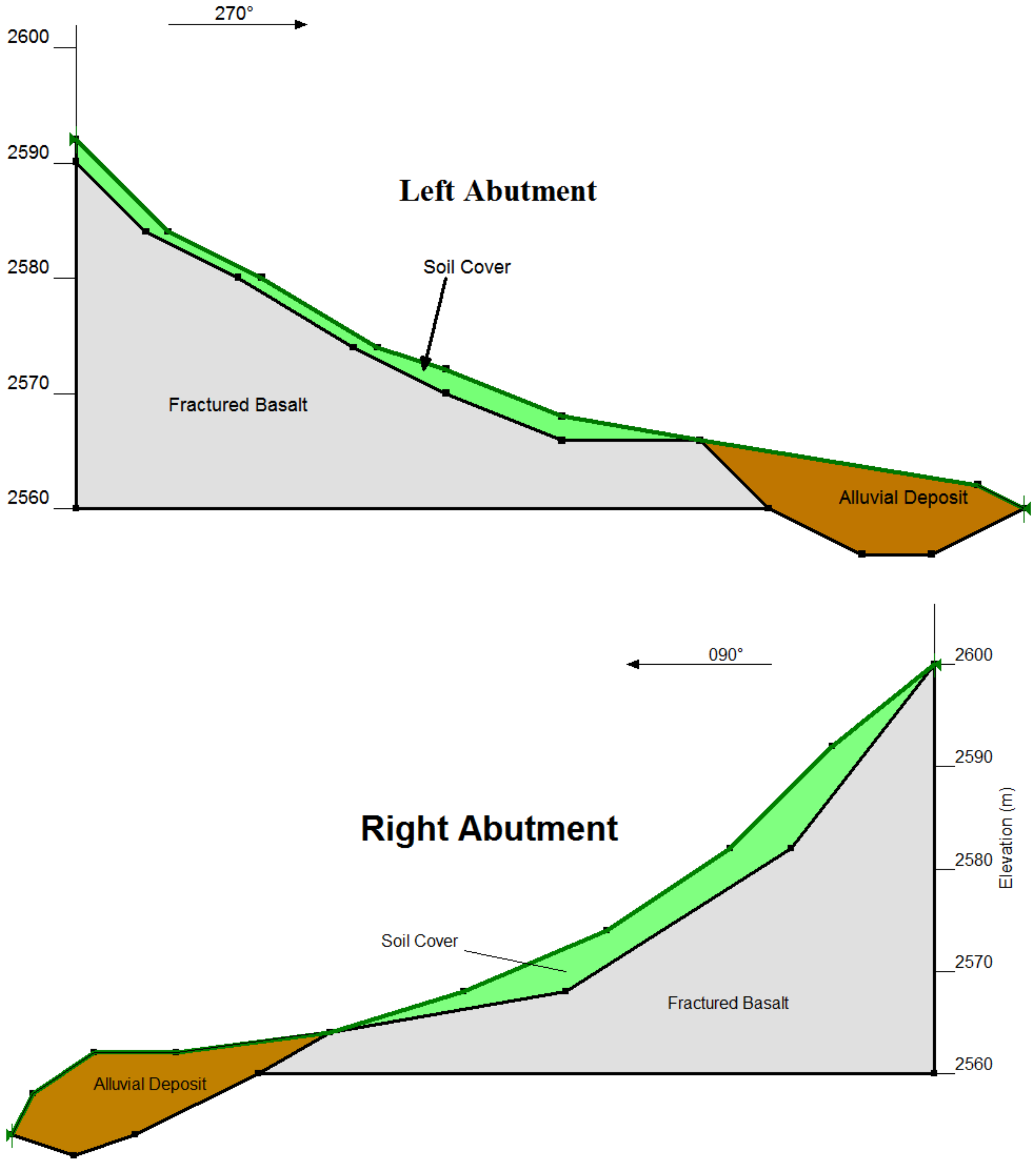


Fig.4.9 Cross Sections of abutments along the dam axis

2. Soil Units

Inorganic clay- This unit is found in the eastern parts of the proposed dam site. According to Indian standard (IS: 1498-1970) this soil type has fair to poor bearing capacity as foundation material. It generally does not require seepage control methods as it is highly impervious. It is highly compressible and also expansive. Generally, this unit is fairly suitable for embankment construction with flat slopes with thin cores.

Inorganic silt- This unit is found in the central parts of the proposed dam site. Such soils possess poor bearing capacity and have poor stability. It can be used to construct cores of hydraulic fill dam but it is not desirable in rolled fill construction. It does not need seepage control methods like the inorganic clay (IS: 1498-1970).

The cross section of the abutments (Fig. 4.9) shows the fractured basalt which is overlain by soil cover. From the eroded parts of the soil cover it was found that it has a thickness up to 1 m (around the top). The soil cover is composed of soil with basaltic rock fragments with varied dimensions ranging in size up to 30 cm.

In terms of bearing capacity, both the abutments have high capacity as they are composed of very strong basalt. Based on its RMR value, especially on the left abutment, the rock is classified as 'good' which makes it suitable to be used as a foundation material for the proposed dam. The seepage potential of the abutments is generally low except where they are weathered and fractured.

4.9 Overall Engineering Geological Appraisal of Dam Site

Engineering geological characterization of dam site is essential in order to have a safe and stable dam structure. With this in mind, in the present study an attempt was made to characterize the dam site area and to check its suitability for dam construction. This characterization focuses on; the classification of rock mass, deformability of rock mass, permeability of dam foundation, slope stability of abutments, engineering geological characterization of soils, determining liquefaction potential of the soils and preparation of the engineering geological map for the dam site.

According to the RMR classification of the left abutment, the slope of the left abutment can generally be considered as suitable to be used as a foundation for the proposed site. The analysis of the deformability of the rock mass in the dam site is determined by using the

RMR value. The permeability of the dam site was determined by using the Indian standard and the soils have in general value of 10^{-4} to 10^{-6} cm/sec. in the case of the abutments they are covered by soil and rock fragments with discontinuities. This condition will facilitate the seepage of water from the reservoir during the full function of the dam. In general, the site is susceptible to slight seepage and during the construction stage, striping of the abutments slopes and grouting must be performed where ever it is necessary.

Slope stability analysis was performed for both left and right abutment. The results of the analysis show that the left abutment is not susceptible to plane nor wedge mode of failure whereas the right abutment may becomes unstable during anticipated static and dynamic conditions. The soils of the dam site are characterized by using the USCS. According to this classification, the soils are MH and CH around the dam axis and in the surround area, respectively. Atterberg’s limits and grain size analysis is used for classification. The liquefaction potential of the foundation was analyzed by using Day (2006) criteria. According to this criterion for a soil to be susceptible to liquefaction, it has to have 0.005mm grain size <15% and liquid limit less than 35. The foundation soils of the dam site have 0% grain size of 0.005mm and liquid limit of 43. Based on this the soil in the foundation area is slightly susceptible to liquefaction as is contains no 0.005 mm size.

Finally, the engineering geological map of the dam site was prepared. This map shows the best suitable axis for the dam with engineering properties of the soil and rock type available in the area. In conclusion the site is very suitable for dam construction with few remedial measures and the construction of the proposed Berga dam is highly recommended in the area.

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Chapter 5 Characterization and Suitability Analysis of Embankment material

5.1 Introduction

Embankment dams are constructed of all types of geologic materials, with the exception of organic soils and peats. Most embankments are designed to utilize the economically available on-site materials for the bulk of construction. Special zones such as filters, drains and riprap, may come from off-site sources. Soil materials used in embankment dams commonly are obtained by mass production from local borrow pits, and from required excavations where suitable (USSD, 2011).

One of the most commonly used materials for engineering purposes are inorganic soils. Inorganic soils generally are divided into two broad categories for engineering purposes: fine grained soils and coarse-grained soils. Many embankments are constructed of broadly graded soils which do not fit entirely into either category or exhibit characteristics of both. The principal characteristics that distinguish fine grained soils from coarse grained soils for the purposes of embankment dam design are that fine grained soils have lower permeability, lower shear strength, and higher compressibility (USSD, 2011).

Soil plasticity serves as an initial indicator of the potential behavior of a clay or silt when placed in an embankment dam. Clean sands and gravels, meaning sands and gravels that have less than about 5 % fines by dry weight are pervious, easy to compact, and are minimally affected by changes in moisture content. The important properties of interest in embankment dam engineering are namely; shear strength, compressibility and permeability. These properties are determined by the gradation, grain size and shape, relative density, and durability of the coarse grained soil. Compressibility is generally of less concern, as these soils are essentially incompressible when compacted to a dense state (USSD, 2011).

With regards to economical view point, all the materials that are being used for the construction of the embankment dam must be located near to the dam site as relatively large quantities of construction materials would be required to finish the structure. Further, the 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 material identified for the construction of the embankment should be examined for existing and anticipated adverse conditions to which these would be subjected after placement in the embankment (USSD, 2011).

5.1.1 Construction Material Sites for Berga Dam

During the engineering geological mapping by Geological Survey of Ethiopia (GSE) (Yohannes Belete et al., 2009), the Berga dam site was proposed for an embankment dam with sufficient amount of suitable construction materials. The borrow areas and the quarry sites were investigated and different analysis was carried out. During study conducted by GSE6 borrow sites were selected for brown clay soil that could be used for construction of the proposed earth dam (Fig. 5.1).

The estimated available quantity of brown clay soil is about 4,837,500 m^3. One site with sand and gravel deposit, at the downstream of the Berga river, was located which can be used as a filter material (Yohannes Belete et al., 2009).

The salient features of proposed Berga dam is presented in Table 5.1.

Table 5.1 Salient features for Proposed Berga Dam Project

Dam Height	40m
Reservoir Area	0.3 km^2
Catchment Area	72.24 km^2
Crest Length	250m
Available Irrigable Land (Command Area)	3000 Hectares

(Source: Yohannes Belete etal. 2009)

In the present study previously selected borrow and quarry areas by GSE were further studied for the suitability of the selected materials to be utilized for the various zones of the proposed embankment dam. Additional samples were collected from the borrow areas and appropriate laboratory test were performed to determine grain size distribution, Atterberg limits, moisture content and dry density of the soil.

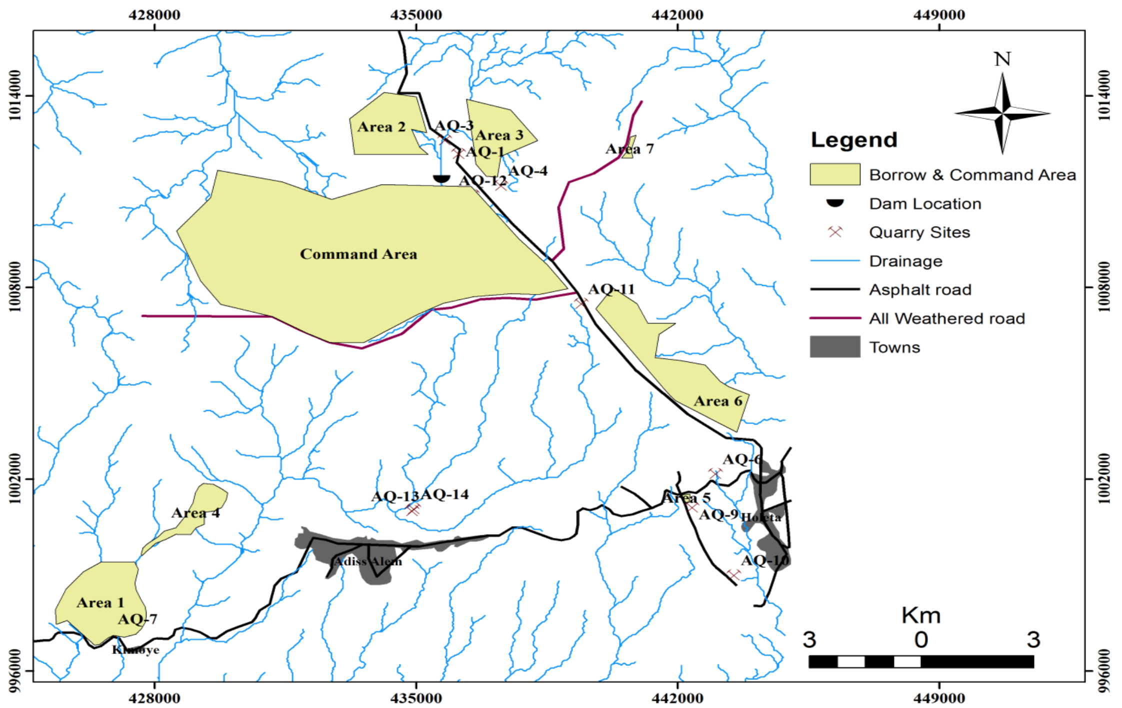
In total 14 quarry sites (Table 5.2) were described in the previous study by GSE (Yohannes Belete et al., 2009). These quarry sites can be used as a source for riprap material for the upstream slope protection and can also be utilized for slope protection material for downstream slope of the embankment dam.

Table 5.2 Quarry Sites

Quarry	Location	Estimated	Type of	Overburden	Weathering	Joint sets	strength	Excavability

No.	Easting	Northing	Volume (m^3)	Material	thickness				
AQ1	436142	1012190	2,000,000	Basalt	Very thin soil cover	Slightly Weathered	Massive but slightly fractured	Strong	Good
AQ2	434053	1021078	8,000,000	Basalt	Very thin soil layer (0.1m)	Slightly to moderately weathered	Massive with no clear joint	Strong	Good
AQ3	435763	1012616	100,000	Basalt	No overburden	Moderately weathered	No joint sets	Strong	Good
AQ4	437275	1011197	200,000	Basalt	No overburden	Fresh to slightly weathered	No joint sets	Strong	Good
AQ5	433274	1006911	7,500	Ignimbrite	No overburden	Slightly Weathered	No joint sets	Moderate strong	Good
AQ6	443016	1002176	50,000	Basalt	Very thin soil layer (0.1m)	Fresh to slightly weathered	2 joint sets	Strong	Good
AQ7	426825	997250		Basalt	No overburden	Moderately weathered	Vertical Joints	Moderate strong	Good
AQ8	419559	995400	2,400,000	Basalt	No overburden	Moderately weathered	Massive	Moderate strong	Good
AQ9	442384	1001126	2,500,000	Basalt	Thin soil layer (0.5m)	Fresh to slightly weathered	No joint sets	Very strong	Fairly Good
AQ10	443509	999012	5,000,000	Basalt	0.1 to 0.5m top soil	Fresh to slightly weathered	No joint sets	Very strong	Fairly Good
AQ11	439417	1007500	500,000	Ignimbrite	0.5m top soil	Slightly weathered	Columnar jointing	Moderate strong	Fairly Good
AQ12	436506	1010894	1,200,000	Basalt	No overburden	Slightly to moderately weathered	No joint sets	Moderate strong	Good
AQ13	434893	1001017	50,000	Ignimbrite	0.5m top soil	Slightly to moderately weathered	2 joint sets	Moderately strong	Good
AQ14	434933	1001078	100,000	Ignimbrite	0.3m top soil	Slightly to moderately weathered	2 joint sets	Moderately strong	Good

(Source: Yohannes Belete et al.2009)



(Source: Yohannes Belete et al., 2009)

Fig. 5.1 Command area and borrow areas

Borrow Area-1 is the only proposed source for the filter and shell materials whereas from Borrow Area-2 to Area-7 the soil as a source for the clay material may be utilized. Quarry sites AQ-1 to AQ-14 are the proposed sources for the rip rap material.

In the previous work (GSE), one additional borrow area was selected for clay source material inside the proposed command area. However, in the present work an alternative area was added from the proposed catchment area (Plate 5.1) around Area 3. The detail description of the borrow areas is presented in Table 5.3;

Table 5.3 Potential material source sites

Area	Location	Areal coverage (m^2)	Distance from Dam site (Km)	Description of the available material
Area – 1	426289E 997377N	4431419.58	30	The area is flat land with river cut. The material is covered by 0.5 m thick top soil. It is composed of different materials which ranges from pebbles to sand. This material can be used as both filter and shell material with treatment and it is the only place for the shell and filter material in more or less economic distance.
Area – 2	435691E 1012372N	3130257.96	1.1	This material is found in almost completely flat land like the other core materials. It is covered by 0.1 m thick Top soil. It is composed of Silty clay which is light brown, firm, moderately plastic, wet. Black cotton clay soil which is dark in colour, firm, highly plastic and wet is found at 1.5m depth. This material can be used as core material with treatment.
Area – 3	436925E 1011795N	2519325.54	1.3	This area is the other source of core material. It is covered by 0.1m thick Top soil. It is composed of silty clay, black colour with grass roots and Clay which is dark brown, slightly firm, moderate to highly plastic, wet. This material can be used as core material with treatment.
Area – 4	428837E 1000762N	1231398.87	33	This material is found a little far from the dam site in flat area. It is covered by 0.2 m thick Top soil. It is composed of dark brown, dry, firm, moderately plastic clay. This material can be used as source of core material.
Area – 5	442209E 1001460N	40731.40	14	This is a flat land which found near holeta city. It is composed of, light brown, dry, firm, slightly plastic Silty clay. The moisture content increases at depth. This material can be used as source of core material.
Area – 6	442098E 1004697N	5250489.43	9	This material covers a large area in a flat land. It is covered by 0.1 m thick Top soil. It is composed of brown, slightly wet, firm, moderately plastic clay. This material can be used as source of core material.
Area – 7	440724E 1012404N	100422.75	8	This material is found in a flat land and is covered by 0.1m thick Top soil.it is composed of dark brown, slightly wet, firm, moderately plastic clay. The moisture increases at depth. This material can be used as source of core material.

The catchment area of the Berga dam is a flat land surrounded by number of hills. This area is very suitable because it is flat and the siltation problem at the base of the dam will be reduced greatly. The area is composed of brownish clay with silt and minor sand. Further, the area is underlain by fractured basalt. This area can also be used as one source for core material (Fig. 5.2).

The command area (Plate 5.2) for the proposed dam site is located on the downstream side of the dam. It covers an area of about 3000 hectare. It is completely a flat area which makes it suitable for cultivation.

Plate 5.1 Proposed Catchment Area

Plate 5.2 Proposed Command Area

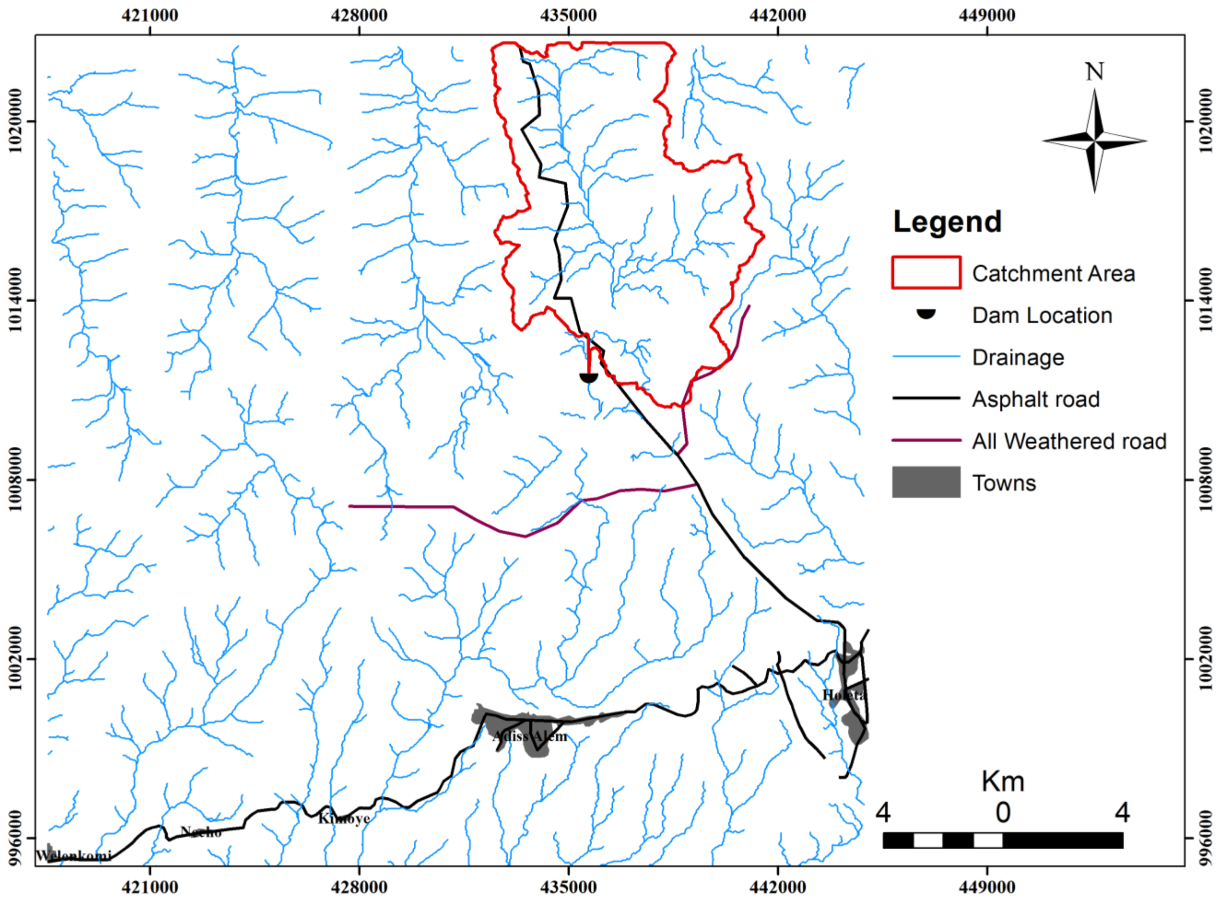


Fig. 5.2 Catchment Area

Other than the availability of construction material, the general setup of the command area was also the reason for the selection of the dam site. It is composed of thick brownish clay. There is hardly any vegetation cover other than some small grass coverage.

5.2 Characterization of Embankment Material

5.2.1 The Core Material

The core is the main structure that helps to prevent the flow of water so that it can be accumulated in the reservoir area. The main criterion for the construction of the core is the availability of the suitable material near the dam site. Available quantities with relevant soil properties are the other suitability factors and this needs proper investigation. Permeability, compacted density, shear strength, compressibility, flexibility and erosion resistance are important soil properties to be considered during the study of core materials. These properties are mainly governed by the grain size and aggregate properties (Murthy, 1989).

Engineering properties of the Core Material

During the previous (GSE) and present works the engineering properties of the soils from borrow areas for the core material was studied (Plate 5.3). The index and engineering properties for soils from 6 borrow areas were determined and is presented in Table 5.4.

Soils from Borrow Area-2, Area-4 and Area-7 are classified as MH (Inorganic silts of High compressibility) according to the Unified Soil Classification system (USCS). Further, soils from Borrow Area-3 and Area-5 are classified as CH (Inorganic clays of High compressibility) and Area-6 as ML (Inorganic silts of High compressibility) as per USCS.

Other than the soil classification using the grain size, different tests were also conducted on the samples. The free swell of the samples of the borrow areas ranges from 45 to 70 %. The liquid limits falls in between 43 and 75 and the plasticity index falls in between 10.7 and 41. The UCS of the soils ranges from 3.5 to 13 kg/cm^2. The dry density and moisture content of soils from borrow Area-2 and Area-6 on an average is 1.7 gm/cc and 14.3 %, respectively.

The soils on an average have Cohesion (C) equal to11Kpa and angle of shearing resistance (ϕ) equal to 20°. As the soils are classified as CH, MH and ML, according to IS: 1498 – 1970 they may have permeability in the range of 10^{-6} to 10^{-8} cm/sec, 10^{-4} to 10^{-6} cm/sec and 10^{-3} to 10^{-6} cm/sec, respectively (Garg, 2005).

Grain-size Analysis

Disturbed samples were collected from the borrow areas to determine the grain size distribution using sieve and pipette analysis. The results are presented in Fig. 5.3. According to the grain size test, except one sample (ATP - 3), in general the samples contain more than 90% fines (silt and clay size).

Table 5.4 Index and Engineering Properties of the Clay Core Material from Borrow Areas

Properties		Areas												
		Area 2					Area 3	Area 4	Area 5	Area 6				Area 7
		BD - 1	ATP – 2B	BD – 3A	BD – 3B	BD - 8	ATP – 3	ATP – 4	ATP - 5	BD - 4	ATP - 6	BD - 7	BD - 9	ATP – 7
Grain size	GR	-	-	-			-	-	-		-			-
	SA	-	1.3	-			20.1	1.2	1.5		1.4			0.9
	SI	-	49.1	-			36.2	35.4	34.5		43.2			74
	CL	-	49.6	-			43.7	63.4	64		55.4			25.1
Atterberg Limits	LL	-	57	-			75	55	67	43	49			59
	PL	-	33	-			34	35	33	32.25	32			37
	PI	-	24	-			41	20	37	10.7	17			22
Classification (USCS)		-	MH	-	-	-	CH	MH	CH	-	ML	-	-	MH
Free Swell (%)		-	60	-	-	-	65	55	70	-	45	-	-	55
UCS (kg/ cm^2)		-	4.5	-	-	-	3.5	7	13	-	10	-	-	10
Dry Density (gm/cc)		-	-	-	-	1.62	-	-	-	-	-	1.56	1.91	-
Moisture Content (%)		-	-	-	-	20.59	-	-	-	-	-	14.72	7.66	-
Permiability (cm/sec)*		-	10^{-4} to 10^{-6}	-	-	-	10^{-6} to 10^{-8}	10^{-4} to 10^{-6}	10^{-6} to 10^{-8}	-	10^{-3} to 10^{-6}	-	-	10^{-4} to 10^{-6}
Shear Strength	C	-	-	-	-	-	-	-	-	-	-	-	-	-
	ϕ	-	-	-	-	-	-	-	-	-	-	-	-	-
Remarks**														

* Using IS of soil characteristics pertinent to Embankments and Foundations

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

GR – Gravel, SA – Sand, SI – Silt, CL – Clay, LL – Liquid Limit, PL – Plastic Limit, PI – Plastic Index, C – cohesion and ϕ - angle of shearing resistance, USC – Unified Soil Classification, UCS – unconfined compressive strength

Atterberg's Limits

The Swedish soil scientist Albert Atterberg (1911) originally defined seven "limits of consistency" to classify fine-grained soils, but in current engineering practice only two of the limits, the liquid limit (LL) and plastic limits (PL), are commonly used. (A third limit, called the shrinkage limit (SL), is used occasionally.) The Atterberg limits are based on the moisture content of the soil. PL is the moisture content that defines where the soil changes from a semi-solid to a plastic (flexible) state. LL is the moisture content that defines where the soil changes from a plastic to a viscous fluid state. SL is the moisture content that defines where the soil volume will not reduce further if the moisture content is reduced. A wide variety of soil engineering properties have been correlated to the liquid and plastic limits, and these Atterberg limits are also used to classify a fine-grained soil according to the Unified Soil Classification System (USCS) (Garg, 2005).

