Addis Ababa University

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Dam Heightening

(Case study: Gefersa I/II dam)

By

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Addis Ababa, Ethiopia

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DAM HEIGHTENING
(CASE STUDY GEFERSA-I/II DAM)

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A Thesis Submitted to the School of Graduate Study of Addis Ababa University in Partial Fulfillment of the Degree of Master of Science (M.Sc.) in Civil and Environmental Engineering

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School of Civil and Environmental Engineering

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This is to certify that the thesis prepared by Melese Ginbaru, entitled: **Dam Heightening (Case Study Gefersa-I/II Dam)** submitted in partial fulfillment of the degree of Masters of Science (Civil and Environmental Engineering (Major Hydraulic Engineering)) complies with the regulation of the university and meets the accepted standards with respect to originality and quality.

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ABSTRACT

Sediment problem is a major problem which highly reduces the live storage capacity of the reservoir. Gefersa-I/II dam is one of the major embankment dams in our country Ethiopia which trap huge amount of sediments in each year. Due to this reason AAWSA decide to heighten the existing dam up to 5m effective height with 2m additional freeboard. This case study mainly focuses on determining the suitable and stable heightening type which is compatible with the existing dam condition. From different point of view this study preferred to heighten the existing embankment as homogenous clay fill embankment. Due to the upstream condition of the existing dam, most of the fill is done at the downstream side of the embankment. this create good environment to start the fill on the crest of existing dam by letting 2m width for working space. While 2m working space is provided at the upstream side of the dam automatically berm is created with 2m width. Since soil material is sensitive for deformation, 7m berm is provided at the downstream side of the dam to prevent such problem. The geometry of the heightening part was fixed as 1V: 3H at the upstream face and 1V:1.45H and 1:4.17 at the downstream side (from top to bottom) after a number of trials. Analysis was conducted using Geostudio 2007 software as a major tool. Steady state seepage analysis was conducted and the seep water to the body of the dam is around $1.952*10^{-7}m^3$/sec/m. During this study the stability of the dam was analyzed and as the result indicates, the dam side slope is safe for all loading condition. Deformation and earth quake analysis also conducted and, as the result shows the vertical and horizontal settlement was found in allowable range.

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List of abbreviations

AAIT	Addis Ababa institute of technology
AAWSA	Addis Ababa water and sewerage authority
asl	above sea level
ACERD	Asphalt Core Earth-Rock fills Dam
BC	Before Christ
CFRD	Concrete Faced Rock Filled Dam
m^3	Cubic meter
^oc	degree centi grade
D/S	Downstream
FoS	Factor of Safety
FEM	Finite Element Method
FSL	Full Supply Level
GSI	Geological strength index
Gpa	Giga Pascal
g	gravity
ha	hectare
H	Height of Dam
ICOLD	International commission of large dams
Km	kilo meter
Kpa	kilo Pascal

Mpa	Mega Pascal
No.	Number
PI	Plastic Index
d	thickness of filter
USACE	United States Army Crop of Engineers
USBR	United states Bureau of Reclamation
USSD	United States Society on Dams
U/S	Upstream

CHAPTER ONE

1.0 INTRODUCTION

1.1 General

Nowadays, embankment dams exist in excess of 300 meters high with volumes of many millions of cubic meters of fill. Development of soil mechanics, study of behavior of earth dams, and the development of better construction techniques have all been helpful in creating confidence to build higher dams with improved designs and more details that are ingenious. The result is that the highest dam in the world today is an earth dam. In spite of these developments it is difficult to establish mathematical solutions to the problems of design, and many of its components are still guided by experience or judgment. For a realistic design of an earth dam, it is necessary that the foundation conditions and materials of construction be thoroughly investigated.

Addis Ababa was established as the capital city of Ethiopia in 1886 and has grown to become the largest urban and commercial center in the country. During its early years, the principal sources of water were the numerous springs located at the foot of the Entoto mountain range and hand dug wells located in the lower areas. The larger springs were tapped and fed into a number of small tanks for local distribution (inception report).