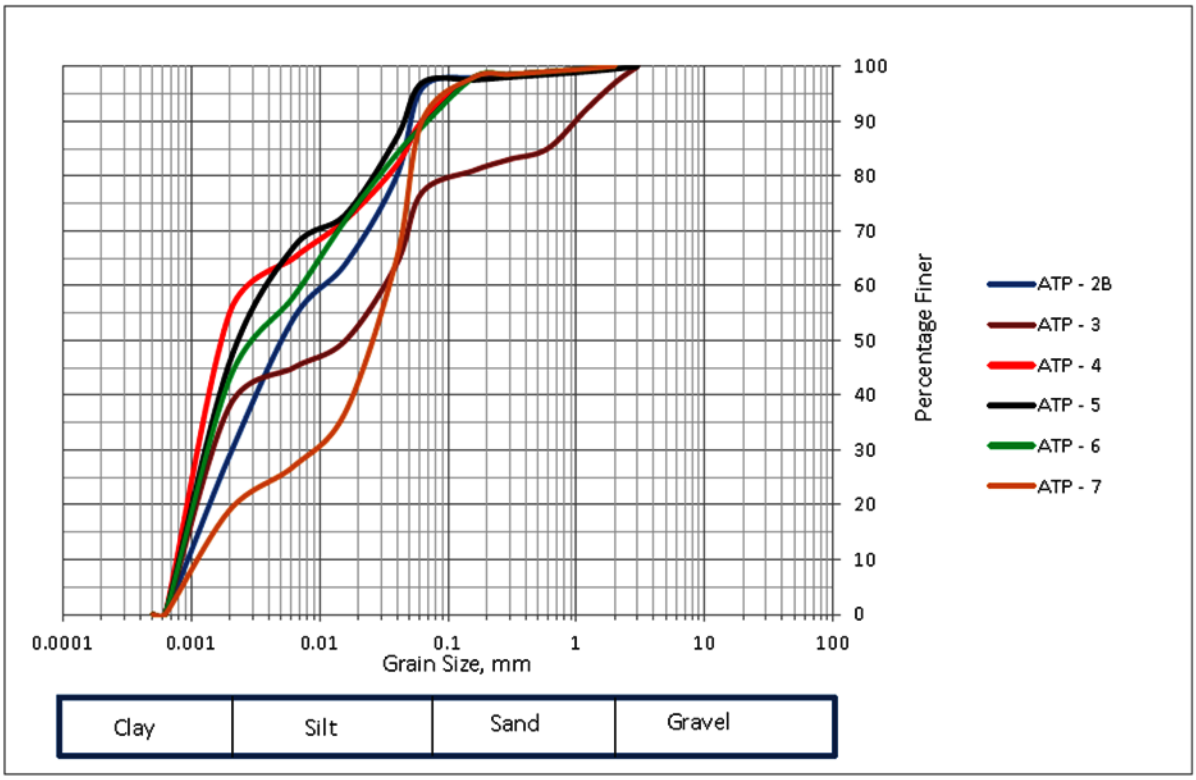


Fig. 5.3 Grain size analysis of Borrow Areas

Plate 5.3 Area 2 - Proposed Core material at the upstream of the Berga River

In the previous (GSE) and the present studies, samples were collected from borrow areas to determine the Atterberg limits of the soils. The results are presented in Fig. 5.4.

The plastic index (PI) is calculated by using eq.5.1.

PI = LL – PL ……………………….*eq.5.1*

Where; 'PI' is Plasticity Index, 'LL' is the Liquid Limit and 'PL' is the Plastic Limit.

The results thus obtained are plotted on plasticity chart (Fig.5.4). A perusal of Fig. 5.4 clearly shows that five samples fall below the 'A line' and two samples above the A line.

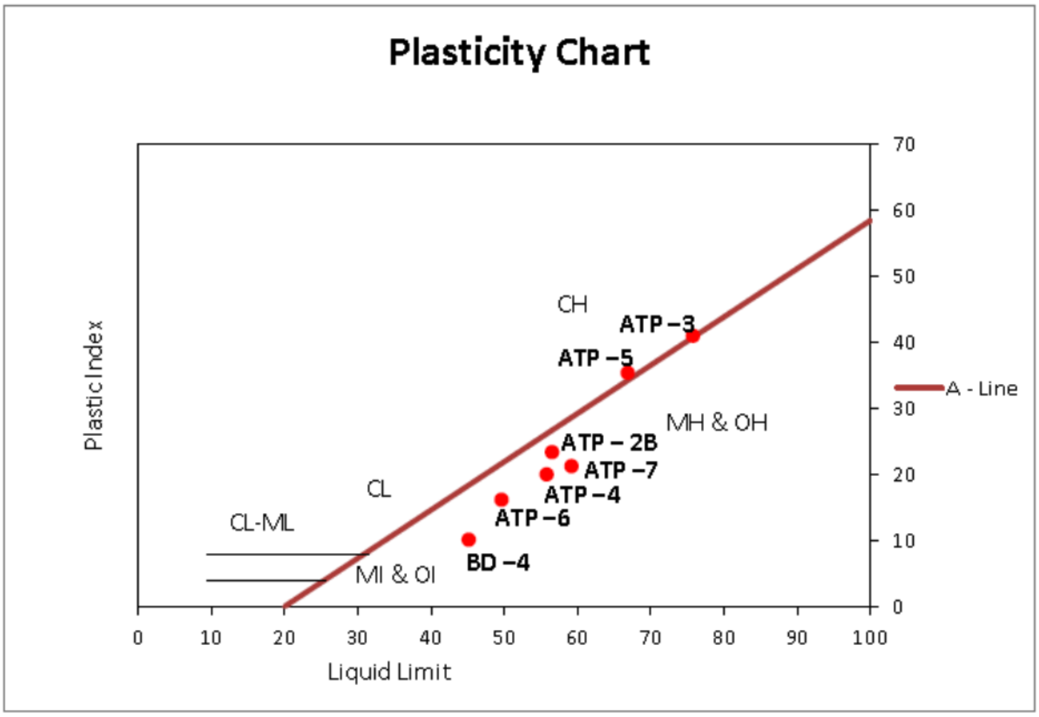


Fig. 5.4 Atterberg limits of soil samples

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 et al., (1962) has empirically related the swelling potential (Sp) with the plasticity index (PI) and can be expressed by eq. 5.2.

$Sp = 60 * K * PI^{2.44}$ ……………………….*eq.5.2*

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

Table 5.5 shows the swelling potential of clays of proposed borrow Areas.

By using chart for evaluation of potential expansiveness (Seed et al., 1960) in Fig. 5.5, two samples (CH) fall in the high expansiveness area and four samples (MH, ML) in medium expansive area.

Table 5.5 Swelling Potential of Clays of Proposed Borrow Areas

Sample No.	Plastic Index (PI)	Swelling Potential (Sp)*	Expansivity**
ATP - 2B	24	5.03	High
ATP - 3	41	18.6	High
ATP - 4	20	3.23	Medium
ATP - 5	34	11.78	High
ATP - 6	17	2.17	Medium
ATP - 7	22	4.073	Medium
BD - 4	10.75	0.7	Low

Degree of Expansivity based on SP	
Sp*	**Expansivity****
<1.5	Low
1.5 – 5	Medium
5-25	High
>25	Very High

(Source: Garg, 2005)

Free Swell Index

This indicates swelling potential of fine-grained soils when water is added to them. If the free swell index of a soil is more than 100, then such behavior may require special attention (Agarwal, 2000). As samples of the study area have the free swell value between 45 to 70%, one sample fall in the category of non-critical degree of severity and five samples are marginal based on I.S. 1498-1970 (Table 5.6).

Table 5.6 Free Swell Classification of Samples of the Study Area

Area	Sample No	Free Swell (%)	Degree of Expansion	Degree of Severity
Area 2	ATP – 2B	60	Medium	Marginal
Area 3	ATP – 3	65	Medium	Marginal
Area 4	ATP – 4	55	Medium	Marginal
Area 5	ATP – 5	70	Medium	Marginal
Area 6	ATP – 6	45	Low	Non Critical
Area 7	ATP – 7	55	Medium	Marginal

Activity of the soils

If a number of samples are taken from a particular clay stratum, and their plasticity indexes are plotted against the clay fraction (i.e. percentage by mass of particles finer than 2 micron), a straight line is generally obtained, which passes through the origin. The slope of this line is defined as the activity of the given clay (Garg, 2005).

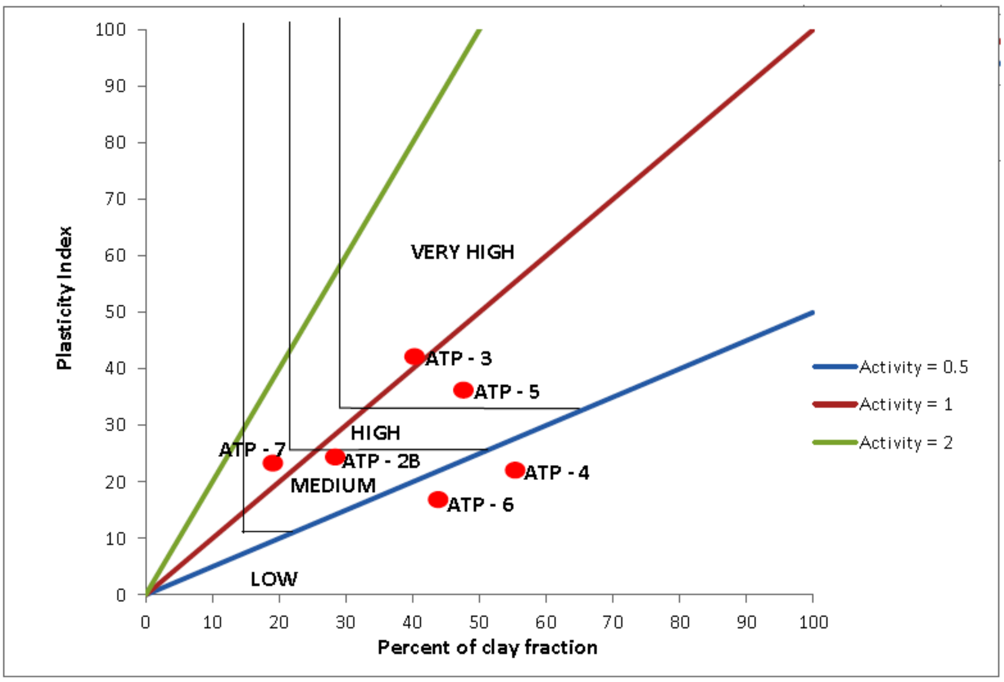


Fig. 5.5 Chart for evaluation of potential expansiveness (*Seed, et al., 1960*)

Hence, the activity number of activity ratio (Ac) is given by eq 5.3;

$$A_c = \frac{\text{Plasticity Index}}{\text{Percentage finer than 2 micron}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots. eq.5.3$$

The activity of clay, in fact, signifies the swelling characteristics of clay. The activity of the samples of the study area was analyzed and is presented in Table 5.7 and Fig. 5.6.

Permeability

The property of a soil, which permits the flow of water through it, is called the permeability. For the present study coefficient of permeability for proposed core samples has been determined by using the empirical technique proposed by Allen Hazene (1892).

According to Allen Hazane's (1892) formula;

$K = C * (D10)^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots. eq.5.4$

Where, 'K' is the coefficient of permeability (cm/sec), 'C' is constant usually taken as 100 and 'D10' is the effective grain size (Arora, 1997).

Table 5.7 Activity of clays

Sample No.	Plastic Index (PI)	Clay Percent	Activity Ratio (Ac)	Classification
ATP - 2B	24	29	0.82	Normal Clay
ATP - 3	41	38	1.08	Normal Clay
ATP - 4	20	55	0.36	Inactive Clay
ATP - 5	34	46	0.8	Normal Clay
ATP - 6	17	43	0.4	Inactive Clay
ATP - 7	22	19	1.16	Normal Clay

Note:

Ac	**Classification**
<0.75	Inactive Clay
0.75 – 1.25	Normal Clay
>1.25	Active Clay

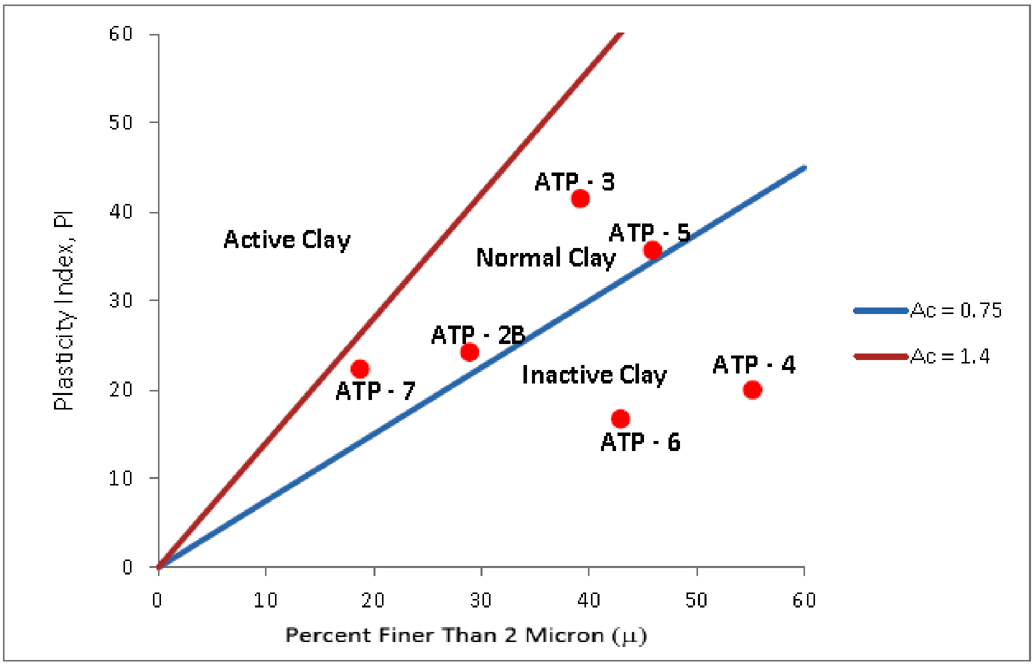


Fig. 5.6 Plot Showing Activity of samples of the study Area

The results thus obtained are presented in Table 5.8.

Table 5.8 Permeability of Proposed Core Materials

Sample No.	ATP – 2B	ATP – 3	ATP – 4	ATP – 5	ATP – 6	ATP – 7
D10 effective size	0.0009	0.0008	0.0007	0.0008	0.0008	0.0011
Permeability (cm/sec)	$8.1x10^{-5}$	$6.4x10^{-5}$	$4.9x10^{-5}$	$6.4x10^{-5}$	$6.4x10^{-5}$	$1.21x10^{-4}$
Classification (USCS)	MH	CH	MH	CH	ML	MH
Suitability as Core material*	Poor core material	Fairly suitable	Poor core material	Fairly suitable	Poor core material	Poor core material

* As per I.S. Soil Classification (IS: 1498 - 1970)

Shear strength Properties

Cohesive soils such as clays, offer shear resistance due to two factors; (i) friction between soil grains, which oppose the shear movement and (ii) the cohesion (Garg, 2005).

The shear strength parameter values for the soils to be used as core material on an average are; Cohesion 'C' equals to 11kpa and angle of shearing resistance 'ϕ' equals to 20°.

Compaction

Compaction may be defined as the process of packing the soil particles by reducing the air in the soil voids, by mechanical means. If the soil is not compacted well, and is piled up in a loose state, it is likely to settle down in future, or wash away easily. It is, therefore, important to compact the soils in the field to a desired extent. Water plays an important role in compaction. The limiting moisture, which is most useful for compaction, is called the optimum moisture content (OMC). At this moisture content, the soil would be compacted to a maximum possible degree, and would have the maximum density or unit weight, i.e. the weight of soil grains in a unit volume of compacted soil mass, would be maximum (Garg, 2005).

According to Middlebrooks (1942) the core material should satisfy the following criteria;

i) It must be placed at a density or moisture, which will not allow further consolidation on saturation.
ii) It must be sufficiently plastic so that differential settlement will not cause cracks to develop through it.

For the present study compaction test was performed in Geological Survey of Ethiopia laboratory on soil samples from borrow Areas. The compaction test results revealed that the average dry density of the samples comes out to be 1.7 gm/cc and the average moisture content of 13.11% (Table 5.9).

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 materials 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 (Singh and Varshney, 1995).

Table 5.9 Water Content and Dry Density of Samples of the Study Area

Sample Number	Water content (%)	Average Water Content (%)	Dry Density (gm/cc)	Average Dry Density (gm/cc)
BD - 7	14.76	14.72	1.56	1.56
	14.67		1.56	
BD - 8	20.68	20.59	1.61	1.62
	20.5		1.62	
BD - 9	7.65	7.66	1.92	1.91
	7.68		1.89	
BD - 10	9.48	9.47		
	9.45			
Average		**13.11**		**1.7**

Singh and Varshney (1995) have made the following recommendations for erosion resistance;

(i) The rate of erosion decreases with increasing plasticity index (PI) up to a value of 15, after which its influence is small. Higher compacted density reduces the rate of erosion.

(ii) The addition of bentonite significantly improves erosion resistance, particularly in well graded soils. The addition or inclusion of stone chips to the extent of 10-20% also improves erosion resistance.

(iii) An attempt should be made to select soil with a 'PI' between 15 and 20%. Stones up to 10 to 20% should be included if found in the deposit, the maximum size being limited by compaction thickness. The soil should be compacted to high density.

The suitability of soil for construction of dam is given in Table 5.10.

For the present study an attempt has been made to estimate the erosion resistance based on the plasticity index (PI) of the material. Among the samples tested two samples have a plasticity index (PI) less than 20 and the rest five have more than 20.

The samples of the study area are classified based on their PI as presented in Table 5.11.

Mineralogical Composition of Core Material

Clayey soils are composed of an aggregate of clay mineral and non-clay minerals (Atwell, 1976). The clayey soils are formed by the disintegration (chemical weathering) of the clay minerals like kaolinite, illite or montmorillonite. Such minerals themselves are formed by the breakdown of rocks; say for example, the kaolinite mineral is formed by the breakdown of feldspar by the action of water and carbon dioxide.

Table 5.10 General Suitability of Soil for Construction of Dams *(Source: Singh and Varshney, 1995)*

Relative Suitability	Homogeneous Dams	Impervious Core	Previous Shells	Impervious Blanket
Very Suitable	GC	GC	SW, GW	GC
Suitable	CL-CI	CL, CI	GM	CL, CI
Fairly Suitable	SP, SM	GM, GC	SP, GP	CH, SM
	CH	SM, SC, CH	-	SC, GC
Poor	-	ML, MI, MH	-	-
Not Suitable	-	OL, OI, OH, Pt	-	-

Table 5.11 Flexibility and erosion resistance of borrow areas soils based on *Singh and Varshney* **(1995) criteria**

Sample Number	PI	USCS	Singh Classification
ATP - 2B	24	MH	Poor Material
ATP - 3	41	CH	Poor Material
ATP - 4	20	MH	**Good Material**
ATP - 5	34	CH	Poor Material
ATP - 6	17	ML	**Good Material**
ATP - 7	22	MH	Poor Material
BD - 4	10.75	ML	**Good Material**

Most of the clay mineral particles of colloidal size (< 0.002 mm) are of plate like (flaky) form, having a high specific surface (i.e. a high surface area to mass ration) with the result that their properties are significantly influenced by the surface forces. Long needle like particles can also occur, but are comparatively rare (Garg, 2005). The behavior of clay mineral is governed by electrical and internal surface forces, which can be analyzed by the fundamental nature of the particles and clay water interaction (Atwell 1976 cited in Ali Aman, 2008). Identification of the type of clay minerals using X-ray diffraction is useful to understand the behavior of the soil.

X-Ray Diffraction (XRD)

XRD is the most widely used method for identification of clay minerals and to study their crystal structure. The complete X-ray diffraction patterns, either film or strip chart record, consist of a series of reflections of different intensities. The different minerals are characterized by 1st order basal reflections at 7, 10 or 14 Å (Mitchell, 1976).

Qualitative X-ray diffraction and few simple tests are sufficed to indicate the minerals present in thesoil. More data is required for precise quantitative estimates (Lambe and Whitman, 1976).

In order to check the mineralogical composition during the present study, the XRD analysis was conducted on one sample of the core material. The analysis indicates presence of quartz, kaolinite and feldspar minerals in the sample. The summery of XRD results is shown in Table 5.12.

Table 5.12 XRD Mineralogical Analysis of core sample

Sample Number	Minerals Identified
BD – 12	Quartz – 73.1 % Kaolinite – 9.8 % Feldspar Potassian – 17.1 %

The sample BD – 12 represents the soil around the dam axis. As it was mentioned in the previous chapter, the soil in the area is classified as ML according to the USCS. This soil possesses plasticity potential. However, the XRD result shows that this soil has about 73% of quartz and 9.8% kaolinite. The high proportion of quartz in the soil sample contradicts to the expansive potential of the soil. Here, it also doest seems reasonable to presume that kaolinite alone which is around 10% in the sample have enough proportion to make soil sample expansive in nature. Thus, it requires further study to understand the fact that why the soils are expansive with such a high proportion of quartz and reasonably low proportion of expansive mineral Kaolinite.

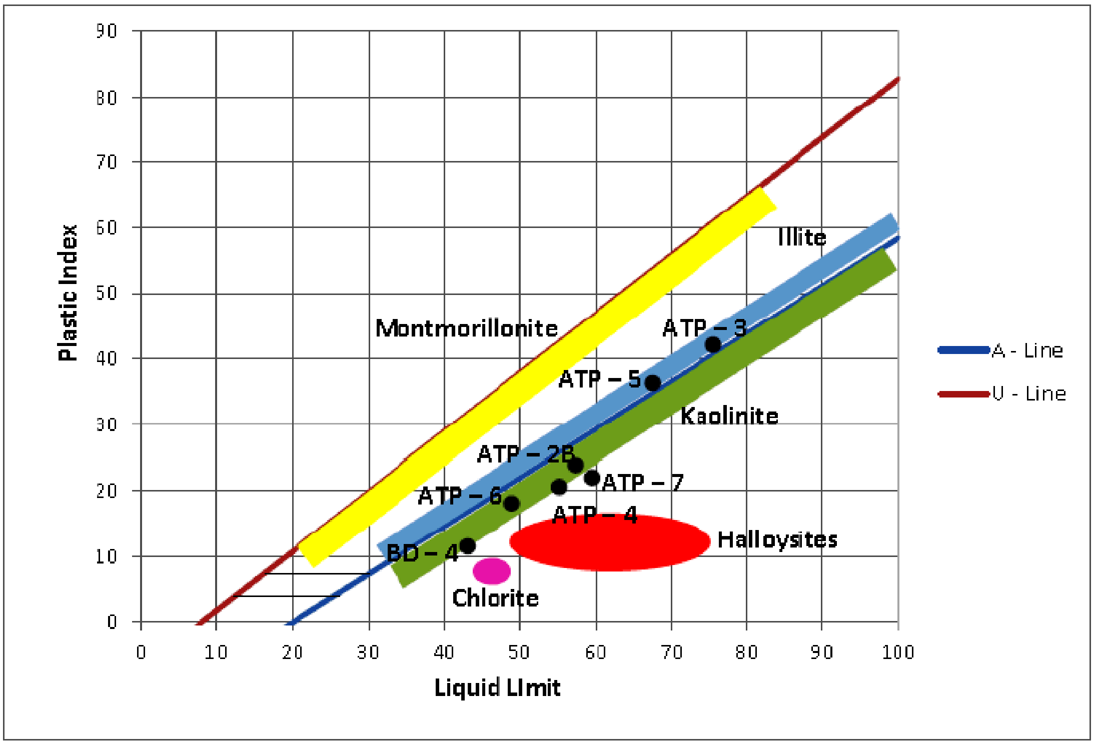
Plasticity chart to estimate clay minerals present in the soil

Another method to analyze clay minerals in a soil sample is to use plasticity chart to determine the clay minerals. It is analyzed by using the liquid limit and plastic index of soil samples as they are plotted on the plasticity chart to identify the type of clay mineral present in the soil. This approach is often inaccurate, because soils can contain more than one type of clay minerals (Fakhry, 2004).

Fig. 5.7 shows the plasticity chart for core material from the proposed borrow areas. The results clearly indicates that plot of5 samples from clay borrow area ATP - 2, 4, 6 falls within MH, ML and Kaolinite region whereas, 2 samples from area ATP-3 and 5 falls within CH region and Illite zone.

Liquefaction/ Dynamic behavior of the core material

The term liquefaction is used to describe large deformations occurring in a soil mass caused by monotonic, temporary disturbance of saturated cohesion less soil (Mogami, 1953).



(Source: Day, 2006)

Fig. 5.7 Plasticity Chart for core material from proposed borrow areas

Generation of excess pore pressure under un-drained loading is the main ingredient for liquefaction, since it is directly related with a sudden decrease of the effective shear strength of the soil. In the evaluation of the liquefaction hazard it is required to (a) determine the soil susceptibility to liquefaction, (b) determine the potential for liquefaction to be triggered, and (c) estimate the extent of damage. The previous concepts applies to any earth retaining structures that contact with water and is located in seismic regions, such as an earth fill dam. In such cases liquefaction potential may exist either in the embankment material itself or within the foundation alluvial (in case the dam is resting on soil), and/or the abutments (Fernandez, 2009).

The response of the embankment for the imposed dynamic load is highly influenced by the dynamic behavior of the embankment material. Important studies on the dynamic strength of the soils were carried out by Seed and Chan (1967). They indicated the superimposing pulsating load; the dynamic strength is lower than normal static strength for sensitive clays or for cohesive clay of low density, especially at higher water content (Singh and Varshney, 1995).

In order to liquefy, cohesive soils must meet the following criteria (Day, 2006):

(i) The soil must have less than 15 % of the particles, based on dry weight, that are finer than 0.005 mm (i.e., percent finer at 0.005 mm < 15 %).

(ii) The soil must have a liquid limit (LL) that is less than 35 (that is, LL < 35).

(iii) The water content '*w*' of the soil must be greater than 0.9 of the liquid limit (that is, *w* > 0.9 (LL)).

Using Day's (2006) criteria the proposed borrow areas soils were analyzed and are presented in Table 5.13.

Perusal of Table 5.13 clearly shows that the soils of the study area are not susceptible for liquefaction. However, as the average water content for the samples and the percent finer than 0.0063 is taken, the individual water content must be determined for better classification of the soils. As the dam site is located near to the active Ethiopian rift, care must be taken during construction especially considering the liquefaction property of the dam materials.

5.2.2 The Filter Material

The filter design for the drainage layers and internal zoning of a dam is a critical part of the embankment design. It is essential that the individual particles in the foundation and embankment are held in place and do not move as a result of seepage forces. This is accomplished by ensuring that the zones of material meet "filter criteria" with respect to adjacent materials.

In a zoned embankment the coarseness between the fine and coarse zones may be such that an intermediate or transition section is required. Drainage layers should also meet these criteria to ensure free passage of water. All drainage or pervious zones should be well compacted. Where a large carrying capacity is required, a multilayer drain should be provided (Gedeon, 2004).

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 (Singh and Varshney, 1995).

Table 5.13 Liquefaction property of soils of proposed borrow areas by using Day (2006) Criteria

Sample No.	LL (%)	Percent finer at 0.0063 mm*	Water Content** (Average)	0.9*LL	Liquefaction Potential
ATP – 2B	55	54	13.11	49.5	Not Susceptible
ATP – 3	75	45	13.11	67.5	Not Susceptible
ATP – 4	55	65	13.11	49.5	Not Susceptible
ATP – 5	67	67	13.11	60.3	Not Susceptible
ATP – 6	49	28	13.11	44.1	Not Susceptible
ATP – 7	59	27	13.11	53.1	Not Susceptible

* As there is no data for percent finer than 0.005 mm on the present analysis the values for percent finer than
0.0063 are used.
** Taken from Table 5.9 – this is because all the samples are taken from the same areas.

Proper testing has to be done to ensure durable particles that will not be unacceptably altered or changed during excavation, hauling, placement and compaction or by long term weathering and erosion. Testing should include grain size distribution, hardness, mechanical breakdown, design stress, density, abrasion, chemical resistance and freeze thaw (USBR, 1987).

Engineering Properties of Filters

For the proposed Berga dam, there is only one borrow site at the downstream of the river in the study are within the economic distance which may provide suitable filter material (Plate 5.4). In order to determine the engineering property of the filter material different tests were performed. According to the grain size analysis of the samples of the filter material about 50% of the sample is sand and the rest are fines. This composition doesn't entirely satisfy the filter criteria that are used for filter design. Table 5.14 and Fig. 5.8 shows test result for gradation and classification of the filter material proposed to be used for the dam.

Filter Criteria

The drainage filters must be designed in such a way that neither the embankment nor the foundation material can penetrate and clog the filters. The permeability or size of the filter material should also be sufficient to carry the anticipated flow with an ample margin of safety (Garg, 2005).

The particle size which are commonly used for filter selection criteria are D_{15}, D_{50} andD_{85} of the filters and the protected layers. These particle sizes of the base and filter material, which are deduced from the gradation curve, are presented in Table 5.15. As discussed in Chapter 2, different criteria give varied emphasis to a different particle sizes.

Plate 5.4 Area 1 -Proposed Filter material at the downstream of the Berga River

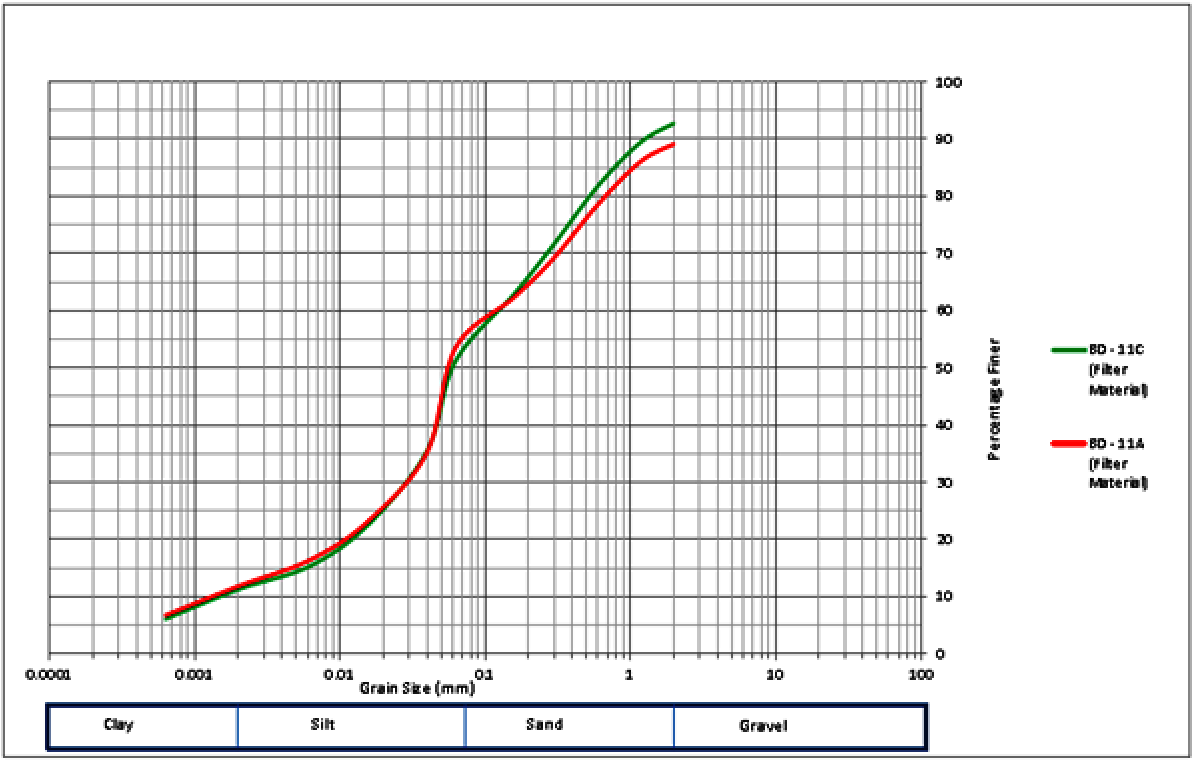


Fig. 5.8 Grain size distribution of filter material

To assess the general suitability of the filter material proposed to be used for Berga dam (borrow Area-1), filter criteria proposed by Terzaghi, India Standard, US Army Corps, USBR and Sherard has been applied. A comparative assessment has been made and the results are presented in Table 5.17.

Table 5.14 Test Results for Gradation and Classification of Filter Material

Sample No.	Grain Size Distribution			Cu	Cc	Classification (USCS)
	Gravel	Sand	Fines			
BD – 11A	-	46.46	53.54	107.9	4.95	SM
BD – 11C	-	48.81	51.19	87.5	4.018	SM

Table 5.15 Size limits for filter materials

Sample Number	D_{15} (mm)	D_{50} (mm)	D_{85} (mm)
BD – 11A	0.005	0.056	1.2
BD – 11C	0.006	0.06	0.8

As 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 are relaxed from this condition. A comparative gradation curve plot for filter and core material is presented as Fig.5.9.

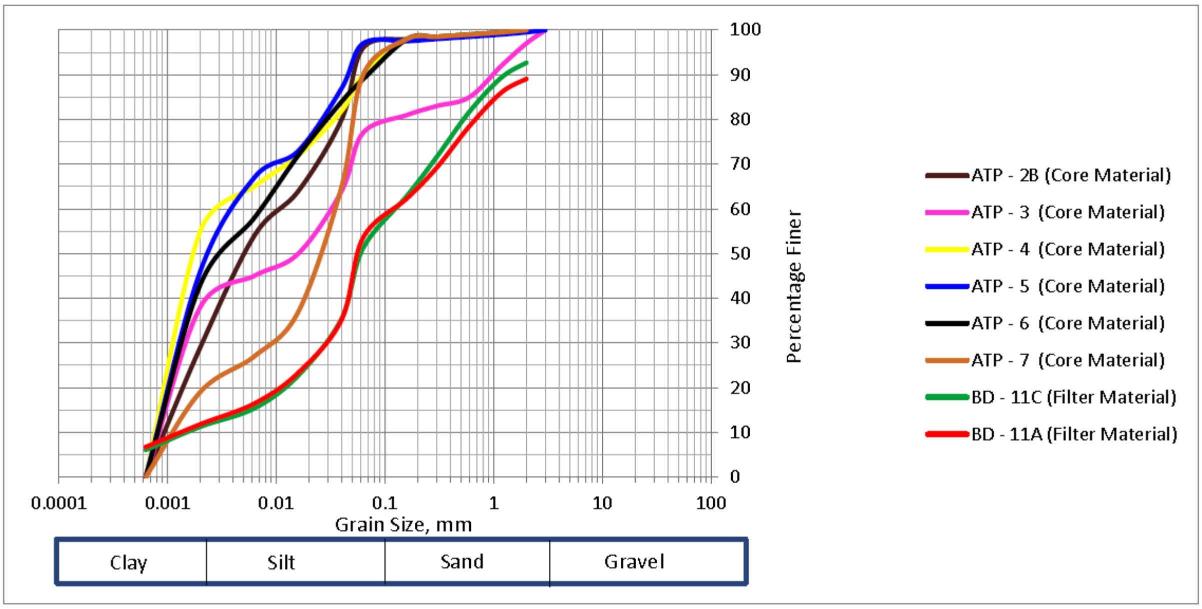


Fig. 5.9 Gradation Curve plot of Filter and Core material

Based on Table 5.16, Table 5.17, Table 5.18 and Fig. 5.9, the curve of the filter materials and the core materials are more or less parallel and most of the criteria are satisfied. However, the content of the fines of the filter materials are more than 5% which does not satisfy the filter criteria. In general about 55% of the criteria are satisfied by the samples.

Dynamic behavior/ property of the filters

According to Day (2006), the main factors that govern liquefaction of soils are; earth quake intensity, relative density, particle size gradation, particle shape, drainage conditions and confining pressure mostly affect the performance of filter zone of the embankment dams. The magnitude of the earthquake is responsible for the number of cyclic loading expected to be imposed in all engineering structures including the embankment dams.

As the magnitude increase the number of the cyclic loading increases resulting in decrease in the strength or resistance to liquefaction. Seed and Dealba (1986) proposed the representative number of cycles loading imposed verse earthquake magnitude as shown in Table 5.19.

Earthquakes of certain magnitude of destructiveness which had occurred in the near past, within a given area, will be expected to occur in the near future (Day, 2006). As the proposed Berga dam site is found near the rift it will have the probability of having higher magnitude of earthquakes in future (EBCS-8). Therefore, seismic investigation must be carried out for the possible earthquake magnitudes and based on the results the number of cycles can be adopted from Table 5.19so that the dam will be less susceptible to liquefaction and other damages related to earthquakes.

Other than the method proposed by Day (2006) to determine the liquefaction potential of the dam materials, Professor H. Bolton Seed and his colleagues at the University of California at Berkeley (1984) proposed the use of Factor of Safety against Liquefaction (FS_L). It is expressed by eq. 5.5;

$$\frac{CRR}{CSR} = FS_L \ldots\ldots\ldots\ldots\ldots\ldots\ldots.eq.5.5$$

The CSR is the ratio of the average cyclic shear stress (τave) to the vertical effective overburden stress (σ'v). The average cyclic shear stress (τave) is defined as 65% of the maximum computed shear stress in the soil (Seed et al., 1984). The CRR was determined by procedures established which focused on evaluating the liquefaction resistance of soils, and was intended to supplement the simplified procedure developed by Seed et al. (1984).

These procedures estimates the capacity or in-situ liquefaction resistance of the soil, as a direct function of the corrected Standard Penetration Test (SPT) blow counts (Fernandez, 2009).

Table 5.16 Result of Filter Criteria

Criteria	Requirement		Filter Material Site	Core Material						Result (Average of BD – 11A and BD – 11C)
				ATP – 2B (1)	**ATP – 3 (2)**	**ATP – 4 (3)**	**ATP – 5 (4)**	**ATP – 6 (5)**	**ATP – 7 (6)**	
Indian Standard	D15 of Filter< 5 D85 of base		BD – 11A	0.1<5	0.008<5	0.09<5	0.09<5	0.1<5	0.08<5	1 – Yes 2 – Yes 3 – Yes 4 – Yes 5 – Yes 6 – Yes
			BD – 11C	0.12<5	0.009<5	0.1<5	0.11<5	0.13<5	0.1<5	
	D15 of Filter>4 &<20 D15 of base		BD – 11A	4.54>4	5.55>4	6.25>4	5.55>4	5.55>4	3.12>4	1 – Yes 2 – Yes 3 – Yes 4 – Yes 5 – Yes 6 – No
			BD – 11C	5.45>4	6.66>4	7.5>4	6.66>4	6.66>4	3.75>4	
	D50 of Filter<25 D50 of base		BD – 11A	1.17<25	0.41<25	3.53<25	2.6<25	2<25	0.17<25	1 – Yes 2 – Yes 3 – No 4 – No 5 – Yes 6 – Yes
			BD – 11C	11.76<25	4.1<25	35.3>25	26>25	20<25	1.7<25	
US Army corp of Eng. (1995)	D15 of Filter≤ 5 D85 of base		BD – 11A	0.1<5	0.008<5	0.09<5	0.09<5	0.1<5	0.08<5	1 – Yes 2 – Yes 3 – Yes 4 – Yes 5 – Yes 6 – Yes
			BD – 11C	0.12<5	0.009<5	0.1<5	0.11<5	0.13<5	0.1<5	
	D50 of Filter<25 D50 of base		BD – 11A	1.17<25	0.41<25	3.53<25	2.6<25	2<25	0.17<25	1 – Yes 2 – Yes 3 – No 4 – No 5 – Yes 6 – Yes
			BD – 11C	11.76<25	4.1<25	35.3>25	26>25	20<25	1.7<25	
Terzaghi	D15 of Filter< 4 D85 of base		BD – 11A	0.1<4	0.008<4	0.09<4	0.09<4	0.1<4	0.08<4	1 – Yes 2 – Yes 3 – Yes 4 – Yes 5 – Yes 6 – Yes
			BD – 11C	0.12<4	0.009<4	0.1<4	0.11<4	0.13<4	0.1<4	
	D15 of Filter>4 D15 of base		BD – 11A	4.54>4	5.55>4	6.25>4	5.55>4	5.55>4	3.12>4	1 – Yes 2 – Yes 3 – Yes 4 – Yes 5 – Yes 6 – No
			BD – 11C	5.45>4	6.66>4	7.5>4	6.66>4	6.66>4	3.75>4	
Sherad's	D15 of Filter< 9 D85 of base		BD – 11A	0.1<9	0.008<9	0.09<9	0.09<9	0.1<9	0.08<9	1 – Yes 2 – Yes 3 – Yes 4 – Yes 5 – Yes 6 – Yes
			BD – 11C	0.12<9	0.009<9	0.1<9	0.11<9	0.13<9	0.1<9	
USBR	Cu = 3 to 4	D50 of Filter= 5to10 D50 of base	BD – 11A	1.17=5to10	0.41=5to10	3.53=5to10	2.6=5to10	2=5to10	0.17=5to10	1 – No 2 –No 3 – No 4 – No 5 – No 6 – No
			BD – 11C	11.76=5to10	4.1=5to10	35.3=5to10	26=5to10	20=5to10	1.7=5to10	
	WG – PG (R)	D50 of Filter=12to58 D50 of base	BD – 11A	1.17=12to58	0.41=12to58	3.53=12to58	2.6=12to58	2=12to58	0.17=12to58	1 – No 2 – No 3 – No 4 – No 5 – No 6 – No
			BD – 11C	11.76=12to58	4.1=12to58	35.3=12to58	26=12to58	20=12to58	1.7=12to58	
		D15 of Filter=12to40 D15 of base	BD – 11A	4.54=12to40	5.55=12to40	6.25=12to40	5.55=12to40	5.55=12to40	3.12=12to40	1 – No 2 – No 3 – No 4 – No 5 – No 6 – No
			BD – 11C	5.45=12to40	6.66=12to40	7.5=12to40	6.66=12to40	6.66=12to40	3.75=12to40	
	WG – PG (A)	D50 of Filter=4to30 D50 of base	BD – 11A	1.17=4to30	0.41=4to30	3.53=4to30	2.6=4to30	2=4to30	0.17=4to30	1 –No 2 –No 3 – No 4 – No 5 – No 6 – No
			BD – 11C	11.76=4to30	4.1=4to30	35.3=4to30	26=4to30	20=4to30	1.7=4to30	

		D15 of Filter=6to18 D15 of base	BD – 11A	4.54=6to18	5.55=6to18	6.25=6to18	5.55=6to18	5.55=6to18	3.12=6to18	1 – No 2 –No 3 – Yes 4 –No 5 –No 6 – No
			BD – 11C	5.45=6to18	6.66=6to18	7.5=6to18	6.66=6to18	6.66=6to18	3.75=6to18	
Note; WG – Well Graded ; PG – Poorly Graded; R – Rounded; A - Angular										

Table 5.17 Result of filter materials other than the criteria for migration and permeability

Filter Criteria		Filter material parallel plot to the Core material plot	Percentage of materials less than 0.075 mm	Percentage of materials greater than 75 mm
Terzaghi/USBR	BD – 11A	Yes	51%	0%
	BD – 11B	Yes	51%	0%

Table 5.18 Summary of Filter Criteria

Filter Criteria		Remark
Indian	BD – 11A	The sample satisfies most of the criteria
	BD – 11B	The sample satisfies most of the criteria
US Army Corp of Eng. (1995)	BD – 11A	The sample satisfies most of the criteria
	BD – 11B	The sample satisfies most of the criteria
Terzaghi	BD – 11A	The sample satisfies most of the criteria
	BD – 11B	The sample satisfies most of the criteria
Sherad's	BD – 11A	The sample satisfies most of the criteria
	BD – 11B	The sample satisfies most of the criteria
USBR	BD – 11A	The sample does not satisfy most of the criteria. That is it does not fall on the specified range of values for different ratios of both the filter and the core material
	BD – 11B	The sample does not satisfy most of the criteria. That is it does not fall on the specified range of values for different ratios of both the filter and the core material

Table 5.19 Earthquake magnitude vs number of representative cyclic loading

Earthquake Magnitude (mb)	Number of Representative cycles	Volumetric strain ratio
8.50	26	1.25
7.50	15	1.00
6.75	10	1.85
6.00	5-6	0.60
5.25	2-3	0.40

(Source: Ali Aman, 2008)

5.2.3 Shell material

In a common type of earth fill embankment, a central impervious core is flanked by much more pervious shells that support the core. The upstream shell affords stability against end of construction, rapid drawdown, earthquake, and other loading conditions. The downstream shell acts as a drain that controls the line of seepage and provides stability under high reservoir levels and during earthquakes. For the most effective control of through seepage and seepage during reservoir drawdown, the permeability should increase progressively from the core out toward each slope. Frequently suitable materials are not available for pervious downstream shells. In this event, control of seepage through the embankment is provided by internal drains (Gedeon, 2004). The borrow areas for the shell 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 erosion resistant rock fragments, gravels and cobbles available around the dam site.

Engineering Properties of the Shell Material

Most of the present study area and its surrounding are covered by residual and alluvial soils which are classified as; MH, CH, and ML. However, there are few locations from where soils can be used as filter material, as mentioned in previous chapters. Considering the shell material, the locations that were selected for filter materials, close to Kimoye, soils contains particles with sizes of gravel and cobble (Plate 5.5). The gravels are without impurities such as clays and iron. The gravel and cobble size grains are generally basaltic in composition. The undesired quality in this zone, which results from the mineralogy of the material, is the solubility and disintegration of the grains which may lead to settlement of the zone in which they will be used. However, the identified material is free from this effect. Therefore, in terms of the mineralogy, the shell material does not have unwanted quality which can affect the performance of the zone (Yohannes Belete et al., 2009). The quarry sites can also be used for the shell material as a support for the core.

Based on the field and laboratory tests, rocks in Addis-Alem sub sheet are classified into medium and high strength units. Positive relationship between bulk density and point load index strength as well as bulk density and laboratory result of UCS, and an inverse relationship between porosity with point load strength and UCS was noticed (Fig.5.10).

Material Gradation

As Area-1 is the only source for both Shell and filter material by combining the existing materials the property of the shell material can be improved. The gravel size deposit that exist in the area have very little amount of fines. Therefore, by combining the material with BD – 11A and BD – 11B with the gravel deposit, the shell material can have more permeability and it is expected that such blended material will prevent the development of pore water pressure in the shell material. In general the material contains about 10% Cobble, 40% Gravel, 45% Sand and 5% Fines.

5.2.4 Riprap Material

Adequate slope protection must be provided for all earth and rock-fill dams to protect against wind and wave erosion, weathering, ice damage, and potential damage from floating debris.

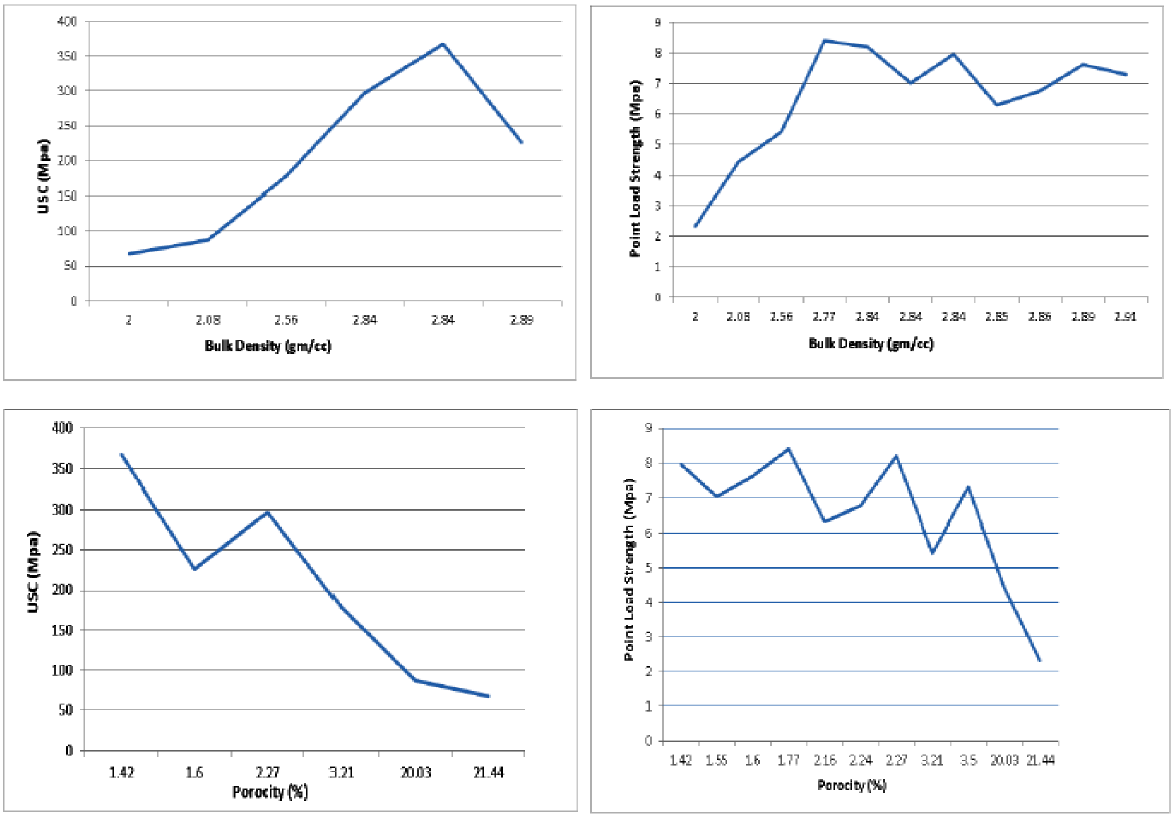


Fig. 5.10 Graphic relationships between engineering parameters of rock

Plate 5.5 Area 1 - Proposed Shell and Filter material at the downstream of the Berga River

The required protection depends on the expected wind velocities and duration, the size and configuration of the reservoir, the permanent water surface elevation, and the frequency of the pool elevation. Methods of protecting slopes include dumped riprap, precast and cast-in-place concrete pavements, soil cement, bituminous soil stabilization, sodding, and planting. The type of protection provided is governed by available materials and economics. Due to the high cost, the initial slope protection design should be accomplished during the survey studies to establish a reliable cost estimate. The final design should be presented in the appropriate feature design document (Gedeon, 2004).

The main purpose of riprap in embankment dams is to prevent erosion and damage from wave action. Rock fragment dumped riprap is the most common type of slope protection used for embankment dams. Quarried rock is the most common source of rock fragments. 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 (USSD, 2011).

Engineering properties of riprap

The rocks that are used for rip rap should be composed of dense, sound durable rock with acceptable shape factor and sufficiently large to withstand the wave action from the reservoir. 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 (USSD, 2011).

A good quality and an adequate volume of basaltic rock quarry sites were selected and inventoried in the study area to be used as crushed stone aggregate for crushed aggregate base, asphalt and concrete (Yohannes Belete et al., 2009) (Plate 5.6).

The basalts found are well jointed with vertical and horizontal joints with joint spacing of 0.5 to 1m. The presence of joints makes the basalt easily workable and economical with regards to the excavation. The basalt is relatively fresh and strong and do not have other soft rocks or soils. The crushed aggregates will not be flaky since the joints are not very closely spaced and have no impurities such as; organic matter and clay.

Further, results of laboratory tests made on basalt samples indicated a density variation between 2.56 and 2.91 gm/cc, while their water absorption is less than 2% and UCS values range from 177 to 307 Mpa. These parameters show the suitability of basalts to be used as riprap material.

Mineralogical properties of riprap

Quality evaluation of riprap relies on petrographic examinations, in addition to examination by experienced and qualified personnel supplemented with data from laboratory durability testing. Petrographic examinations may reveal defects in rock which may seem satisfactory from laboratory testing. Because of the large size of the rock fragments required, the testing needs to be performed on small representative samples.

Plate 5.6 Quarry site for Riprap

Ideally rock for riprap should meet the quality specifications of concrete aggregate. No minimum quantitative specifications can be given for the rock quality; the best available material should be used. Generally, natural boulders, because of their more rounded shape, will have poorer interlocking than quarried rock pieces and slightly lower resistance to wave action for the same piece size. Elongated or flat pieces will have less stability for the same mass than equi-dimensional pieces when dumped randomly (USSD, 2011).

The basalts of the study area generally classified as upper and lower basalts which are black in color. Both types of basalts are either aphanitic or porphyritic. Based on the previous petrographic studies the lower basalt consists on an average90% ground mass , 4% plagioclase, 3% olivine , 2% opaque, and 1%sericite whereas the upper basalt contains 37-65% ground mass, 25-35% plagioclase, 19% olivine, 10%opaque, 5% sericite, 4% iddingsite (Yohannes Belete et.al., 2009).

5.2.5 General Characterization of the Embankment Material

In the present study all the proposed borrow areas and the quarry sites were characterized based on their engineering properties, mineralogical properties and dynamic behaviors. The core material was classified as; MH, CH or ML based on the USCS. Based on the plasticity charts, most of the core materials fall in the kaolin region and few on the Illite region. Filter and shell material is only available at one Borrow site. Based on the grain size analysis the material contains about 50% of sand and rest is fines. Gravel and cobble size materials are also available at this site which can be used for the shell material. The available quarry site are very suitable to be used for the riprap material as they are very strong and have high density based on the laboratory test results. In the following paragraphs, a detailed description on suitability of embankment construction material is presented.

5.3 Suitability Analysis of Embankment Material

During the present study, the construction material from proposed borrow areas and quarry sites was evaluated for its suitability to be used in the construction of proposed embankment dam at Berga River.

Since, most of the construction materials are found within the economic distance from the proposed dam site, therefore their suitability may help to construct an economical dam.

According to Manual on Small earth dams by USBR (1987) materials in general which are not suitable for the embankment dam construction is listed below;

- Organic material (except when used to top dress the embankment and other parts of the dam site at the end of the construction period).
- Decomposing material.
- Material with a high proportion of mica, which forms slip surfaces in soils of low clay percentages.
- Calcitic soils such as clays derived from limestone which, although generally stable, is usually very permeable.
- Fine silts, which are unsuitable for any zone of the dam.
- Schists and shales which, although often gravelly in texture, tend to disintegrate when wet. Schists may also contain a high proportion of mica.

- Cracking clays that fracture when dry and may not seal up when wetted in time to prevent piping through them.
- Sodic soils (fine clays with a high proportion of sodium). They are difficult to identify in the field, so any fine clay should be analyzed.

5.3.1 Suitability of Core material

Historically it was believed that core should be constructed of clay rather than silt. This belief was popular because clay is less permeable than silt, and clays were considered to be less vulnerable to internal erosion under concentrated leaks. Current practice tends to place more emphasis on utilizing economically available resources; including silty materials properly moisture conditioned and compacted. It is preferable to avoid brittle cores and use more plastic materials, if available. Placing silty material at higher moisture content mitigates the brittleness at the expense of added construction pore pressure, and settlement. Dam design requires making the best use of engineering characteristics of the various materials available for construction of the embankment. The size and shape of the impervious core in a zoned dam will depend on the availability of materials and their properties, especially hydraulic conductivity. Site conditions requiring specific construction sequencing may also be a factor in the zoning design (USSD, 2011).

Engineering Property of the Core material

The previous (GSE) and the present studies conducted on the proposed core materials shows that they are; MH, ML or CH as per USCS classification. According to the IS: 1498 - 1970 for soil characteristics pertinent to embankments and foundation; the MH is poorly stable and can only be used for cores of hydraulic fill dam and not in rolled fill construction. The ML is also poorly stable and may be used for embankments with proper control. The CH on the other hand is fairly or moderately stable with flat slopes, thin cores, blanket and dike sections. The test conducted on soil samples during the present study shows plasticity index (PI) values greater than 20 for ATP – 2B, ATP – 3, ATP – 5 and ATP – 7. Thus, indicating that such soils are not erosion resistant. However, samples ATP – 4, ATP – 6 and BD – 4 have PI less than 20 which satisfies the Singh (1995) criteria for erosion resistant.

As per the USBR permissible limits the permeability for CH class of soils must lie within the range of 0.05 + 0.05 x 10^{-6} cm/sec, and the permeability values for the samples are between 10^{-6} and 10^{-8} cm/sec according to IS:1489 – 1970. However, these values must be verified with proper permeability test. 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. Conversely, USBR practice is to place the fill at 1 to 3% below optimum. It was found that this limited the construction pore pressure to not more than 30% of the weight of the overlying fill, which resulted in appreciable economy. The USBR dams have been built with compaction moisture content for impervious zone ranging from 0.7 to 2.5% dry of optimum (Singh and Varshney, 1995).

Moreover, as per the engineering use chart for compacted soil after USBR (1974) indicates that the workability of MI and CH group of soils is fair to poor and such soils are placed at $6^{th}$and 7^{th} place for their suitability to be used as core material for rolled compacted earth dam.

Referring to the mineralogical analysis; kaolinite, quartz and feldspar are the minerals identified in the sample tested. Quarts mineral dominates (73%) in the tested soil sample.

The available core materials do 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 may exist.

5.3.2 Suitability of Filters Material

According to USSD (2011), two fundamental functions are required from filters and drains in earth, earth-rock, and rock fill dams: (i) **Retention function:** The filter must prevent migration of soil particles from adjacent foundation or fill materials. Thus, a fine filter must prevent migration of finer grained impervious fill or foundation material; a coarse filter or drain must prevent any tendency for movement of the fine filter. This first requirement is often referred to as the piping or stability criterion. More recently, the term retention criterion has been used. (ii) **Permeability function:** The filter must accept seepage flows from adjacent foundation or fill materials without the buildup of excess hydrostatic pressure. Thus, a fine filter must readily accept seepage flows from a finer-grained impervious fill or foundation material; a coarse filter or drain must readily accept flow from an adjacent fine filter. Permeability ratios between adjacent materials of at least 25 are often quoted.

According to Jansen (1988), rounded, non-interlocking particles are essential for filters placed upstream of the impervious core, whose purpose is to readily flow into cracks in the core that might develop from normal consolidation of the embankment or due to earthquake loading. The quality of the material should also be considered during the selection of the filter material.

Engineering Property

The filter material selected for the proposed dam has about 50% of sand particles. In order to analyze this material, different filter criteria were applied. The analysis showed that material satisfies some of the criteria. The material in general has the permeability of about 10^{-3} to 10^{-6} cm/sec according to IS: 1498 – 1970. However, it contains large amount of fines than the desired specifications. Therefore, blending of this material must be performed so that it will have better and proper properties as per the standard requirements. As this material is the only possible source for filter, further detailed permeability studies needs to be carried out to establish its general suitability to be used as desired filter material.

Mineralogical Property

The mineral composition of the filter material is very important for the stability of the dam structure. If there are any soluble minerals found in the filter, in course of its performance it will not sufficiently prevent the migration of core material which will result in to piping. To identify the suitability of the filter material in terms of mineralogy, it requires a detailed analysis on sufficient number of samples. However, due to limitation on time and resources the required mineralogical analysis on filter material was not conducted during the present study. The mineralogical analysis is very important to known the performance of filter layer in placement position. Therefore, it is strongly recommended that mineralogical analysis should be conducted for potential sand source for filters.

Dynamic property of filters

According toSeed et al. (1985), the standard penetration test (SPT) values can indicate the approximate potential damage on a particular material (Table 5.20).

Table 5.20 Converted SPT value verse approximated potential damage

SPT Value	Potential Damage
0-20	High
20-30	Intermediate
>30	No Significant damage

(Source: Seed et.al., 1985)

The stability of the filter material under dynamic condition can be determined by performing the SPT so that the potential damage can be determined. The grain size, relative density, permeability and particle shape determines the stability of the filter material under dynamic condition (Seed et.al., 1985).

5.3.3 Suitability of shell material

Primary purpose of shell 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**.** The important engineering properties, which influences the performance of the shell material are permeability, compacted density and shear strength (EM - 1110-2-1901, 1986).

In the present study area the selected soils for shell material consists of gravel and cobble size particles. This material is basaltic and has higher density and lower porosity (Yohannes Belete et al., 2009).

The quarry sites that are found in the study area have high strength and durability to be used as a supporting shell material.

General Suitability in Terms of Permeability

The selected material for shell is classified as SM with large deposit of cobble and gravel at the downstream. Its permeability can be determined by using Allen Hazane's (1892) Formula. According to which the permeability of the soil is a product of its D10 square and a constant C. Based on this formula the proposed soils for shell material has a permeability of 2.56×10^{-4} cm/sec. This value can be improved by mixing it with an appropriate proportion of gravel. For this testing can be performed with different blending proportions, so that the material can have a better permeability.

5.3.4 Suitability of the riprap material

Of the various embankment protection materials, riprap has been the most used and the most economical and successful material. Riprap is a layer, facing or protective mound of stones randomly place to prevent erosion, scour, or sloughing of an embankment slope (USSD, 2011).

According to Fell et.al. (2005) the riprap comprises quarried blocks of rock which have to be;

- Large enough to dissipate the energy of the waves without being displaced;
- Strong enough to do this without abrading or without breaking down to the smaller size;
- Durable enough to withstand the effect of long term exposure to weather and varying period of wave without becoming weaker and breaking down to smaller size.

As per USACE (1994), for riprap up to 24 inch thick, the rock should be well graded from spalls to the maximum size required. Riprap sizes and thicknesses are determined based on the significant wave height (design wave). Riprap in the upstream slope should have a minimum thickness of 12 in. If the material is produced from quarry, identification of geological formations that can produce acceptable material is necessary. The blasting techniques and other excavation methods should be controlled to obtain the required size.

For the present embankment dam the ignimbrite rock has been identified for the riprap. This rock on an average has a bulk density of 2.04 t/m^3, uni-axial compressive strength (UCS) of 77.7 Mpa and water absorption ratio of 10.4 whereas, the basalt rock on an average has bulk density of 2.79 t/m^3, uniaxial compressive strength (UCS) of 267 Mpa and water absorption ratio of 0.72 (Yohannes Belete et al., 2009).

As per the durability value for rock, used for riprap layers, given by Singh (1995) (Table 5.21), the ignimbrite is classified as ‘marginal’ and the basalt as ‘excellent’ in terms of point load strength. In terms of water absorption ratio the ignimbrite is classified as ‘poor’ and the basalt as a ‘good’ rock. Also, in terms of bulk density the ignimbrite is classified as ‘poor’ and the basalt as ‘good’ rock. Therefore, based on these analyses for the proposed embankment dam the basalt is more suitable to be used as a riprap than the ignimbrite rock.

During the present study different charts and criteria were used to analyse the construction materials for the proposed Berga dam. Based on the laboratory tests, most of the soil samples were classified as highly plastic. However, on the activity chart these samples are located in the normal and inactive zone. For example ATP – 4 and ATP – 6 were classified as MH and MI by using USCS; respectively however, in the activity chart they are located in the inactive region. Similarly, the samples ATP – 3 and ATP – 5 were classified as highly expansive soils on the expansiveness chart however; they were classified as normal in the activity chart. Therefore, these charts do not seems to provide results appropriate for these soils and these empirical charts must be reviewed to account for such discrepancies. Never the less before making any such inferences on the appropriateness of these standard charts it may be required to check for number of samples rather than making any decision based on the present study in which only few samples were used.

5.3.5 Contradiction between Swelling Potential and Activity ratio of proposed core materials

Swelling potential of an expansive soil is the percentage swell of a soil after being compacted to maximum dry density at optimum moisture content. When the swelling potential is less than 1.5 %, the soil is low expansive type; when between 1.5% and 5%, it is of medium expansive type; when between 5% and 25%, it is of high expansive type; and when it is > 25%, the soil is of very high expansivity (Reference).

The activity of clay, in fact, signifies the swelling characteristics of clay. It qualitatively signifies the behavior of the soil as active, normal or inactive (Reference). In the present study, the proposed core material for the embankment dam was analyzed using different charts and properties. In these analysis most of the result are in good agreement with each other. However, a contradiction was observed between the swelling potential and the activity ratio.

In the swelling potential analysis (Table 5.5), the samples are classified as medium to high expansive soils except one sample which is classified as low expansive soil. In the activity ratio analysis (Table 5.7), on the other hand, the soil samples were classified as normal and in active soils.

These results do not agree with each other. Therefore, in the present study an attempt was made to understand such discrepancy in results.

As it was described in the swelling potential analysis, the main property that was considered was the plasticity index. Plasticity index is the range of water content in which the soil behaves like a plastic material. It is the difference between the liquid limit and the plastic limit of a soil. It indicates the degree of plasticity of a soil. As the plasticity index increases, the swelling potential of the soil also increase. The activity of the soil is ratio between its plastic and the percentage of clay sized particle. Like the swelling potential, as the plastic index increase, the activity ratio increases (Garg, 2005).

During the Atterberg limit analysis no material was lost as a result of sample preparation. This may help to increase the plasticity index as it was determined by Dumbleton and West, (1966). In this analysis it was found out that the liquid limit increase for natural kaolinitic soil than pure kaolinite soil. This was especially the case for tropical soil. The reason for this fluctuation was primary because of the presence of iron oxide. The iron oxide serves as a coating material in the soils. This effect was more effective in kaolinite than the montmorillonite. The presence of iron oxide, especially in the form of hematite, makes the measurement of the plastic and liquid limits more difficult. Another factor which has been shown to affect the plasticity of soils is the size distribution of the clay particles less than 2 μ range. In addition, fine-grained particles which are not clays in the mineralogical sense can also produce plasticity.

By considering the above results, the soils of the study area have different source as it can be seen from the regional geological set up of the area. The kaolinite is produced from the weathering of the feldspar from the granite and other rock types that are found in the area.

The presence of iron oxide can be indicated as a result that the area, especially the dam site, is composed of basalt as the underlying bed rock. Therefore, as the River brings the kaolin from upstream area and deposit, the soil in the area is dominantly basalt; the weathering results into iron oxide which may have the chance to interact with the kaolin from the upstream site. This may probably result in to high liquid limit, plastic limit and plastic index. However, if the samples are washed before preparation, the effect of iron oxide will decrease and at the same time the value for Atterberg limits will decrease.

In conclusion, the samples of the study area show these contradicting results as a result of sample preparation. More samples must be taken and both Atterberg and chemical analysis must be performed in order to understand the relation between the content of iron oxide, clay particles content and the Atterberg limits. If the result of such analysis confirms the relation, the charts for plasticity and the activity must be considered for improvement especially in the case of tropical soils like that are found in the study area.

5.4 Seepage Control

All earth and rock-fill dams are subject to seepage through the embankment, foundation, and abutments. Seepage control is necessary to prevent excessive uplift pressures, instability of the downstream slope, piping through the embankment and/or foundation, and erosion of material by migration into open joints in the foundation and abutments. The purpose of the project, i.e., long-term storage, flood control, etc., may impose limitations on the allowable quantity of seepage. The three methods for seepage control in embankments are flat slopes without drains, embankment zonation, and vertical (or inclined) and horizontal drains (EM - 1110-2-1901, 1986).

Table 5.21 Guide to Rock Durability for Riprap layer

Test	Excellent	Good	Marginal	Poor	Comments
Rock Density (t/m^3)	> 2.9	2.6-2.9	2.3-2.6	< 2.3	Physical property affecting hydraulic stability. Good indicator of durability except dense but weathered basic rocks
Water absorption (%)	< 0.5	0.5-2.0	2.0-6.0	> 6	Single most important indicator of resistance to degradation. Good indicator of weathering resistance. Often misleading for porous limestone with large free draining pores.
Magnesium sulphate soundness	< 2	2-12	12-30	> 30	Indicate resistance to weathering important test for porous sedimentary rock for use in hot dry climate. Good correlation with water absorption
Mill abrasion resistance index	< 0.002	0.002-0.004	0.004-0.15	> 0.015	Indicate resistance to abrasion by mutual grinding of saturated rock surfaces.
Point load index (ISMR) Mpa	> 8	4.0-8.0	1.5-4	<1.5	Quick test with high test variability. Can be misleading for impact resistance of large block.

(Source: Singh , 1995)

For some dams constructed with impervious soils having flat embankment slopes and infrequent, short duration, high reservoir levels, the phreatic surface may be contained well within the downstream slope and escape gradients may be sufficiently low to prevent piping failures. For these dams, when it can be assured that variability in the characteristics of borrow materials will not result in adverse stratification in the embankment; no vertical or horizontal drains are required to control seepage through the embankment (EM - 1110-2-1901, 1986).

Embankments are zoned to use as much material as possible from required excavation and from borrow areas with the shortest distances and the least wastage and at the same time maintain stability and control seepage. For most effective control of through seepage and seepage during reservoir drawdown, the permeability should progressively increase from the core out toward each slope (EM - 1110-2-1901, 1986).

Vertical (or inclined) and horizontal drains may be required to control seepage through the embankment by preventing material eroded through a crack in the core from washing into the downstream shell by seepage water under reservoir head. Also, because of the often variable characteristics of borrow materials, it is frequently advisable to provide vertical (or inclined) and horizontal drains within the downstream section of the embankment to ensure satisfactory seepage control. For a stratified soil, the vertical permeability is controlled by the least permeable layer. Therefore, the horizontal permeability is always greater than the vertical permeability. Compacted soils in earth dams are stratified due to variability in the characteristics of borrow materials and the tendency for soil particles to align horizontally during compaction (EM - 1110-2-1901, 1986).

For dams located where earthquake effects are likely, there are several considerations which can lead to increased seepage control and safety. The core material should have a high resistance to erosion (Arulanandan and Perry, 1983). Relatively wide transition and filter zones adjacent to the core and extending for the full height of the dam can be used. Additional screening and compaction of outer zones or shells will increase permeability and shear strength, respectively. Geometric considerations include using a vertical instead of inclined core, wider dam crest, increased freeboard, and flatter embankment slopes, and flaring the embankment at the abutments (Sherard 1966, 1967).

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 particles 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 (Singh and Varshney, 1995). As a result, it is important to control the migration of soil particles resulting in to 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 (Singh and Varshney, 1995).

The saturation of seepage forces is mainly due to the excessive pore pressure causing slope failure, liquefaction failures due to earthquake shocks, foundation blowout 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 (Singh and Varshney, 1995).

The soils of the proposed borrow areas are classified as CH, MH and ML. According to IS 1498 – 1970, these soils do not need any seepage control. However, the ML soils may need toe trench at the downstream side. These soils are also found in the dam foundation area. However, detailed water pressure test must be conducted in the foundation area to determine the seepage potential particularly in the foundation area. If the area is susceptible to seepage any of the following seepage control methods, individually or in combination, can be used as needed; Curtain Grouting, Diaphragm wall, Upstream Impervious Blanketing, Cut off Trench, Downstream Free-Draining Zone or shell, Rock toe and Drains, Horizontal Drainage and Chimney Drains. However, any of such drainage control methods needs proper assessment of degree of seepage potential and other ground conditions for anticipated conditions.

Seepage Potential at dam area

In the present study one permeability test was conducted. However, on the basis of the guide for "resistance to leak" through soils (Table 5.22) as proposed by Sherald (1967) the soils were further assessed. Sherard (1967) has graded different material according to their potential for concentrated leaks (Table 5.22).

Table 5.22 Approximate classification of core materials on the basis of resistance to concentrated leak

1. Very Good Materials
(a) Very well-graded coarse mixtures of sand, gravel and fines, D_{85} coarser than 50 mm, D_{50} coarser than 6 mm, it fines are cohesionless, not more than 20% finer than the 75-μ sieve.
2. Good Materials
(a) Well-graded mixture of sand, gravel and clayey fines. D_{85} coarser than 25 mm. Fines consisting of inorganic clay (CL) with plasticity index greater than 12.
(b) Highly plastic tough clay (CH) with plasticity index greater than 20.
3. Fair Materials
(a) Fairly well-graded, gravelly, medium to coarse sand with cohesionless fines. D_{85} coarser than 19 mm. D_{50} between 0.5 mm and 3.0 mm. Not more than 25% finer than the 75-μ sieve.
(b) Clay of medium plasticity (CL) with plasticity index greater than 12.
4. Poor Materials
(a) Clay of low plasticity (CL and CL-ML) with little coarse fraction. Plasticity index between 5 and 8. Liquid Limit greater than 25.
(b) Silts of medium to high plasticity (ML or MH) with little coarse fraction. Plasticity index greater than

10.

(c) Medium sand with cohesionless fines

5. Very Poor Materials

(a) Fine, uniform, cohesionless silty sand. D_{85} Finer than 0.3 mm.
(b) Silt from medium plasticity to cohesionless (ML). Plasticity index less than 10.

(After Sherarld, 1967 and IS 8826-1978, Appendix B)

The study area, in general, is covered by soils which has a thickness up to 3 m. The core material, ATP – 3 and ATP - 5, are 'good materials' as resistant to leak or seepage whereas ATP 2B, ATP -4, ATP – 6, ATP – 7 and BD - 4 are 'poor materials' (Table 5.22).

The soil in the foundation area is classified as MH and it is 'poor materials' as resistant to leakage. The abutments are formed from very strong basalt. However, they are covered by a soil and are highly fractured. Therefore, in order to improve the permeability stripping of undesired material is required. Further, it is strongly recommended that in situ water pressure tests must be performed and based on the results appropriate remedial measure/s must be evolved.

In general the study area, especially the dam area, is covered by slightly plastic soil which may possess permeability. All necessary preventive measures must be used to improve the foundation conditions, so that no excessive seepage can take place.

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Chapter 6 Stability Analysis of Dam

6.1 Introduction

An embankment of any size must be designed to be safe and stable during its entire life, including construction. To assess the safety of a dam and the possibility of failure, the different potential failure mechanisms must be recognized. All embankment dams are subject to some seepage passing through, under, and around them. If uncontrolled, seepage may be damaging to the stability of the structure as a result of excessive internal pore water pressure or by piping. Seepage should be effectively controlled to preclude structural damage or interference with normal operations (Ghassan, 2002).

According to IIT (2006), the various modes of failures of earth dams may be grouped under three categories: Hydraulic failures, Seepage failures, and Structural failures.

6.1.1 Hydraulic Failures

This type of failure occurs by the surface erosion of the dam by water (Fig. 6.1).

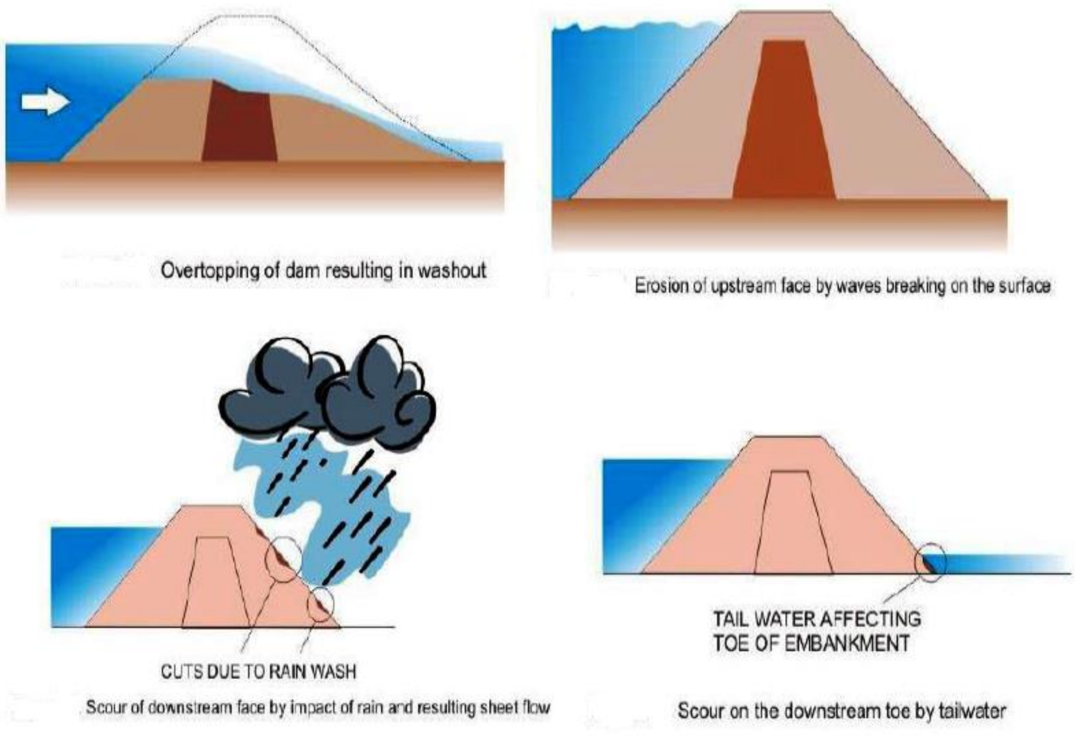


Fig. 6.1 Hydraulic failures

This may happen due to the following reasons:

(i) Overtopping of the dam which might be caused by a flood that exceeds the design flood for the spillway.

(ii) Erosion of upstream face and shoulder by the action of continuous wave action which may cause erosion of the surface, unless it is adequately protected by stone riprap and filter beneath it.

(iii) Erosion of downstream slope by rain wash.

(iv) Erosion of downstream toe of dam by tail water.

6.1.2 Seepage failures

Seepage failures may be caused in the following ways (Fig. 6.2):

(i) Piping through dam and its foundation

(ii) Conduit leakage

(iii) Sloughing of downstream face

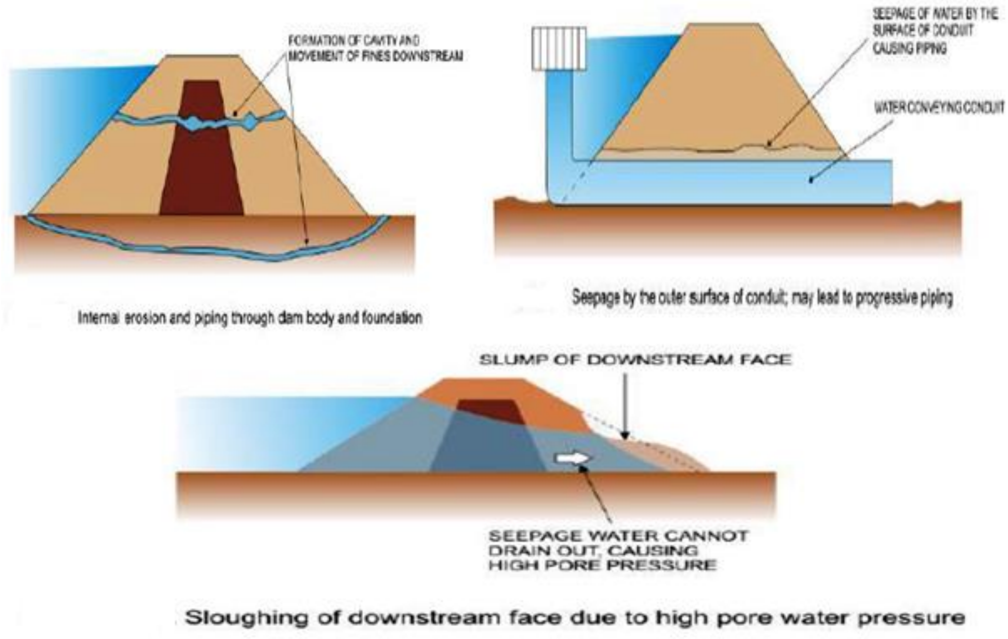


Fig. 6.2 Seepage Failures

6.1.3 Structural failures

These failures are related to the instability of the dam and its foundation, caused by reasons other than surface flow (hydraulic failures) or sub-surface flow (seepage-failures). These failures can be grouped into the following categories (Fig. 6.3);

(i) Sliding due to weak foundation

(ii) Sliding of upstream face due to sudden drawdown

(iii) Sliding of the downstream face due to slopes being too steep

(iv) Flow slides due to liquefaction

(v) Damage caused by burrowing animals or water soluble materials

(vi) Embankment and foundation settlement

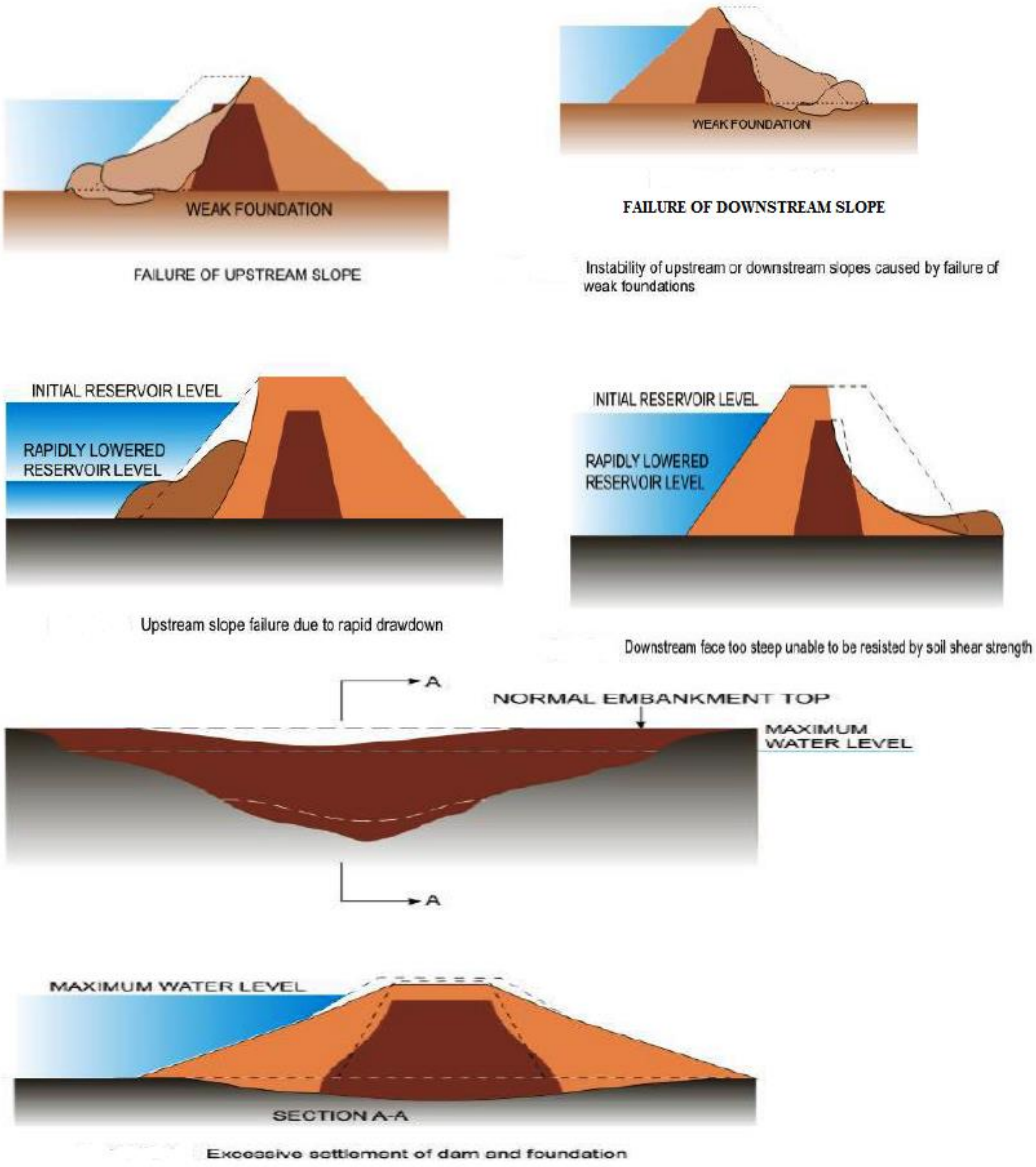


Fig. 6.3 Structural Failures

In general failure and damage of embankment dams can be grouped as during and after construction (Narita, 2000);

Failures or damage during construction

(i) Pore water pressure built-up during construction
(ii) Reduction of shear strength

Failures or damage after construction

(i) Hydraulic fracturing/ internal erosion/ Piping
(ii) Excess hydrostatic pressure due to rapid draw down
(iii) Reduction in shear strength/ Weathering, swelling of compacted soil
(iv) Settlement and cracking
(v) Earthquake forces

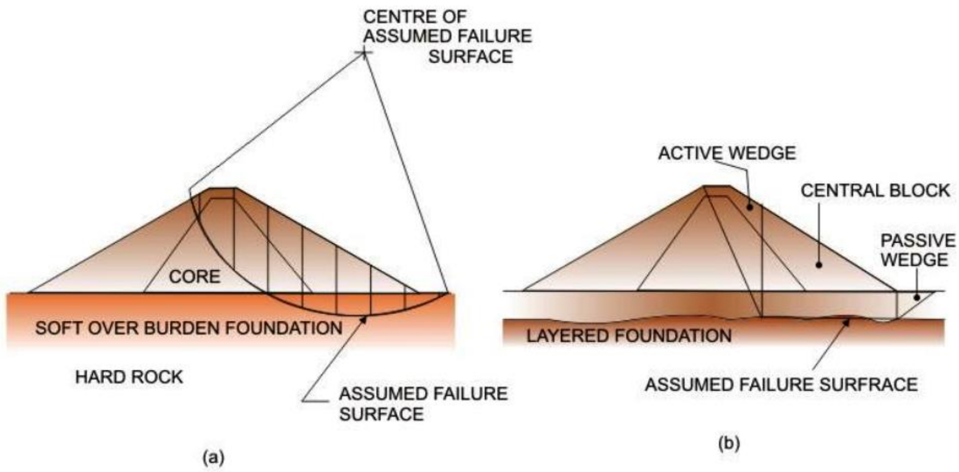
While planning and designing dam structures to be rested on natural grounds, soil testing is done to find out the bearing capacity and settlement characteristics of the ground. Quite often the selected site is not suitable to take the load of the proposed dam structure. It means that either the bearing capacity at shallow depth for the locating shallow footings may be too low or the likely settlements may exceed the tolerable limits. Usually, both of these factors are affected by void ratio and ground water. The ground condition to bear foundation load can, thus, be improved by these two methods; i.e. by reducing the void ratio and by lowering the water table. When using these two methods becomes un-economical, other methods like mechanical stabilization (vibro flotation and heavy weight compaction) and stabilization of compressible surface soil layers by pre-loading method can be used (Garg, 2005).

6.2 Stability Analyses

According to IS: 7894-1975, the slope stability methods generally employed to analyse the failure modes are two (Fig. 6.4), depending upon the profile of the assumed failure surface. These methods are; (a) Circular Arc method, and (b) Sliding Wedge method.

6.2.1 Circular Arc method

In this method (Fig 6.4(a)), the stability analysis consist of analyzing the forces acting on an assumed circular failure surface, and thus determining the total disturbing as well as the resisting moments about the center of that circle. The disturbing moments divided by the resisting moments will represent the factor of safety (F_s) that will be available for that particular assumed failure surface. This factor of safety would be different for different assumed failure surfaces, and its minimum value would represent the critical case. In other words, failure surface for which the minimum factor of safety is calculated would represent the critical failure surface. This method can be used even both for homogenous and zoned dams because the stability of the entire failure mass or the wedge is not considered as a whole but this mass is divided into a number of parts (slices) (Garg, 2005).



(a) Swedish slip circle method; (b) Sliding wedge method

Fig. 6.4 Stability analysis techniques for earth dams

The general principle of circular wedge is described below (Garg, 2005);

Let 'O' be the center and 'r' be the radius of the possible slip surface as shown in Fig. 6.5. Let the total arc 'AB' be divided into slices of equal width say 'b' meters each. The width of the last slice will be something different; say let it be 'm.b' meters. Let these slices be numbered as 1,2,3,…….and let the weight of these slices be W_1, W_2,W_3,……

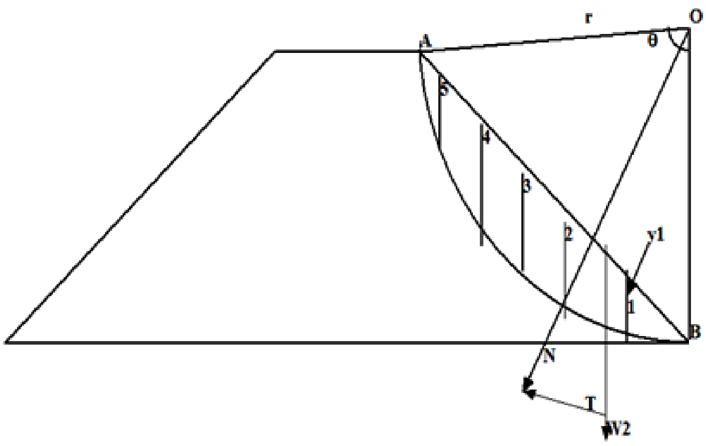


Fig. 6.5 Slip Surface

The forces between these slices are neglected and each slice is assumed to act independently as a vertical column of soil of unit thickness and width 'b'.

The sum weight of the slices is calculated by eq.6.1

$$\Sigma W = W_1 + W_2 + W_3 + \cdots + W_n$$

$$= \left[y_1 + y_2 + y_3 + \cdots + y_{n-1}\left(\frac{1+m}{2}\right)\right]\gamma.b\text{.........}eq.\ 6.1$$

The sum normal component (N) of the weight is calculated by eq. 6.2;

$$\Sigma N = cos\alpha(\Sigma W)\text{...............}eq.\ 6.2$$

The sum tangential component (T) of the weight is calculated by eq. 6.3;

$$\Sigma T = sin\alpha(\Sigma W)\text{.................}eq.\ 6.3$$

Therefore the factor of safety (F.S) against sliding is calculated by eq. 6.4;

$$F.S. = \frac{[c.AB + (tan\,\phi)\Sigma N]}{\Sigma T}\text{...........}eq.\ 6.4$$

Where;

y_1, y_2, y_3... = the vertical extreme ordinates (boundary ordinates) of slices 1, 2, 3…., respectively.

n = the total number of slices

γ = unit weight of soil

α = is the angle between the weight line passing through the centriod of slice and the normal passing through the point where weight line intersect the arc of failure surface within the slice base and joining the centre of the failure surface.

c = the unit cohesion

ϕ = the angle of internal friction of the soil

AB = length of the slip circle = $\left[\frac{2\pi.r}{360°}\right]\theta$

Where; 'θ' is the angle in degrees, formed by the arc 'AB' at centre 'O'.

6.2.2 Sliding Wedge method

Sliding wedge method of analysis is generally applicable in the circumstances where it appears that the failure surface may be best approximated by a series of planes rather than a smooth continuous curve.

This method is generally applicable under the following two circumstances:

(i) Where one or more horizontal layers of weak soil exists in the upper part of the foundation, and

(ii) Where the foundation consists of hard stratum through which failure is not anticipated and the dam resting on it has a core of fine grained soil with relatively large shells of dense granular material.

In sliding wedge method of analysis, the trial sliding mass is divided into two or three segments (Fig. 6.4(b)). The top segment is called the active wedge and the bottom segment is called the passive wedge. The middle wedge in case of a three wedge system is called the central block. The resultant of the forces acting on the active wedge and the passive wedge are first determined. These resultants acting on the central block along with other forces on the block shall give a closed polygon of forces for stability (IS: 7894-1975).

6.2.3 Conditions for Analysis

Among the above mentioned methods, critical surface analysis method is used for the present study. Other than the critical surface analysis, few site conditions were selected for further stability analysis.

An earth dam has to be safe and stable during all phases of its construction and operation of the reservoir. Hence, the analyses have to be carried out for the most critical combination of external forces which are likely to occur in practice (IS: 7894-1975).

The following conditions are usually considered critical for the stability of an earthen embankment dam (IIT, 2006);

Case I Construction condition with or without partial pool: Check stability of upstream and downstream slopes.

Case II Reservoir partial pool: Check stability of upstream slope.

Case III Sudden drawdown: Check stability of upstream slope.

Case IV Steady seepage: Check stability of downstream slope.

Case V Steady seepage with sustained rainfall: Check stability of upstream and downstream slopes.

Earthquake Condition

In the regions of seismic activity, stability calculations of the slope of an embankment dam has to include earthquake forces also because seismic activities reduces the margin of safety or may even bring about the collapse of the structure (IS: 1893-1975/2002). Where the analysis is carried out by the circular arc or sliding wedge method, the total weight of the sliding mass considered for working out horizontal seismic forces has to be based on saturated unit weights of the zones below the phreatic line and moist weights above it. If the zone above the phreatic line is freely draining, drained weights shall have to be considered for that zone.

6.3 Initial Design of Dam

The proposed Berga dam is situated in more or less very suitable area for the construction of embankment dam (by being flat area covered by impervious soil). The Berga Dam is in investigation stage and the dam has not been designed so far. In the present study attempt is being made to prepare the initial design of the dam. For this design the construction material available at various proposed borrow sites was considered. Further, the stability of embankment dam with initial design prepared during the present study was analyzed. The initial design of embankment was prepared by following "Tailor's stability number" approach. Later, the stability analysis was carried out by utilizing SLOPE/W software. The stability analysis was made for different anticipated conditions to which the dam may likely be subjected during its performance stage.

6.3.1 Taylor's Stability Analysis

It is well understood that gravity forces (due to unit weight 'γ') are a cause of instability while the cohesive forces (C_u) contributes to stability in a soil mass. These two (γandC_u) are body forces distributed throughout the soil mass and the above statement is valid for every point within the sliding mass. The maximum height 'H_c' of a slope that can be built without failure is thus directly proportional to the unit cohesion 'C_u' and inversely proportional to 'γ'. 'H_c' is also related to the values of '$\emptyset_u$' and 'β' (Arora, 1997; Ranjan, and Rao, 2002).

Thus, $H_c = \frac{C_u}{\gamma} f(\emptyset_u, \beta)$……………..eq.6.5

Where; $f(\emptyset_u, \beta)$ means a function of both '$\emptyset_u$' and 'β'.

Eq. 6.5 will be dimensionally correct if $f(\emptyset_u, \beta)$ is a dimensionless function.

Taylor (1937) expressed this as a reciprocal of a dimensionless number which he called the 'Stability number' (S_n). According to Taylor,

$H_c = \frac{C_u}{\gamma . S_n}$ eq.6.6

$S_n = \frac{C_u}{\gamma . H_c}$ eq.6.7

If a factor of safety 'F_c' is introduced with respect to cohesion, then;

$S_n = \frac{C_m}{\gamma . H} = \frac{C_u}{F_c} . \frac{}{\gamma H}$ eq.6.8

Where; 'H' is the height of a slope ($<H_c$) which has a Factor of Safety 'F_c' and 'C_m' is mobilized unit cohesion for equilibrium of a slope of height 'H'.

Taylor utilized the friction circle analysis along with an analytical procedure to determine the values of 'S_n' as a function of '$\emptyset_u$' and 'β' and presented them in the form of tables and graphs. These solutions are strictly valid only for the simple, homogeneous, finite slopes and for cases involving no seepage, but they can also be used for approximate and preliminary solutions of more complex cases. The graphs by Taylor are prepared indicating the stability number 'S_n' and slope angle 'β' for various values of '$\emptyset_m$' (Fig. 6.6). There are five important parameters which are $\gamma, H, C_m, \beta and \varphi_m$. If $\varphi_m = 0$, a sixth parameter is also used, it is 'D_f'. The parameter 'D_f' depends upon the depth of the hard strata below the top of the soil and is defined as;

$D_f = \frac{Depth of hard strata below the top of the soil}{Height of slope}$ ....*eq. 6.9*

The graph in Fig.6.6 is based on the most critical circle passing through the toe of the slope. For slope angle 'β' greater than53^0, the toe failure occurs. For $\beta \leq 53^0$ and small values ofφ_m, a more critical surface may pass below the toe. The graph in Fig. 6.7 is applied for$\emptyset_m = 0$. In soils, with $\emptyset_m = 0$ and the slope angle greater than53^0, the failure surface extends below the toe as deep as possible. The stability number also depends upon the parameter 'D_f'. Factor of Safety of a slope can be determined with the help of stability numbers (S_n). For the known value of 'β' and '$\emptyset_m$' the value of stability number (S_n) is determined from the chart in Fig. 6.6 or Table 6.2 and the Factor of Safety is determined as shown through eq. 6.10

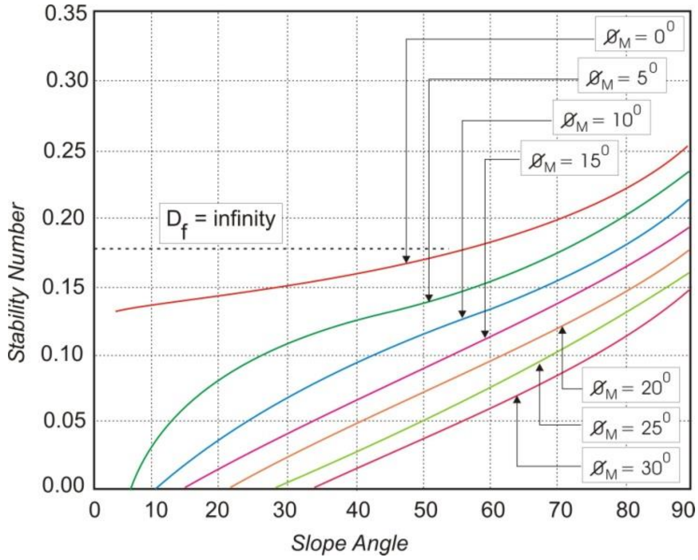


Fig. 6.6 Graph between stability no S_n and slope angle β

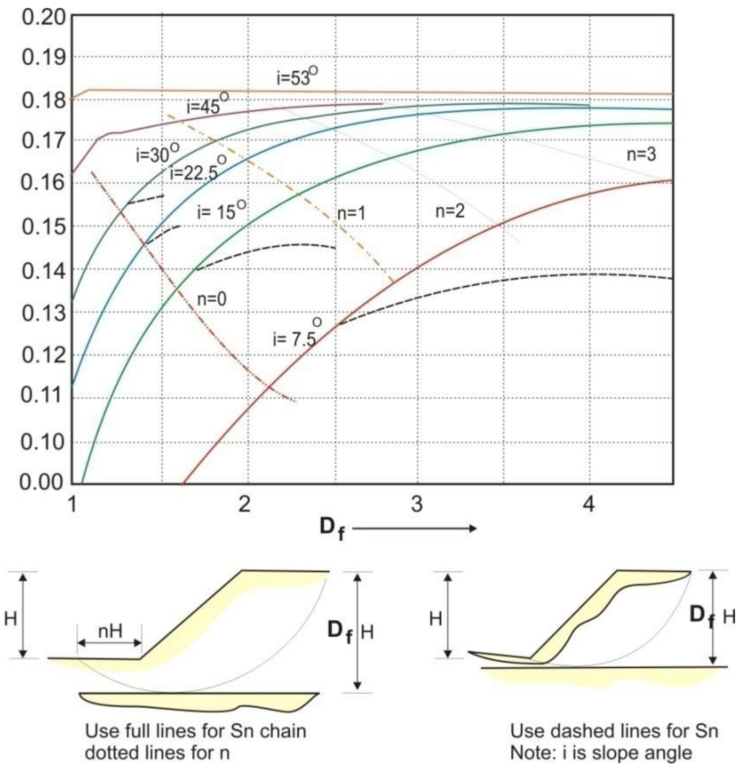


Fig. 6.7 Stability Chart ($\varnothing_m = 0$)

$$F_c = \frac{c}{c_m} = \frac{c}{S_n \gamma H} \quad \text{……………..eq.6.10}$$

If $\varnothing_m = 0$, the chart in Fig. 6.7 or Table 6.2 is used to determine the stability number (S_n) for the given value of 'β' and 'D_f'. The chart can be used to determine the distance 'nH' from the toe where the slip circle cuts the horizontal line (Arora, 1997; Ranjan, and Rao, 2002).

Table 6.1 Stability Numbers

Slope Angle 'β'	$\emptyset_m$					
	0^0	5^0	10^0	15^0	20^0	25^0
90^0	0.261	0.239	0.218	0.199	0.182	0.166
75^0	0.219	0.195	0.173	0.152	0.134	0.117
60^0	0.191	0.162	0.138	0.116	0.097	0.079
45^0	(0.170)	0.136	0.108	0.083	0.062	0.044
30^0	(0.156)	(0.110)	0.075	0.046	0.025	0.009
15^0	(0.145)	(0.068)	0.070	(0.023)	-	-
* Figures in brackets are for the most dangerous circles through the toe when a more dangerous circle exists below the toe						

Table 6.2 Stability Numbers for Cohesive soils ($\emptyset_m = 0$) and $\beta \leq 53^0$

Slope Angle 'β'	D_f				
	1.0	1.5	2.0	3.0	Infinity
53^0	0.181	0.181	0.181	0.181	0.181
45^0	0.164	0.174	0.177	0.180	0.181
30^0	0.133	0.164	0.172	0.178	0.181
22.5^0	0.113	0.153	0.166	0.175	0.181
15^0	0.083	0.128	0.150	0.167	0.181
7.5^0	0.054	0.080	0.107	0.140	0.181

The following points should be carefully noted;

i) If the Factor of Safety with respect to friction '$F_\emptyset$' is unity, $\emptyset_m = \emptyset$.

ii) If the Factor of Safety with respect to shear strength is required a trial and error procedure is adopted. A value of '$F_\emptyset$' is assumed and the value of '$\emptyset_m$' is used to determine 'F_c' from the stability chart.

If 'F_c' is not equal to assumed value of '$F_\emptyset$', another value of '$F_\emptyset$' is assumed and the procedure is repeated. At least 3-4 trials are required to obtain a curve between 'F_c' and '$F_\emptyset$', and to get the correct value of 'F_c' from the curves.

iii) For a submerged slope, the stability number is computed using the submerged unit weight (γ'). The angle of shearing resistance should also be for the submerged conditions.

iv) For a sudden drawdown case, the stability number is computed using the saturated unit weight (γ_{sat}).

The weighted angle of internal friction as obtained below is used for finding out the stability numbers.

$$tan\emptyset_w = \frac{\gamma'}{\gamma_{sat}}\left(\frac{1}{F_\emptyset} tan\emptyset'\right) \qquad \text{......................eq.6.11}$$

or $$\emptyset_w = \frac{\gamma'}{\gamma_{sat}}\emptyset_m = \frac{\gamma'}{\gamma_{sat}}.\left(\frac{\varphi'}{F_\varphi}\right) \qquad \text{......................eq.6.12}$$

where '$\emptyset'$' is the effective angle of internal friction.

v) For purely frictional soils, the cohesion intercept (C) is zero. As the stability number reduces to zero, the stability charts cannot be used for such soils.

vi) The values of shear strength parameters (C and $\emptyset$) should be obtained from the tests conducted in the laboratory simulating the drainage conditions in the field.

vii) For submerged slopes, the stability number S_n = ($C_{dev}/\gamma H$) can be calculated using γ_{sub} for γ; and angle of friction$\emptyset$ as equal to $\emptyset dev$ to calculate Fs. For examining the stability of slopes under sudden drawdown condition, however, the stability number should be calculated using γ_{sat} for γ and the value of $\emptyset$ also gets reduced in the ratio of $\gamma_{sub}/\gamma_{sat}$; and this value of $\emptyset$ is called the weighted angle of friction ($\emptyset\omega$) (Arora, 1997; Ranjan, and Rao, 2002).

Initial Design of Dam

Using the Taylor's stability approach, the initial design of Berga dam which includes providing the safe inclination angle for both upstream and downstream slopes of the embankment, was carried out and the dam section was prepared, as presented in Fig. 6.8.

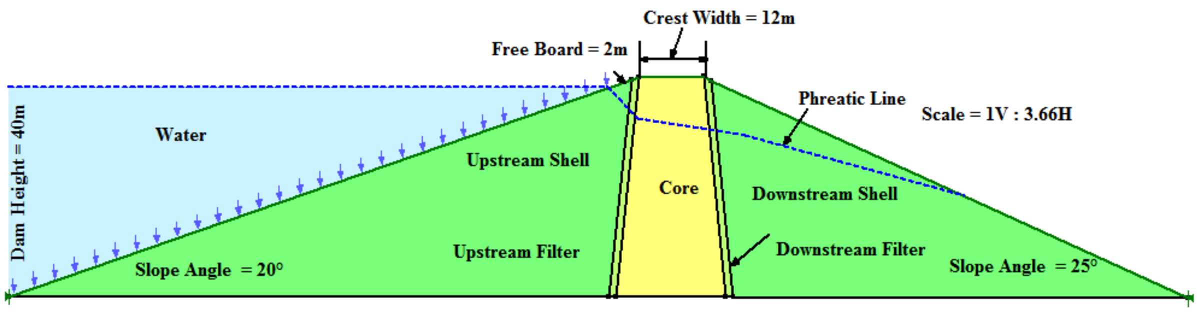


Fig. 6.8 Initial Design of Proposed Berga Embankment Dam

The design considerations for the initial design of Berga embankment dam is presented below;

(i) Determination of Slope angles for Dam

To begin the designing of the dam, first the angles of both upstream and downstream slope must be determined by using the stability number S_n and Taylor's chart;

$$Sn = \frac{Cdev}{\gamma H} \text{..................} \textit{eq. 6.13}$$

For the present design purpose parameters used are described below;

$$Cdev\ (developed\ cohesion) = \frac{C}{Fs}$$

Where; 'C' is the average cohesion and 'Fs' is the factor of safety which is taken as 1.5 (Garg, 2005).

γ = the average unit weightβ = slope angle H = height of the proposed dam

ϕ = angle of internal friction

$$\phi dev\ (developed\ angle\ of\ internal\ friction) = tan^{-1}\left(\frac{\tan\phi}{Fs}\right)$$

$$\frac{\gamma sub}{\gamma sat}\ x\ \phi dev = \ \phi\omega = Weighted\ angle\ of\ friction$$

Table 6.3 Calculated upstream and downstream slopes of the proposed embankment dam

Design Parameters	**Upstream**	**Downstream**
Average Cohesion 'C' (KN/m^2)	11	11
Design Factor of Safet 'Fs'	1.5	1.5
Proposed Dam Height 'H' (m)	40	40
Dry Unit weight 'γdry' (KN/m^3)	-	19.29
Saturated Unit Weight 'γsat' (KN/m^3)	21.823	-
Submerged unit weight 'γsub' (KN/m^3)	12.013	-
Angle of internal friction 'ϕ' (°)	20	20
Developed Cohesion 'Cdev'	7.33	7.33
Developed angle of internal friction 'ϕdev' (°)	13.64	-
Weighted angle of friction 'ϕω' (°)	7.5	-
Taylors Stability Number 'Sn'	0.0084	0.0095
Calculated Slope Angle 'β' (°)	**≃ 20**	**≃ 25**

(ii) Crest width of dam

The crest width is often governed by the requirement of transport during construction and after the dam construction. The minimum width adopted for even a small dam should be 3m. For high dams the crest width varies from 6 to 12m. The actual width of crest provided in USBR dams of up to 150 m height is given by; $W_c = 3.6H^{1/3}$ where, 'H' is the height of the dam in `m','$W_c$' is the crest width in `m'. For small dams up to 20 m height the suggested USBR formula is; $W_c = (0.2H + 3)$ m. Where, 'H' is the height of the dam in `m', '$W_c$' is the crest width in `m'. As per Japanese code, 1957 the crest width is given by; $W_c = (3.6\ H^{1/3} – 3)$ m. Large crest widths are provided in seismic areas to resist the larger accelerations near the top. A smaller width is provided in concrete faced rock fill dams than in earth core dams (Singh and Varshney, 1995).

The USBR practice for providing free board is kept 1.8m and the maximum freeboard of 3m for free spillway (USBR, 1987).

During the present study for the initial dam design the *crest width* was computed as per USBR Criteria;

$W_c = 3.6H^{1/3} = 3.6 \text{ x } (40)^{1/3} =$ **12 m,**

Where; 'W_c' is crest width and 'H' is the proposed dam height. For Berga Dam the proposed dam height is 40m.

(iii) Free Board

For the over topping the wave height was calculated so that the free board can be determined to protect the upstream side from erosion by possible wave action in the reservoir.

Formula for estimation of wave height is proposed by Stevenson and modified by Moliter to include wind velocity.

$hw = 0.032\ x\ \sqrt{VF} + 0.76 - 0.27\ x\ F^{1/4}$ ....*eq. 6.14*

Where, `hw' is the wave height in meters, `v' is the wind velocity in km/h, `F' is the "Fetch" in km.

Fetch is defined as the un-obstructed expanse of water over which the wind can blow to generate waves. It is taken as the longest straight distance from a point on the dam axis to the shoreline of the reservoir.

For the present study the free board was determined based on the possible water wave height that may be developed in reservoir due to wind action. The wave height was calculated by using eq. 6.14as;

$$hw = 0.032\ x\ \sqrt{(5x11)} + 0.76 - 0.27\ x\ (11)^{\frac{1}{4}} = 0.50m$$

where; `v', the wind velocity taken as 5km/h and `F', the "Fetch" taken as 11 km.
Therefore, in order to accommodate the possible water wave a safe free board must be **2 m.**

(iv) Phreatic Line

The phreatic line was calculated by using the formula proposed by Garg (2005);

$\sqrt{x^2 + y^2}$ = x + S………*eq. 6.15*

Where; ‘S’ is the distance of the point (x, y) from the toe and ‘S’ was found to have the value of 5.28 m (by taking the coordinate (134, 38) where (0,0) is the toe)

A few co-ordinates of the parabola at known distances (x) were worked out and are presented in Table 6.4.

Table 6.4 Calculated coordinates for phreatic line

x	y = $\sqrt{S^2 + 2xS}$
0	5.28
10	11.55
20	15.46
40	21.22
80	29.54
88	30.93
100	32.9
102	33.24
134	38

Therefore, by utilizing the calculated x, y coordinates (Table 6.4) the Phreatic line was constructed (Fig. 6.8).

Thus, as described above based on upstream and downstream slope angles, crest width, freeboard height and phreatic line computations initial design for Berga dam was prepared. Further, this dam design was checked for its stability under anticipated conditions to which it will be subjected during its performance stage. An elaborate description of stability analysis is presented in the following paragraphs.

6.3.2 Overall Stability of the Dam Section against failure due to horizontal shear

For determining the factor of safety against the foundation shear approximate method is used on the assumption that a soil has an equivalent liquid unit weight which would produce the same shear stress as the soil itself (Table 6.5).

The factor of safety against failure due to horizontal shear at base is given as;

F.S = R/P…………………………*eq. 6.16*

Where;

$R = C + W \tan\phi$

$P = ½ \gamma_w h^2$

R = shear resistance at the base

C = total cohesive strength of the soil at the base = c x B x 1 by considering 1m length

B = Width of the dam

W = total weight of the dam by considering 1m length

P = horizontal force = horizontal pressure of water

The stability of Berga dam was determined by using the above parameters and is presented in Table 6.5;

Table 6.5 Stability Calculation

<table>
<tr><td>Base of the dam (B) (m)</td><td>Total area of the dam (calculated from Fig. 6.8) (m^2)</td><td>Area above seepage line (m^2)</td><td>Area below seepage line (m^2)</td><td>Weight of dry portion of the dam (KN)</td><td>Weight of submerged portion of the dam (KN)</td><td>Total weight of the dam (W)</td><td>C (KN/m^2) =c x B x 1 by considering 1m length</td><td>ϕ (°)</td></tr>
<tr><td>210</td><td>4200</td><td>1100</td><td>3100</td><td>21,219</td><td>37,200</td><td>58419</td><td>2310KN</td><td>20</td></tr>
<tr><td>h (max. water level in the reservoir) (m)</td><td colspan="2">Shear resistance of the dam at the base
R = C+ W tan ϕ</td><td colspan="2">Horizontal force (Horizontal Pressure of water)
$P = ½ \gamma_w h^2$</td><td colspan="4">Factor of safety against failure due to horizontal shear at the base
F.S = R/P</td></tr>
<tr><td>38</td><td colspan="2">23572.7</td><td colspan="2">7082.82</td><td colspan="4">3.33</td></tr>
</table>

A perusal of results presented in Table 6.5 clearly indicates that factor of safety (F.S) against sliding is 3.33, which is much higher than the safe design F.S of 1.5. Thus, it can safely be concluded that the dam will be safe against failure due to horizontal shear at the base, even in its full operational reservoir capacity of MRL at 38m.

6.3.3 Stability of Upstream Slope (Under sudden drawdown)

This is based on a simple principle that a horizontal shear force (say P_u) is exerted by the saturated soil (i.e. by the soil as well as by the water contained within the soil). The resistance to this force (say R_u) is provided by the shear resistance developed at the base of the soil mass, contained within the upper slope which is expressed as follows;

$$Pu = [\left(\frac{\gamma 1.h^2}{2}\right) tan^2\left(45° - \frac{\phi}{2}\right) + \left(\frac{\gamma w.h1^2}{2}\right)] \qquad \text{.................. eq. 6.17}$$

Where; 'γ_1' is the weighted density at the center of the triangular shoulder upstream, given by

$$\gamma 1 = \frac{\gamma sub.h1 + \gamma dry(h - h1)}{h} \qquad \text{.................. eq. 6.18}$$

Shear resistance Rd of the downstream slope portion of the dam, developed at the base is given by;

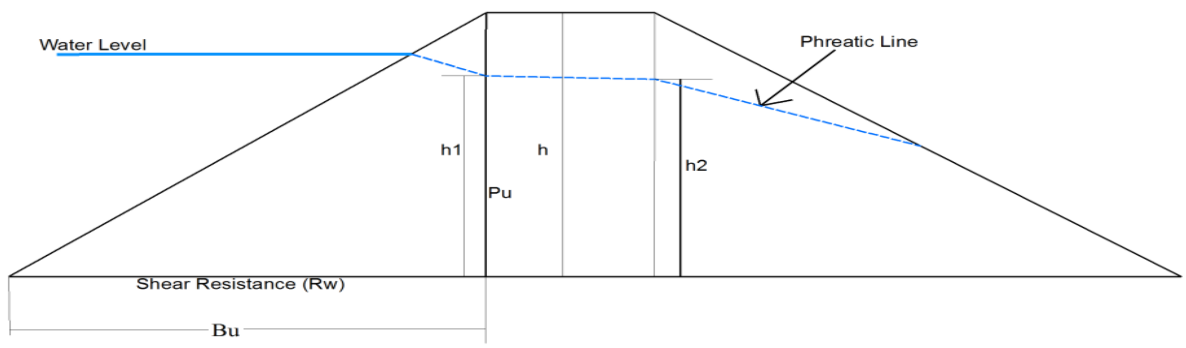


Fig. 6.9 Stability of upper slope

$Ru = C + W tan\phi$ ....*eq. 6.19*

Where W = the weight of the upstream slope portion of the dam which is;

$W = \gamma sub.\frac{1}{2}.Bu.h$ ....*eq. 6.20*

$C = c.Bu.1$ ....*eq. 6.21*

Where, c is the unit cohesion and Bu is the length of the base of the upstream slope

Therefore, the factor of safety will be;

$Fs = \frac{Ru}{Pu}$..................*eq. 6.22*

This value must be more than 1.5.

Using this formula the stability of the upper slope of Berga dam was calculated and is presented in Table 6.6;

A perusal of results presented in Table 6.6 shows that Factor of Safety for upstream slope under sudden drawdown condition comes out to be 0.88 which is far below the safe design F.S of 1.5. Thus, it may be concluded that the upper slope of the dam will not be safe under sudden drawdown condition.

Table 6.6 Stability analysis of Upstream Slope Under sudden drawdown condition

h	**h1**	**ϕ**	**c**	**γw**	**γsub**	**γdry**	**Bu**	**C**	**W**	**γ1**
40	38	20	11	9.81	12.013	19.29	107	1179.4	25707.82	12.37
Ru	**Pu**	**Fs**								
10536.3	**11931.86**	**0.88**								

The factor of safety calculated above is with respect to average shear (τ_{av}), which will be equal to;

$$\tau av = \frac{Pu}{Bu.1} \text{ ....eq. 6.23}$$

The maximum shear stress induced can be expressed as:

$$\tau \max = 1.4 \,.\, \tau av$$

And the factor of safety at the point of maximum shear is: $\frac{\tau f}{\tau max}$ where;

$$\tau f = c + 0.6 \,.h.\gamma sub.tan\phi \text{ ....eq. 6.24}$$

Using the above equations the stability of Berga dam at the point of maximum shear is presented in Table 6.7;

Table 6.7 Stability of upstream slope calculated at the maximum shear

Pu	Bu	τav	c	h	γsub	ϕ	τf	τmax	Fs
11931.86	**107**	**111.5**	**11**	**40**	**12.013**	**20**	**115.9**	**156.11**	**0.74**

A perusal of results presented in Table 6.7 shows that Factor of Safety for upstream slope under sudden drawdown condition with maximum shear comes out to be 0.74 which is far below the safe design F.S of 1.5. Thus, it may be concluded that the upper slope of the dam will not be safe under sudden drawdown condition.

6.3.4 Stability of Down Stream Slope during steady seepage

The factor of safety against the horizontal shear forces can be evaluated on the same principles as followed for the upper stream slope.

The horizontal shear force P_d is given as;

$$Pd = [\left(\frac{\gamma 2.h^2}{2}\right) tan^2\left(45^\circ - \frac{\phi}{2}\right) + \left(\frac{\gamma w.h2^2}{2}\right)] \text{ ....eq. 6.25}$$

Where; 'γ_2' is the weighted density at the center of the triangular shoulder downstream, given by;

$$\gamma 2 = \frac{\gamma sub.h2 + \gamma dry(h - h2)}{h} \text{ ....eq. 6.26}$$

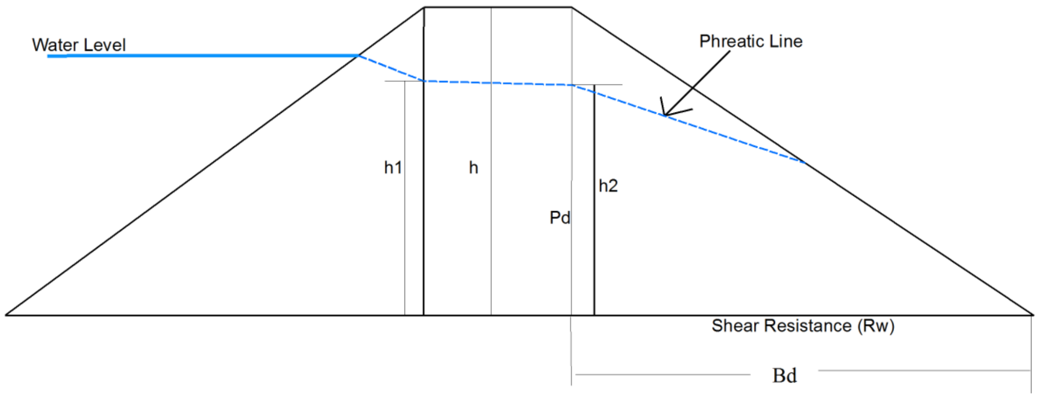


Fig. 6.10 Stability of Downstream slope

Shear resistance Rd of the downstream slope portion of the dam, developed at the base is given by;

$Rd = C + W tan\phi$ ………………*eq. 6.27*

Where W = the weight of the downstream slope portion of the dam which is;

$W = \gamma sub.\frac{1}{2}.Bd.h$ ………………*eq. 6.28*

$C = c.Bd.1$

Where; ‘c’ is the unit cohesion and Bd is the length of the base of the downstream slope Therefore the factor of safety will be;

$Fs = \frac{Rd}{Pd}$…………………*eq. 6.29*

This value must be more than 1.5.

Using above equations the stability of the downstream slope of Berga dam was calculated as presented in Table 6.8;

From the results presented in Table 6.8 it may be noted that the F.S of downstream slope of dam during steady seepage stage is 0.93 which is less than the design F.S. It may be concluded from the results that the downstream slope of dam during steady seepage stage will be unstable. Thus, it is required to provide proper drainage to the downstream slope of the dam so the destructive water forces do not develop during steady seepage stage.

The factor of safety calculated above is with respect to average shear (τ_{av}), which will be equal to;

Table 6.8 Stability analysis of down Stream Slope Portion of the Dam (during steady seepage)

h	h2	ϕ	c	γw	γsub	γdry	Bd	C	W	γ2
40	**26**	**20**	**11**	**9.81**	**12.013**	**19.29**	**85**	**935**	**20,422.1**	**14.56**
Rd	Pd	Fs								
8368.03	**9023.3**	**0.93**								

$\tau av = \frac{Pd}{Bd.1}$………………….*eq. 6.30*

The maximum shear stress induced can be expressed as:

$\tau \max = 1.4 . \tau av$………………….*eq. 6.31*

And the factor of safety at the point of maximum shear is: $\frac{\tau f}{\tau max}$ where;

$\tau f = c + 0.6 . h. \gamma sub. tan\phi$………………….*eq. 6.32*

Using the above equations the stability of Berga dam at the point of maximum shear is presented in Table 6.9;

Table 6.9 Stability of downstream slope calculated at the maximum shear

Pd	Bd	τav	c	h	γsub	ϕ	τf	τmax	Fs
9023.3	**85**	**106.16**	**11**	**40**	**12.013**	**20**	**115.9**	**148.62**	**0.78**

From the results presented in Table 6.9 it may be noted that the F.S of downstream slope of dam during steady seepage stage with maximum shear is 0.78 which is less than the design F.S. It may be concluded from the results that the downstream slope of dam during steady seepage stage with maximum shear will be unstable.

6.3.5 Factor of safety with different failure surfaces using SLOPE/W software;

Failure of an embankment dam can result from instability of either upstream or downstream slopes. The failure surface may lie within the embankment or may pass through the embankment and the foundation soil. The critical stages in an upstream slope are at the end of construction and during rapid drawdown. The critical stages for the downstream slope are at the end of construction and during steady seepage when the reservoir is full.

It is common to install piezometers to measure pore water pressures and compare data with the predicted values used in design. Since pore water pressures has a dominant influence on the factor of safety of slopes, remedial action should be taken if the factor of safety, based on the measured values, is considered to be too low. The safety against failure can be increased by reducing the gradient of the slopes (Ghassan, 2002).

According to Gedeon (2004), the process of evaluating slope stability involves the following chain of events;

(i) Explore and sample foundation and borrow sources.
(ii) Characterize the soil strength.
(iii) Establish the 2-D idealization of the cross section, including the surface geometry and the sub-surface boundaries between the various materials.
(iv) Establish the seepage and groundwater conditions in the cross section as measured or as predicted for the design load conditions.
(v) Select loading conditions for analysis.
(vi) Select trial slip surfaces and compute factors of safety.
(vii) Repeat step (vi) above until the “critical” slip surface has been located.
(viii) Compare the computed factor of safety with experienced-based criteria.
(ix) The specifications should be written consistent with the design assumptions.
(x) The design assumptions should be verified during construction. This may require repeating steps (ii), (iii), (iv), (vi), (vii) and (viii) and modifying the design if conditions are found that do not match the design assumptions.
(xi) Following construction, the performance of the completed structure should be monitored.

During the present study SLOPE/W software was used to calculate the factor of safety for presumed different failure surfaces so that the critical surface can be identified. SLOPE/W is one component in a complete suite of geotechnical products called Geo Studio. SLOPE/W has been designed and developed to be a general software tool for the stability analysis of earth structures (GEO-SLOPE International, 2008). The analysis was performed both for the upstream and downstream slopes separately.

A detailed description on stability analysis for upstream and downstream slopes of proposed Berga dam by utilizing SLOPE/W software is presented in following section;

For the slope stability analysis the input parameters used are cohesion, phi, and unit weight of the core, filter and shell materials (Table 6.10). The values used for the analysis were the average for all the materials. The values for cohesion and angle of shearing resistance were taken from the standard values proposed by USBR (1987) based on soil classification. The value for unit weight was taken from the laboratory test results. The analysis performed by SLOPE/W considers the condition when the reservoir is full and there is no effect of dynamic forces i.e. static fully saturated condition. In the present study it was not possible to use the software to analyze dynamic conditions. Therefore, the resent failure surfaces occur when the dam is full and no external force (dynamic forces) are present.

Table 6.10 Input Parameters used in SLOPE/W analysis

No.	Parameter	Value Taken	Slope
1.	Cohesion (kpa)	11	Upstream
2.	Phi (°)	30	Upstream
3.	Unit Weight (kN/m^3)	12	Upstream
4.	Slope Height (m)	40	Upstream
5.	Slope Width (m)	115	Upstream
6.	Slope angle (°)	20	Upstream
7.	Phreatic line level (m)	38	Upstream
8.	Cohesion (kpa)	11	Downstream
9.	Phi (°)	30	Downstream
10.	Unit Weight (kN/m^3)	12	Downstream
11.	Slope Height (m)	40	Downstream
12.	Slope Width (m)	95	Downstream
13.	Slope angle (°)	25	Downstream
14.	Phreatic line level (m)	33	Downstream

The analysis by using SLOPE/W was performed for both upstream and downstream slopes of the initial design. For the analysis of the upstream slope about 48 slip surfaces were analyzed. As it can be seen from Table 6.11, all the slip surfaces have a factor of safety greater than 1.5. This show that the slip surfaces can be stable even under full reservoir condition when there will be no dynamic force effect. One reason for this could be, as it can be seen in Fig. 6.11 the slope has low angle and the impounded water weight acts against the slope. This condition may increase the stability of the slope.

For the downstream slope, similarly, 48 slip surfaces were analyzed for stability (Fig. 6.12). All the slip surfaces, as it can be seen from Table 6.12 have a factor of safety less than 1.5. This shows that the downstream slope will not be stable under full reservoir level even in the absence of dynamic force. One of the reasons for this instability could be, the slope has higher angle than the upstream side. As the slope angle increases the stability of the slope decreases. The material quality could also be the other reason as the material is highly plastic. In this regards, probably the seeping water will have much effect on such soil.

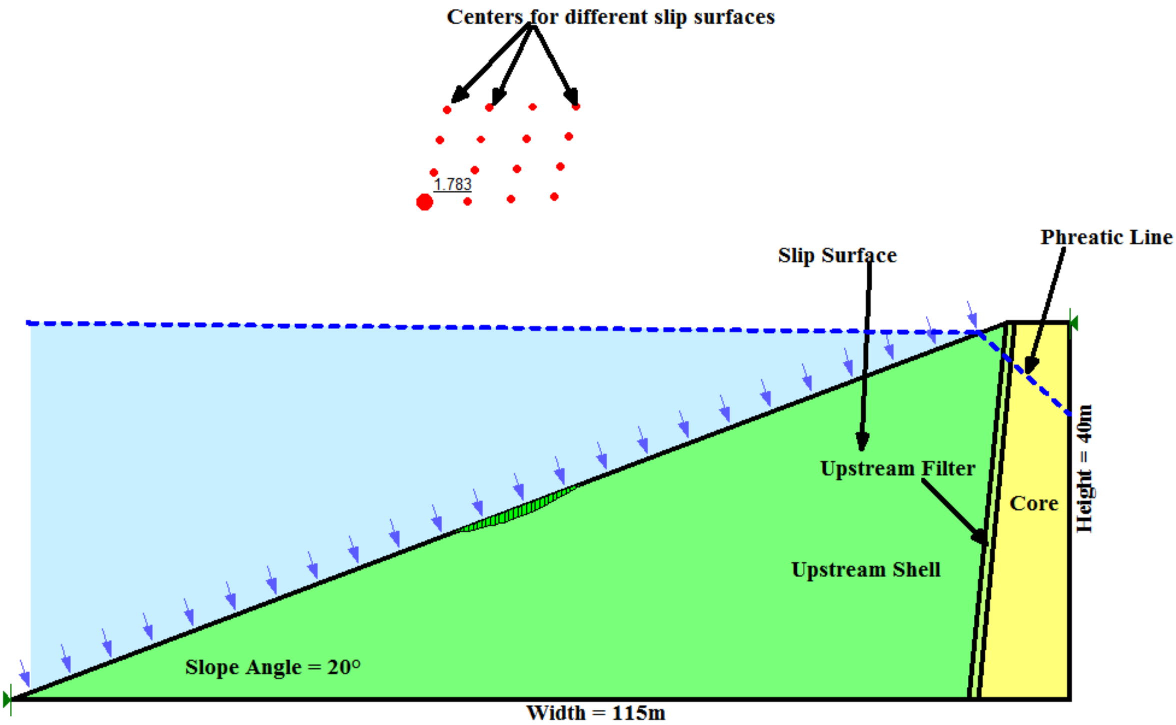
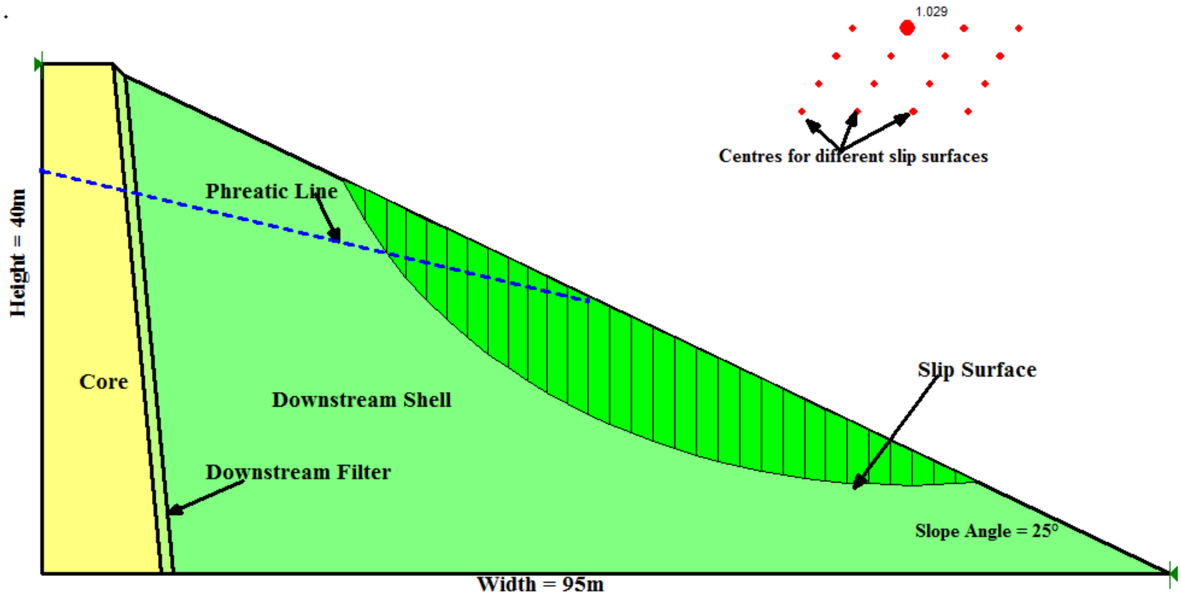


Fig. 6.11 SLOPE/W result of upstream slope (with Critical slip surface for illustration)



(Note: The critical slip surface is too small to be seen in this diagram)

Fig. 6.12 SLOPE/W result of downstream slope

Fig. 6.13 and Fig. 6.14 show the graphs drawn from the result of the analysis by SLOPE/W. The graph for the upstream slope shows that as the distance from the reservoir water increases towards the core, the pore water pressure decrease. This shows that the effect of the impounded water is very high at specific sites on the slope. As a result only a part of the upstream slope is susceptible to failure that will be caused by the water pressure.

Table 6.11 Slip surfaces with factor of safety of upstream slope

Slip #	F.S	X Center	Y Center	Radius	Remark
1	1.783	45.047	48.028	32.024	Least Factor of Safety – Critical Slip Surface
13	1.798	45.823	50.992	34.988	
25	1.811	46.599	53.955	37.951	
37	1.821	47.375	56.919	40.915	
4	1.86	49.702	48.184	32.18	
16	1.869	50.478	51.148	35.144	
28	1.876	51.254	54.111	38.107	
40	1.883	52.03	57.075	41.071	
7	1.94	54.357	48.34	32.336	
19	1.943	55.133	51.304	35.3	
31	1.945	55.909	54.267	38.263	
43	1.946	56.685	57.231	41.227	
38	1.984	47.375	56.919	45.361	
26	1.985	46.599	53.955	42.397	
14	1.987	45.823	50.992	39.433	
2	1.988	45.047	48.028	36.47	
46	2.013	61.339	57.387	41.383	
34	2.016	60.564	54.423	38.419	
22	2.02	59.788	51.46	35.456	
10	2.025	59.012	48.496	32.492	
41	2.046	52.03	57.075	45.517	
29	2.052	51.254	54.111	42.553	
17	2.058	50.478	51.148	39.589	
5	2.066	49.702	48.184	36.626	
44	2.11	56.685	57.231	45.673	
32	2.12	55.909	54.267	42.709	
20	2.132	55.133	51.304	39.745	
39	2.134	47.375	56.919	49.806	
27	2.145	46.599	53.955	46.843	
8	2.147	54.357	48.34	36.782	
15	2.158	45.823	50.992	43.879	
3	2.173	45.047	48.028	40.915	
47	2.177	61.339	57.387	45.829	
35	2.192	60.564	54.423	42.865	
42	2.196	52.03	57.075	49.962	
23	2.21	59.788	51.46	39.901	
30	2.212	51.254	54.111	46.999	
18	2.23	50.478	51.148	44.035	
11	2.232	59.012	48.496	36.938	
6	2.25	49.702	48.184	41.071	
45	2.26	56.685	57.231	50.118	
33	2.281	55.909	54.267	47.155	
21	2.304	55.133	51.304	44.191	
48	2.326	61.339	57.387	50.274	
9	2.331	54.357	48.34	41.227	
36	2.352	60.564	54.423	47.311	
24	2.382	59.788	51.46	44.347	
12	2.416	59.012	48.496	41.383	

Table 6.12 Slip surfaces with factor of safety of downstream slope

Slip #	F.S.	X Center	Y Center	Radius	Remarks
46	0.926	82.289	60.351	44.573	**Critical**
34	0.934	80.856	57.295	41.517	
22	0.942	79.423	54.24	38.461	
10	0.953	77.99	51.184	35.406	
43	0.985	77.631	60.351	44.573	
31	0.997	76.198	57.295	41.517	
19	1.012	74.765	54.24	38.461	
7	1.03	73.331	51.184	35.406	
40	1.047	72.973	60.351	44.573	
47	1.054	82.289	60.351	50.502	
28	1.065	71.54	57.295	41.517	
35	1.071	80.856	57.295	47.446	
16	1.087	70.107	54.24	38.461	
23	1.09	79.423	54.24	44.391	
11	1.112	77.99	51.184	41.335	
4	1.113	68.673	51.184	35.406	
44	1.113	77.631	60.351	50.502	
37	1.113	68.315	60.351	44.573	
32	1.134	76.198	57.295	47.446	
25	1.138	66.882	57.295	41.517	
20	1.159	74.765	54.24	44.391	
13	1.167	65.448	54.24	38.461	
48	1.168	82.289	60.351	56.431	
41	1.175	72.973	60.351	50.502	
8	1.188	73.331	51.184	41.335	
36	1.191	80.856	57.295	53.375	
29	1.202	71.54	57.295	47.446	
1	1.202	64.015	51.184	35.406	
45	1.206	77.631	60.351	56.431	
42	1.216	72.973	60.351	56.431	
24	1.217	79.423	54.24	50.32	
39	1.227	68.315	60.351	56.431	
33	1.227	76.198	57.295	53.375	
17	1.233	70.107	54.24	44.391	
38	1.238	68.315	60.351	50.502	
30	1.241	71.54	57.295	53.375	
12	1.248	77.99	51.184	47.264	
21	1.253	74.765	54.24	50.32	
27	1.254	66.882	57.295	53.375	
26	1.267	66.882	57.295	47.446	
18	1.27	70.107	54.24	50.32	
5	1.271	68.673	51.184	41.335	
9	1.282	73.331	51.184	47.264	
15	1.29	65.448	54.24	50.32	
14	1.301	65.448	54.24	44.391	
6	1.304	68.673	51.184	47.264	
3	1.334	64.015	51.184	47.264	
2	1.342	64.015	51.184	41.335	

On the other hand, the pore water pressure on the downstream side will increase up to a certain distance and then it will become constant. This shows that during the filling of the reservoir the downstream side of the dam will be subjected to high pore water pressure as the water seeps to the downstream side. However as the distance from the reservoir increases the water pressure drops and it will become constant.

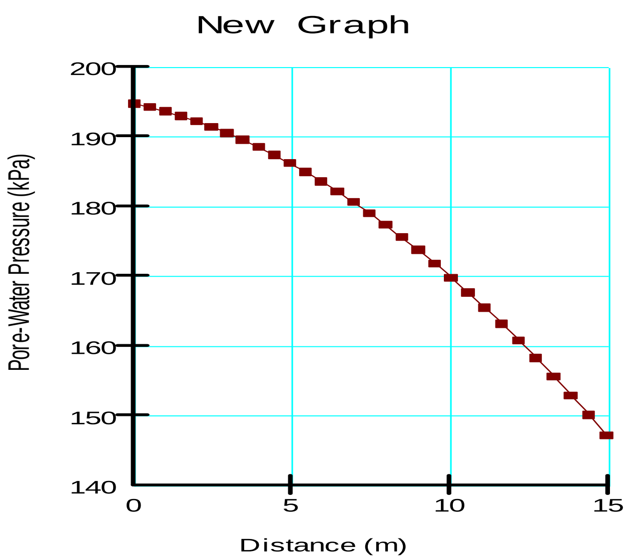


Fig.6.13 Graph showing pore water pressure vs distance of the upstream slope

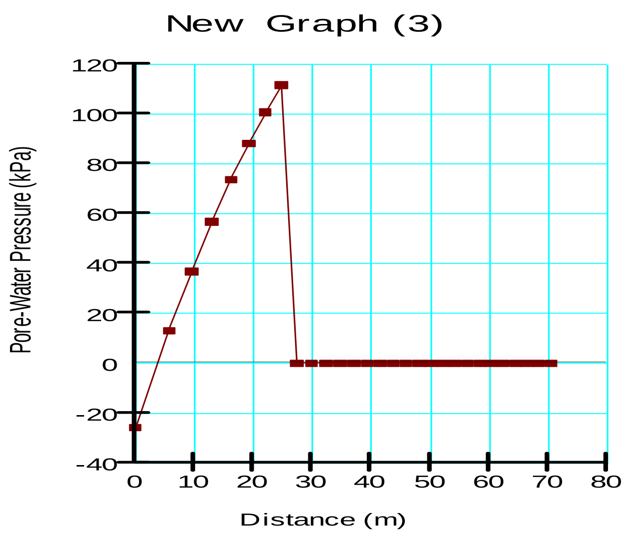


Fig.6.14 Graphs showing pore water pressure vs distance of the downstream slope

This shows that the effect of water pressure is effective at specific distance from the reservoir water. From this it can be concluded that as the distance from the reservoir increase the possibility of failure that could be caused by the water pressure will decrease.

6.3.6 Stability analysis of dam by utilizing SARC Software

This program facilitates to compute the factor of safety with circular failure surface emerging at the toe. The factor of safety is computed using Bishop's equation for various slip surfaces until a minimum factor of safety is obtained. The input parameters used in analysis and results obtained from SARC analyses are presented in Fig. 6.15, Fig. 6.16, Table 6.13, Table

6.12 and Table 6.13. In the analysis the conditions considered are; static condition with dry, moderately saturated and fully saturated water conditions and dynamic conditions with dry, moderately saturated and fully saturated. For the downstream slope as full saturation is not expected instead of full saturation, 80% saturation is considered.

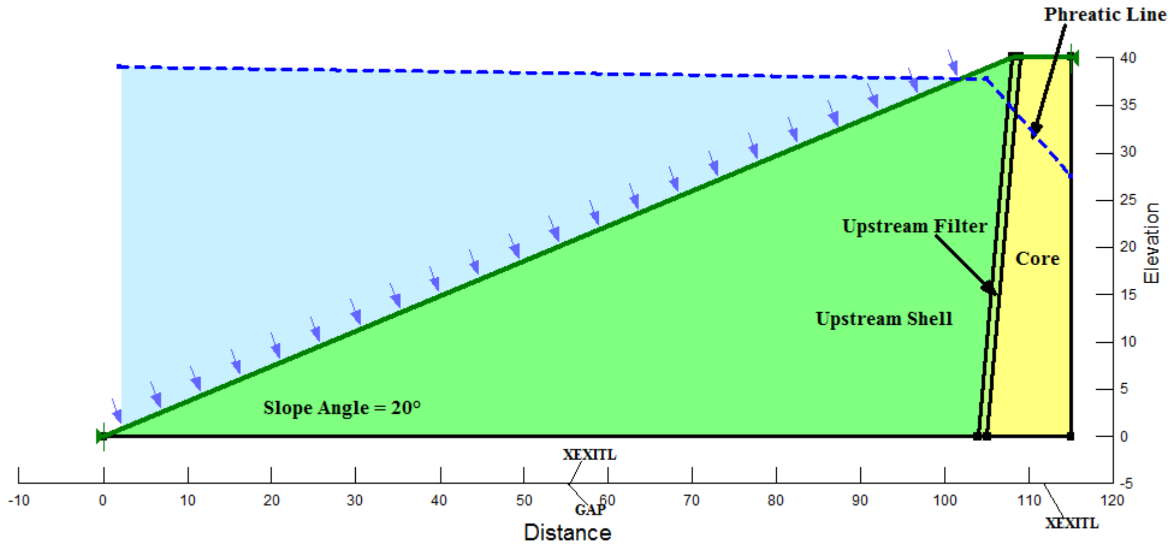


Fig. 6.15 Upstream slope with coordinates

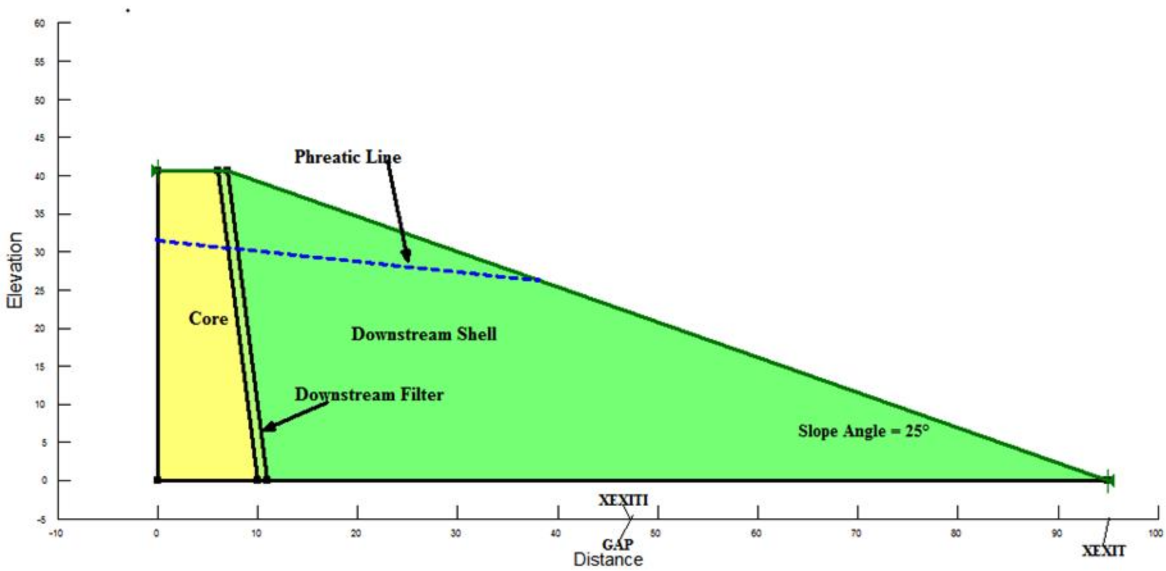


Fig. 6.16 Downstream slope with coordinates

Both the upstream and downstream slopes were analysed by using the SARC software (Table 6.14 and Table 6.15). From the analysis it is found that the upstream slope becomes unstable during all anticipated conditions. The downstream slope will also become unstable in all anticipated conditions except during static dry conditions. As the material properties used for the analysis are unsuitable for construction of the dam, the results confirm the need for improvement of the materials.

Table 6.13 Input parameters used to calculate factor of safety for static and dynamic conditions for the embankment slope

Input Parameters		
Parameter	**Value**	**Parameter description**
N	2	N = Number of profile coordinates(< 50)
X(I),Z(I) I =	1 to N	(X,Z) = Coordinates of profile points (X(I) < X(I+1))
RWL =	38 (US) & 33 (DS)	C = Cohesion of soil/rock
C =	11	PHI = Angle of internal friction (Degree) of soil/ rock
PHI =	30	GAMA = Unit weight of soil/rock
GAMA =	1.9	GAMAW = Unit weight of pore water
GAMAW =	1.0	BBAR = Pore water pressure/(GAMA * Average height of slices)
BBAR =		AH = Horizontal component of EQ. acceleration near crest of slope
AH =	0.2	AVR = Vertical component of EQ. acceleration / AH
AVR =	1.0	EQM = Corresponding EQ. magnitude on Richter Scale
EQM =	8	NENP = Number of entry points of slip circles (<10)
NENP = 1		ENTX = X-Coordinate of entry point of circle
ENTX (I), ENTY (I) =		ENTY = Y-Coordinate of entry point of circle
(0.0,0.0)		NOPT = 0 ,When only minimum factor of safety is required
NEP = 0.0		= 1,When all F.S. corresponding to all exit points are also required
NOPT = 0.0		NEP = Number of exit points(<50)
XEXITI =	55(US) &48(DS)	= 0 , When no individual point is given)
XEXITL =	111(US) &95(DS)	XEXITI= X-Coordinate of first exit point of circle
GAP =	55(US) &48(DS)	XEXITL= X-Coordinate of last exit point of circle
		GAP = Horizontal distance between consecutive exit points
		XEXIT = X-Coordinate of exit point of circle

Table 6.14 Results of stability analysis (Upstream slope)

Anticipated Conditions	**FOS**	**Weight**	**Coordinate of center**	**Exist point**	**Radius**
Static Dry	4.2287	$.13x10^5$	94.89,136.81	110,39.64	98.34
Static Moderately Saturated	2.1539	$.94x10^4$	94.89,136.81	110,39.64	98.34
Static Fully Saturated	0.8901	$.96x10^4$	94.89,136.81	110,39.64	98.34
Dynamic Dry	2.5112	$.17x10^5$	94.89,136.81	110,39.64	98.34
Dynamic Moderately Saturated	1.1385	$.14x10^5$	94.89,136.81	110,39.64	98.34
Dynamic Fully Saturated	0.4275	$.14x10^5$	94.89,136.81	110,39.64	98.34
FOS = Factor of Safety,					

Table 6.15 Results of stability analysis (Downstream slope)

Anticipated Conditions	**FOS**	**Weight**	**Coordinate of center**	**Exist point**	**Radius**
Static Dry	3.4128	$.13x10^4$	2.14,107.84	86,40	107.86
Static Moderately Saturated	1.7096	$.11x10^4$	2.14,107.84	86,40	107.86
Static 80% Saturated	0.7530	$.84x10^3$	2.14,107.84	86,40	107.86
Dynamic Dry	1.2157	$.13x10^4$	1.71,108.77	86,40	108.79
Dynamic Moderately Saturated	0.5527	$.98x10^3$	-4.63,122.41	86,40	122.50
Dynamic 80% Saturated	0.3974	$.49x10^3$	-31.01,179.11	86,40	181.78
FOS = Factor of Safety,					

The stability analysis of the prepared initial design of the dam, many conditions were anticipated (Table 6.16).

The dam design will be stable when there is no water in the reservoir and no dynamic loading takes place. This shows that the initial design is proper as it was prepared by following standard procedures. However, when the reservoir starts to fill, the stability of the dam will be reducing under varied water saturation. As it is observed from the results of the analysis by SLOPE/W and SARC software, the factor of safety will decrease under anticipated adverse conditions which may possibly result in to the failure of the slopes. Such instability may impart due to material incapability to be stable under anticipated adverse site conditions at the specified slope angles. This probably may be improved by reducing the plasticity characteristics of the materials and by improving the filter materials so that no destructive pore pressure can develop which may likely increases the chances for failures of the dam. However, to see such type of improvement in the slope stability condition it may require further investigation to either find the alternative construction material with better quality or to attempt blending of existing material with other material.

Table 6.16 Summary of factor of safety of different conditions and analyses

Conditions and Software			Factor of Safety
Due to horizontal shear			3.3
Under sudden drawdown	**Average Shear**		0.88
	Maximum Shear		0.74
During steady seepage	**Average Shear**		0.93
	Maximum Shear		0.78
SLOPE/W Software Analysis	**Upstream Slope**		1.783
	Downstream Slope		0.926
SARC Software Analysis	**Upstream Slope**	**Static – Dry**	4.2287
		Static – Moderately Saturated	2.1539
		Static – Full Saturated	0.8901
		Dynamic – Dry	2.5112
		Dynamic – Moderately Saturated	1.1385
		Dynamic – Full Saturated	0.4275
	Downstream Slope	**Static – Dry**	3.4128
		Static – Moderately Saturated	1.7096
		Static – 80% Saturated	0.7530
		Dynamic – Dry	1.2157
		Dynamic – Moderately Saturated	0.5527
		Dynamic – 80% Saturated	0.3974

6.3.7 Other conditions that must be considered during Stability Analysis

Other than the above discussed conditions seepage through the dam and the foundation, the effects of piping on the dam structure and earth quake effects must be determined in detail.

These can be done by using flow nets, seepage calculation and detailed study on liquefaction.

6.4 Construction of embankment dams

Below general recommendations are given which may be considered during and after the construction of embankment dams (IIT, 2006).

6.4.1 River diversion

Arrangements have to be made to divert the river while constructing an embankment dam. This temporary exclusion of river flow is necessary to provide dry or semi-dry area for the work to continue.

6.4.2 Foundation preparation

Basically, the surface under the entire core and under a portion of the upstream filter and downstream transition zone shall be completely excavated to such rock as will offer adequate resistance to erosion of fines in the core. All loose or semi-detached blocks of rock should be removed. The quality of rock shall be judged characteristic of core material.

6.4.3 Treatment of rock defects and discontinuities

In evaluating and planning for excavation and seepage control measures, special attention shall be given to discontinuities such as faults and relief (sheet) joints, which may extend for long distance as nearly plane surfaces.

6.4.4 Grouting

There are three main objectives in the grouting programme. These are: to reduce the seepage flow through the dam foundation; to prevent possible piping or washing of fines from the core into cracks and fissures in the foundation; and to reduce the hydrostatic pressure in the downstream foundation of the dam.

The latter is generally a problem only for dams on fairly weak foundations and critical abutment configurations. This is usually accomplished in conjunction with an abutment drainage system.

6.4.5 Quality Control

The performance of an earth or rock fill dam depends upon the control exercised during construction, supervision and inspection.

An entirely safe design may be ruined by careless and shoddy execution. Proper quality control during construction is as important as the design. The skill, experience and judgment required of the engineer in charge of construction, is in no way lesser than that of the design engineer.

6.4.6 Instrumentation

After the completion of the dam different instruments must be installed so that the dam can be monitored. If there are any problems like Pore Pressure, Seepage, Strains and Stresses, Dynamic Loads (Earthquakes), Vertical and horizontal movements they can be solved quickly before the caused damages on the dam structure.

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Chapter 7 Conclusion and Recommendations

7.1 Conclusion

Berga earth dam is proposed to be constructed across the river Berga, a tributary of Awash River. It is located nearly 11.5 km north of Holeta town. The Berga dam could be earth dam or a rock fill dam with a height of 40 m and 250 m crest length. The main objective of the present study was to characterize and assess the suitability of the construction material proposed to be used for various zones of the embankment. Other objective of the present research work was to characterize the dam site for its general suitability.

In order to characterize the construction material different tests were performed which includes; grain size analysis, Atterburg limits, dry density and moisture content determination. Besides, different empirical formulas and charts were also utilized to determine the properties like; permeability, liquefaction potential, possible clay minerals in the core samples, the swelling potential of the materials, activity and permeability of the core, filter and shell materials.

In the present study, the dam foundation area was characterized using the RMR classification for the left abutment and rotational failure analysis for the right abutment. Further, analysis was made to characterize the embankment construction material and to assess its suitability for the construction of dam. Detailed analyses were performed for core, filter and shell materials using engineering properties, mineralogical composition, dynamic behaviour/ properties and its relative response to the site conditions. Attempt was also made to prepare the initial design of the proposed Berga dam by using both laboratory results and standard values. Later, the design of the dam was analyzed for its stability under existing and anticipated site conditions.

During the present research work, the engineering geological map of the proposed dam site and its surroundings was prepared where engineering geological properties of rocks and soils were considered. According to the investigation carried out at the dam site, it was found that the site is mainly composed of fractured basalt and two soil types which were classified as CH and MH according to the USCS.

Based on the previous and present works the rock mass found at the proposed dam site possesses high rock mass strength. The main rock type in the area is basalt which is either porphyritic or aphanitic. Both the abutments consist of highly disintegrated rock mass. According to the results of point load test the upper basalt, which the dam foundation is composed, has rock material strength that ranges from 6.3 – 8.2 Mpa. The unconfined compressive strength calculated from point load strength value ranges from 151.2 to 196.8 Mpa. From point load and unconfined compressive strength values, the upper basalt is classified as rock with high mass strength. Generally, the rocks exposed on abutment have RQD with ratings ranging from 13 to 17 according to the rock mass rating system.

The discontinuities in the rock mass exposed on the left abutment in general have spacing up to 4 cm wide and were observed to be rough, with no infillings and weathering. This condition shows that the rocks are very strong and may not easily weather. The roughness and waviness of the rock mass may increase the shear strength between the adjacent rocks which may provide stability to the slopes of the abutments. At the time of investigation both the abutments were completely dry. Further, based on the analysis performed by using the Markland criteria, the left abutment is stable from both plane and wedge mode of failure.

By using RMR values the shear strength parameters and modulus of deformation of rock mass exposed on the left abutment was estimated. The cohesion, angle of internal friction and modulus of deformation comes out to be 3.425 Mpa, 39.25° and 3.7 x 10^{-5} Kg/cm^2, respectively. Hoek and Brown Failure Criteria were also used to calculate the cohesion and angle of internal friction which were found to be 3.59 MPa and 39.13°, respectively. During the present study, it was found that both the abutments are covered by thin soil cover and disintegrated rocks. It is anticipated that such condition may help to develop destructive pore water pressures with the rock mass and in general there will also be susceptibility to seepage during operational stage.

The discontinuities within rock mass exposed on left abutment has the preferred orientation of N290°/44°, N018°/66° and N206°/18° and the abutment slope is orientation 10° towards N270°. The stability analysis of right abutment revealed that it will be stable only for static dry condition and for anticipated worst conditions the abutment slope would be unstable.

The laboratory results for the soils from the dam site revealed that 80% of the sample is composed of silt and clay size particles. The remaining is the sand sized particles. Further, the soils have liquid limit 50, plastic index of 14 and free swell value of 40%. The foundation soils of the dam site do not have grain size equivalent to 0.005mm and the soils have liquid limit of 43. Thus, according to Day's (2006) criteria the soil in the foundation area is slightly susceptible to liquefaction.

During the present study the engineering geological map was prepared for the dam site and its surrounding areas. For this purpose the rocks at the dam site were classified. The uniaxial compressive strength of rocks was determined from the point load test values. Whereas, the weathering conditions and the ground water conditions were characterized visually. Finally, the RMR values of the rock mass were determined. According to Bieniawiski's Geomechanics classification system in general the rock mass at dam site falls in Good Class.
In terms of bearing capacity, both the abutments have high capacity as they are composed of very strong basalt rock. Based on RMR values, especially on the left abutment, the rock is classified as 'good' which makes it suitable for dam foundation. However, in general the seepage potential on the abutments is low except where the rock mass is weathered and fractured.

From the investigations conducted during present study it was found that the dam site in its present condition possesses weakness in terms of seepage potential and abutment slope stability condition. Thus, in order to improve the foundation condition at the dam site it is required to strip off the alluvial soil and highly disintegrated rock on the abutments. More detailed studies on permeability condition, particularly in underlying fractured basalt, is also required. For this systematic water pressure tests needs to be conducted and accordingly proper remedial measures to improve the permeability conditions of foundation rocks may be worked out.

In the present study previously selected borrow and quarry areas by GSE were further studied. Attempts were made to assess the general suitability of the construction material proposed for the various zones of the abutments. For this representative samples were collected from the borrow areas and laboratory test were conducted to know the grain size distribution, Atterberg limits, moisture content and dry density. The test results revealed that the in general soils have liquid limits in the range of 43 to 75 and plasticity index varies from 10.7 and 41. The dry density and moisture content of soils from borrow Area-2 and Area-6 on an average is 1.7 gm/cc and 14.3 %, respectively. Also, the free swell ranges from 45 to 70 %. The UCS of the soils ranges from 3.5 to 13 kg/cm^2. Further, the soils on an average have Cohesion (C) equal to 11Kpa and angle of shearing resistance (ϕ) equal to 20°.

The grain size distribution analysis in general shows that the proposed core material is composed of 4.4% sand, 45.4% silt and 50.2% clay. The clays from proposed borrow areas shows medium to high expansive property, except one sample. In general, the soils have free swell value ranging from 45 to 70%. Thus, according to Indian Standard (I.S. 1498-1970) 1 sample fall in the category of non-critical and 5 samples are marginal in terms of degree of severity. Based on the activity analysis, 2 samples are inactive while the rest are normal.

During present study the permeability of core material was estimated by using Allen Hazane's formula and on an average it was found to be 6.6 x 10^{-5} cm/sec. The flexibility and erosion resistance analysis for core material revealed that soil samples ATP - 2B, ATP – 3, ATP – 5 and ATP – 7 are classified as 'Poor' material and ATP – 4, ATP – 4 and BD – 4 as 'Good' core material as per Singh Criteria.

The sample BD – 12 from dam axis was classified as ML according to the USCS and in general it possesses plasticity. However, the XRD result shows that this soil has about 73% of quartz and 9.8% of kaolinite. The high proportion of quartz in the soil sample contradicts to the expansive potential of the soil. Here, it also doest seems reasonable to presume that kaolinite alone which is around 10% in the sample have enough proportion to make soil sample expansive in nature. Thus, it requires further study to understand the fact that why the soils are expansive with such a high proportion of quartz and reasonably low proportion of expansive mineral Kaolinite. Though, high proportion of quartz in soil sample may be because of external sources as the Berga River has tributaries which flow through Mugher valley and it is very likely that the source for quartz may be sandstone found in Mugher valley.

The plot of 5 samples from clay borrow area ATP - 2, 4, 6 samples falls within MH, ML and Kaolinite region. Whereas, 2 samples from area ATP-3 and ATP-5 falls within CH region and Illite zone on plasticity chart.

According to the criteria proposed by Day (2006), the soils of the study area are not susceptible for liquefaction. As the dam site is located near to the active Ethiopian rift, care must be taken during construction especially considering the liquefaction property of the dam materials.

For the proposed Berga dam, there is only one borrow site for suitable filter material at economic distance. In order to determine the engineering property of the filter material different test were performed. According to the grain size analysis of the samples of the filter material it was found that about 50% of it is sand and the remaining is fines. This composition doesn't entirely satisfy the filter criteria.

Further, the test results for gradation and classification of Filter Material indicate that the soil is SM as per the USCS. The Filter material in general has uniformity coefficient (Cu) and coefficient of curvature (Cc) 97.7 and 4.48, respectively. Sands with a value of 'C_u' of '6' or more, are well graded. During the present study various filter criteria proposed by USBR, Indian Standard, Sheralrd and USAC were applied on the proposed filter material. The results indicate that the curve of the filter materials and the core materials are more or less parallel and most of the criteria ratios were satisfied. However, the fines of the filter material are more than 5% which does not satisfy the filter criteria. In general about 55% of the criteria were satisfied by the samples.

In general the proposed shell material contains particles of gravel to cobble size. The gravels are without impurities such as clays and iron. The gravel and cobble size grains are generally basaltic in composition. Positive relationship between bulk density and point load index strength as well as bulk density and laboratory result of UCS, and an inverse relationship between porosity with point load strength and UCS was also noticed. In general the material contains about 10% Cobble, 40% Gravel, 45% Sand and 5% Fines.

The basalt rock that is selected for riprap is, well jointed with vertical and horizontal joints with joint spacing of 0.5 to 1m. The presence of joints makes the basalt easily workable and economical with regards to its excavation. The rock in general is fresh and strong. There are no intercalations of other soft rocks or soils, it is entirely fresh basalt. The crushed aggregates will not be flaky since the joints are not very closely spaced, and have no impurities such as; organic matter and clay. Based on petrographic studies on an average the lower basalt rock contain 90% ground mass, 4% plagioclase, 3% olivine, 2% opaque, and 1% sericite whereas the upper basalt contains 37-65% ground mass, 25-35% plagioclase, 19% olivine, 10% opaque, 5% sericite, 4% iddingsite.

During the present study different charts and criteria were used to analyse the construction materials for the proposed Berga dam. Based on the laboratory tests, most of the soil samples were classified as highly plastic. However, on the activity chart these samples are located in the normal and inactive zone. For example; ATP – 4 and ATP – 6 were classified as MH and MI by using USCS; respectively however, in the activity chart they are located in the inactive region. Similarly, the samples ATP – 3 and ATP – 5 were classified as highly expansive soils on the expansive chart however; they were classified as normal in the activity chart. Therefore, these charts do not seems to provide results appropriate for these soils and these empirical charts must be reviewed to account for such discrepancies. Never the less before making any such inferences on the appropriateness of these standard charts it may be required to check for number of samples rather than making any decision based on the present study in which only few samples were used.

In general the proposed construction material for the dam is slightly plastic in nature. Thus, it is anticipated that this may result in to high compressibility which may lead to instability. Also, the proposed filter materials contain more fine materials than the desired percentage. This will prevent the passage of water from the core material which will result in to the development of pore water pressure. Thus, one of the possible remedial measures to improve the properties of the filter material is to practice blending with coarse material which probably may make the material more permeable and at the same time it may prevent the migration of particles from the core materials. However, this requires further investigation where various available materials may be blended in different proportions with the filter material and relative improvement in filter material properties/ criteria can be assessed.

In the present study it was observed that the results for swelling potential and the activity are contradicting. One of the reasons could be the occurrence of iron oxide inside the clay sized material. During the laboratory tests, some of the soil samples were washed before the analysis. This might have resulted in to change in the properties of the clay sized material. Previous studies on tropical soil reported that soils which have more iron oxide, they tend to have more liquid limit than pure clay minerals like kaolin. Therefore, this could be one reason for such contradicting results. Thus, there is a need to conduct further studies so that more confirmatory results can be generated to find reasons for contradicting results on swelling potential and the activity of the soils.

The proposed Berga dam is situated in more or less very suitable area for the construction of embankment dam (by being flat area covered by impervious soil). In the present work an attempt was made to analysis the stability of the initial design of the proposed Berga dam. In order to perform that first the initial design was prepared by using Taylor's stability numbers. Later, the stability of the embankment slopes was analyzed by deterministic approach. The Factor of safety was calculated for anticipated conditions defined for various water saturations under static and dynamic conditions. For stability analysis SLOPE/W and SARC software was used.

The embankment design by Taylor's stability number provided upstream slope inclination as 20° and downstream slope as 25°. The crest width as per USBR criteria was taken as 12m and the freeboard height was computed based on the possible wave height likely to be generated in the reservoir and it comes out to be 2m. This initial dam design was prepared by considering the fact that in the previous study no such design of dam was attempted. Therefore, during the present study it was felt to prepare the initial design of the dam by utilizing the proposed construction material and to assess its stability under various anticipated conditions. Thus, it is expected that the results from the present research may be utilized when the actual dam will be designed.

Further, the stability analysis revealed that the dam design will be stable when there is no water in the reservoir and no dynamic loading takes place. However, when the reservoir starts to fill, the stability of the dam will be reducing under varied water saturation. The factor of safety will decrease under anticipated adverse conditions which may possibly result in to the failure of the slopes. Such instability may impart due to material incapability to be stable under anticipated adverse site conditions at the specified slope angles. This probably may be improved by reducing the plasticity characteristics of the materials and by improving the filter materials so that no destructive pore pressure can develop which may likely increases the chances for failures of the dam. However, to see such type of improvement in the slope stability condition it may require further investigation to either find the alternative construction material with better quality or to attempt blending of existing material with other material.

7.2 Recommendations

From the results of the present study the following recommendations are forwarded;

- The dam site is located near the active Ethiopian rift. Therefore, detailed seismic study must be performed and appropriate seismic parameters must be incorporated in the design.

- Dam abutments are covered by thin soil cover and fragmented rocks. Stripping of abutment slopes upto desired depth is required to provide a stable foundation for the dam. As such conditions may pose problems of slope stability during construction stage and if left un-attended may result into seepage problems during operational stage. Further, the underlying fractured basalt need to be grouted to avoid any possibility for seepage.

- A part of existing road from Holeta to Mugher Cement factory will be submerged under the proposed reservoir therefore; realignment of the road is required.

- In the river section alluvial soil is present at dam foundation level which is underlain by fractured basalt this was revealed by the previous geophysical investigation. The alluvial soils need to be assessed for its bearing capacity and seepage potential. Further, such alluvial material may likely possess liquefaction potential. Thus, there is a need to assess the liquefaction potential of this material. If the material is found unsuitable from bearing capacity and liquefaction point of view it may be removed.

- The seepage potential in foundation area must be assessed through water pressure tests both in the abutments and the river section particularly below the core section.

- The proposed core material is composed of silt and clay in large amounts which makes it suitable as it possesses relatively low permeability. However, through laboratory results it was found that the core material is expansive in nature which is not desirable as it may result into development to pore water pressures. Thus, blending with appropriate material will certainly improve the quality of core material. However, it requires further studies where core material can be blended with other available coarser material in varied proportions and the resulting performance may be evaluated.

- The filter material is composed of only 50% sand sized particles. This composition is not sufficient to prevent the migrating of materials from the core. It is recommended that the material must be blended with more sand and gravel material. The blending proportion can be determined by checking different proportions.

- The initial design of the proposed dam must be checked for the improved core, filter and shell material and if the design doesn't become stable for the existing and anticipated conditions, it has to be revised.

- Finally, it is recommended that the proposed dam site is suitable for the construction of the dam. The construction material for various embankment zones is available within the economic distance and the quality of material can be improved by adopting minimum measures.

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Annexures

Annex I Liquid Limit Analysis of Sample BD – 6

GEOLOGICAL SURVEY OF ETHIOPIA
Geoscience Laboratory Directorate
Liquid limit

Case Team: - Chemical: Lab Section: - Silicate ☐ Gold & Base metal ☐ Water ☐
Hydrocarbon ☐

Case Team: - Mineralogical: Lab section: - Mineralogy ☐ Physical ☒
Client /Originator Name:- Mineral Exploration and Evaluation Core Process

Client Category: - Survey ☒ Gov. ☐ Pvt. ☐
File name:- 2770/13GSE Area Ref :- No of Samples:- Sample No.
Sample Type: - Soil Lab No : -
Type of Analysis:- Liquid Limit Preparation required :- Date Submitted:- 08/06/05

Coll.No.	Lab.No.	Number of Beats	Weight of petridish with cover g	Weight of wet sample with petridish g	Weight of wet sample g	Weight of dried sample with petridish g	Weight of dried sample g	Liquid limit W.L %	Liquid limit %
BD-6	2770/13	150	31.2463	38.6963	7.45	36.3138	5.0675	47.02	
		194	25.9184	35.50202	9.58362	32.2464	6.328	51.45	
		230	31.3575	40.9887	9.6312	37.603	6.2455	54.21	
		250	31.5951	40.8426	9.2475	37.5207	5.9256	56.06	52

Described By / Analysts I. Lakech Teferi Checked by Misrak Tefera Date Completed 2/7/05

Annex II Liquid Limit Analysis of Sample BD – 4

GEOLOGICAL SURVEY OF ETHIOPIA
Geoscience Laboratory Directorate
Liquid limit

Case Team: - Chemical: Lab Section: - Silicate ☐ Gold & Base metal ☐ Water ☐
Hydrocarbon ☐
Case Team: - Mineralogical: Lab section: - Mineralogy ☐ Physical ☒
Client /Originator Name:- Mineral Exploration and Evaluation Core Process
Client Category: - Survey ☒ Gov. ☐ Pvt. ☐
File name:- 2770/13GSE Area Ref :- No of Samples: - Sample No.
Sample Type:- Soil Lab No : -
Type of Analysis:- Liquid Limit Preparation required :- Date Submitted:- 08/06/05

Coll.No.	Lab.No.	Number of Beats	Weight of petridish with cover g	Weight of wet sample with petridish g	Weight of wet sample g	Weight of dried sample with petridish g	Weight of dried sample g	Liquid limit W.L %	Liquid lim %
BD-4	2771/13	150	31.7009	39.4633	7.7624	37.2584	5.5575	39.67	
		190	26.6467	35.0614	8.4147	32.5567	5.91	42.38	
		230	40.391	49.9827	9.5917	47.0477	6.6567	44.09	
		250	33.2464	41.5834	8.337	38.9814	5.735	45.37	43

Described By / Analysts
1. Lakech Teferi

Checked by
Misrak Tefera

Date Completed 2/7/05

Annex III Plastic Limit Analyses of Samples; BD – 4 and BD - 6

Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate [ ] Gold & Base metal [ ] Water [ ]
Hydrocarbon [ ]

Case Team: - Mineralogical: Lab section: - Mineralogy [ ] Physical [x]
Client /Originator Name:- Mineral Exploration and Evaluation Core Process

Client Category: - Survey [x] Gov. [ ] Pvt. [ ]
File name:- 2770/13 GSE Area Ref:- No of Samples:- Sample No.
Sample Type :- Soil Lab No:-
Type of Analysis:- Plastic Limit Preparation required:- Date Submitted:- 08/06/05

Coll.No.	Lab.No.	Weight of petridish with cover (g)	Weight of wet sample with petridish g	Weight of wet sample g	Weight of dried sample with petridish g	Weight of dried sample g	Plastic limit W.P %	Average
BD-4	2771/13	18.2556	21.536	3.2804	20.7362	2.4806	32.24	**32.25**
		17.225	20.7133	3.4883	19.8625	2.6375	32.25	
BD-6	2770/13	17.5035	20.6087	3.1052	19.7585	2.255	37.70	**37.70**
		17.6432	21.0838	3.4406	20.1419	2.4987	37.70	

Described By / Analysts
1. Lakech Teferi

Checked by
Misrak Tefera

Date Completed 27/6/05

Annex IV Sieve Analysis of Sample BD – 11C

Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate ☐ **Gold & Base metal** ☐ **Water** ☐
Hydrocarbon ☐

Case Team: - Mineralogical Lab section: - Mineralogy ☐ **Physical** x
Client /Originator Name:- Mineral Exploration and Evaluation core procces

Client Category: - Survey x **Gov.** ☐ **Pvt.** ☐
File name:- 2767/13gse Area Ref-: No of Samples: - Sample No. BD-11C
Sample Type :- Soil Lab No:- 2769/13
Type of Analysis:- Sieve Analysis **Date Submitted:- 08/06/05**

Sieve Opening mm	Sample weight retained gm	Weight % Retained	Cumulative weight percent oversize	Cumulative weight percent undersize
>2	14.6308	7.3154	7.3154	100
2-1.18	6.6261	3.31305	10.62845	92.6846
1.18-0.6	15.2657	7.63285	18.2613	89.37155
0.6-0.3	20.243	10.1215	28.3828	81.7387
0.3-0.16	17.3479	8.67395	37.05675	71.6172
0.16-0.063	23.5081	11.75405	48.8108	62.94325
<0.063	102.3784	51.1892	100	51.1892

Described By / Analysts 1. Lakech Teferi **Checked by** Misrak Tefera **Date Completed 20/6/05**

Annex V Sieve Analysis of Sample BD – 11A

Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate ☐ **Gold & Base metal** ☐ **Water** ☐
Hydrocarbon ☐

Case Team: - Mineralogical Lab section: - Mineralogy ☐ **Physical** ☒
Client /Originator Name:- Mineral Exploration

Client Category: - Survey ☒ **Gov.** ☐ **Pvt.** ☐
File name:- 2767/13GSE Area Ref-: No of Samples: - Sample No. BD-11A
Sample Type :- Soil Lab No:- 2768/13
Type of Analysis:- Sieve Analysis Date Submitted:- 08/06/05

Sieve Opening mm	Sample weight retained gm	Weight % Retained	Cumulative weight percent oversize	Cumulative weight percent undersize
>2	21.8612	10.9306	10.9306	100
2-1.18	6.101	3.0505	13.9811	89.0694
1.18-0.6	15.034	7.517	21.4981	86.0189
0.6-0.3	18.5734	9.2867	30.7848	78.5019
0.3-0.16	13.7242	6.8621	37.6469	69.2152
0.16-0.063	17.6381	8.81905	46.46595	62.3531
<0.063	107.0681	53.53405	100	53.53405

Described By / Analysts 1.Lakech Teferi **Checked by** Misrak Tefera **Date Completed 20/6/05**

Annex VI Sieve Analysis of Sample BD – 5

Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate ☐ Gold & Base metal ☐ Water ☐
Hydrocarbon ☐

Case Team: - Mineralogical Lab section: - Mineralogy ☐ Physical ☒
Client /Originator Name:- Mineral Exploration and Evaluation Core Process
Client Category: - Survey ☒ Gov. ☐ Pvt. ☐
File name:- 2767 /13GSE Area Ref: No of Samples:- Sample No. BD-5
Sample Type :- Soil Lab No:- 2767/13
Type of Analysis:- Sieve Analysis Date Submitted:- 08/06/05

Sieve Opening mm	Sample weight retained gm	Weight % Retained	Cumulative weight percent oversize	Cumulative weight percent undersize
>2	0	0	0	0
2-1.18	0.0359	0.01795	0.01795	100
1.18-0.6	0.3465	0.17325	0.1912	99.98205
0.6-0.3	1.7733	0.88665	1.07785	99.8088
0.3-0.16	2.2160	1.108	2.18585	98.92215
0.16-0.063	2.4287	1.21435	3.4002	97.81415
<0.063	193.1996	96.5998	100	96.5998

Described By / Analysts: 1 Lakech Teferi
Checked by: Misrak Tefera
Date Completed 20/6/05

Annex VII Pipette Analysis of Samples; BD – 11A, BD – 11C and BD - 5

Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate ☐ **Gold & Base metal** ☐ **Water** ☐
Hydrocarbon ☐

Case Team: - Mineralogical: Lab section:- Mineralogy ☐ **Physical** ☒
Client /Originator Name: - Mineral Exploration and Evaluation Core Process

Client Category:- Survey ☒ **Gov.** ☐ **Pvt.** ☐
File name:- 2767/13GSE Area Ref :- No of Samples:- 4 Sample No.
Sample Type :- Soil Lab No:-
Type of Analysis:-Pipette analysis Preparation required: - Date Submitted:- 08/06/05

Sample No& Lab.No		Data of pipette Analysis							
Lab. No.	Coll.No		<0.00063 mm. 1. Weight in g. 2.Mass %	0.00063-0.002 mm 1. Weight in g. 2.Mass %	0.002-0.0063 mm 1. Weight in g. 2.Mass %	0.0063-0.016 mm 1. Weight in g. 2.Mass %	0.016-0.04 mm 1. Weight in g. 2.Mass %	0.04-0.063 mm 1. Weight in g. 2.Mass %	>0.063 mm 1. Weight in g. 2.Mass %
BD-11C	2769/13	1	12.08	10.34	8.29	14.74	25.49	31.43	97.62
		2	6.04	5.17	4.14	7.37	12.75	15.72	48.81
BD-11A	2768/13	1	13.38	10.17	9.31	13.81	23.77	36.62	92.93
		2	6.69	5.08	4.66	6.91	11.88	18.31	46.47
BD-5	2767/13	1	51.39	24.92	27.43	40.77	48.30	0.39	6.80
		2	25.70	12.46	13.72	20.38	24.15	0.19	3.40

Described By / Analysts J.Lakech Teferi **Checked by** Misrak Tefera **Date Completed 5/7/05**

Annex VIII Moisture Content Analysis of Samples; BD – 7, BD – 8, BD – 9 and BD - 10

Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate [ ] Gold & Base metal [ ] Water [ ]
Hydrocarbon [ ]
Case Team: - Mineralogical: Lab section: - Mineralogy [ ] Physical [x]
Client /Originator Name: - Mineral Exploration and Evaluation Core Process
Client Category: - Survey [x] Gov. [ ] Pvt. [ ]
File name: - 2772/13GSE Area Ref:- No of Samples :- 4 Sample No :-
Sample Type:- Soil Lab No:-
Type of Analysis:- Moisture content Preparation required: - Date Submitted:- 08/06/05

Coll. No.	Lab. No.	Weight of petridish with cover g	Weight of wet sample with petridish g	Weight of wet sample g	Weight of dried ample with petridish g	Weight of dried sample g	Water content Wn mass %	Average
BD-7	2772/13	58.0418	93.7644	35.7226	89.1705	31.1287	14.76	14.72
		45.932	78.4747	32.5427	74.3108	28.3788	14.67	
BD-8	2773/13	47.0254	81.4956	34.4702	75.5896	28.5642	20.68	20.59
		46.7103	83.9346	37.2243	77.6013	30.891	20.50	
BD-9	2774/13	52.4028	90.8621	38.4593	88.1276	35.7248	7.65	7.66
		52.3035	90.3781	38.0746	87.6631	35.3596	7.68	
BD-10	2775/13	66.2707	104.3763	38.1056	101.0763	34.8056	9.48	9.47
		56.8894	92.602	35.7126	89.5185	32.6291	9.45	

Described By / Analysts Lakech Teferi   Checked by Misrak Tefera   Date Completed 18/6/05

Annex IX Dry Density Analysis of Samples; BD – 7, BD – 8 and BD - 9

Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate ☐ Gold & Base metal ☐ Water ☐
Hydrocarbon ☐

Case Team: - Mineralogical: Lab section: - Mineralogy ☐ Physical ☒
Client /Originator Name:- Mineral Exploration and Evaluation Core Process

Client Category: - Survey ☒ Gov. ☐ Pvt. ☐
File name:- 2772/13GOV Area Ref:- No of Samples: 4 Sample No.
Sample Type:- Soil Lab No: -
Type of Analysis :- Dry Density Preparation required: - Date Submitted:- 08/06/05

Coll.No.	Lab.No.	Natural Sample Weight gm	Weight Covered with paraffin at air gm	Weight covered with paraffin under water gm	Bulk- Density g/cm^3	Average
BD-7	2772/13	65.2	85.52	17.02	1.56	1.56
		85.19	111.11	22.18	1.56	
BD-8	2773/13	68.69	90.4	19.2	1.61	1.62
		97.8	116.61	31.33	1.62	
BD-9	2774/13	74.06	88.27	30.94	1.92	1.91
		96.16	121.56	37.14	1.89	

Described By / Analysts
1.Lakech Teferi

Checked by :- Misrak Tefera

Date Completed :- 29/6/05

Annex X XRD Analysis of Sample BD – 12

Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate ☐ **Gold & Base metal** ☐ **Water** ☐

Hydrocarbon ☐

Case Team: - Mineralogical: Lab section: - Mineralogy ☑ **Physical** ☐

Client /Originator Name: Tutan Negash (Geological Survey of Ethiopa)

Client Category:- Survey ☑ **Gov.** ☐ **Pvt.** ☐

File name:- 1447/13GSE **Area Ref-:-** - **No of Samples :-** 1 **Sample No.** BD-12

Sample Type:- Soil **Lab No** 1447/13

Type of Analysis:- XRD **Preparation required:-** powder <160mic. **Date Submitted:-** 19/02/2013

I) Identified Minerals

Mineral	(%)
Quartz low	73.1
Kaolinite	9.8
Feldspar potassian	17.1

II) Remarks:-

Described By / Analysts Workelul G/K

Checked by Workelul G/Kirstos Mineralogy & Physical Analysis Case Team Co-ordinator

Date Completed 11/03/2013

Page 1 of 1

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