Continued growth necessitated the construction, in 1938, of a plant at the foot of Entoto to treat water from a number of springs and the nearby Kechene River, and in 1944 the original Gefersa dam located North West of the city was completed.

The Gefersa Dam was raised and a treatment plant built in 1960, while many of the springs were taken Out of service because their quality was deteriorating. In 1966, the raw water storage capacity in the Gefersa watershed was increased with the construction of another small dam north of the existing dam. This dam was also assumed to assist as a sediment trap. The reservoir formed behind a main dam built in 1938 (and modified in 1954) and a second, smaller dam, built in 1966, upstream from the main dam; the water-storage Capacities are 6, 500,000 m^3 and 1,500,000 m^3 respectively, and the two dams control a catchment area of 5,700 ha.

Gefersa Dam is located on the Addis Ababa - Ambo Asphalt road approximately 18 km north-west of Addis Ababa city center. Specifically, it is found in Oromiya National Regional State Finfine Zuria Burayu City Administration. Gefersa-I/II dam is situated to the south of the main road, geographically at $8^o38'30''$ East and $9^o4'15''$ North. Gefersa-III is found upstream of Gefersa-I/II dam north of the main Addis Ababa – Ambo asphalt road. It is geographically located at $38^o38'00''$ East and $9^o4'45''$ North.

The project area is part of the central highlands of Ethiopia with elevation ranging from 2000 to 2800masl. The topography of the project site and its surrounding area consists of hills, valleys and rivers and many small streams. The temperature is fairly constant throughout the year between 20^oC and 25^oC during day time and between 7^oC and 11^oC during the night. The average rainfall is 1200mm per year with the major rainy season from July to September.

As we know everything in the world has its own virus e.g. for human being HIV virus, for computer virus of computer, etc. just the like sediment deposition is the main virus of reservoir. Sediment deposition is one of the major problems which occur in the reservoir through time. This problem highly reduces the capacity and design period of the reservoir. The cause of such series problem is may be early design fault or/and constructional fault (Garge, 2009).

Gefersa reservoir is one of the reservoir which highly affected by silt deposition problem. Since the silt deposit fill the whole capacity of the reservoir, Gefersa-III dam is automatically stop its function. Now the deposition is continued to Gefersa-I/II dam it highly reduce the live storage capacity of the reservoir. Due to this reason the reservoir cannot fulfill the demand water for the targeted purpose (inception document).

After a long process of selecting the best alternative to increase the capacity of the reservoir, dam height extension become the best one and 5m height extension from the existed dam height is decided. Now this research will stand to design the existed dam with 5m height extension.

1.2 Back Ground

The selection of the type of dam embankment to be used at particular site is affected by many factors. The dam engineer's task is to consider these factors and adopt a suitable design. The overriding consideration is most case will be to construct an adequately safe structure for the lowest total cost. Foundation condition, availability of construction material, cost (economy) of construction are key factors for dam type selection for specific site. The current condition of the reservoir and availability of suitable construction material will be the base for this thesis.

1.3 Statement of the Problem

Any engineer wishes a long period of life for his/her dam. However, dam faces a series of problem which highly reduce the capacity and design period of the reservoir(Garge, 2009). Among the problem silt deposition is the major one. Silt deposition is becoming a series problem because it reduces the capacity of the reservoir which in turn reduces the design period of the reservoir. Gefersa reservoir is one of the reservoir which highly affected by silt deposition problem. The basic objective of constructing this dam was to fulfill the water demand of Addis Ababa peoples. But now, as the study indicated the dam (i.e. Gefersa-III dam) is automatically stop its operation due to silt problem and the deposition of sediment is continued to Gefersa I/II which highly reduce the live storage capacity of the reservoir. This problem highly affects the peoples of Addis Ababa who depend on the water reserved in the dam to satisfy their water requirement. At preliminary study stage, two alternatives was stated to reduce/or eliminate the effect of sediment deposition in Gefersa-I/II dam. The first one is excavating the deposited sediment from the reservoir, and another one is extending the existing dam to the recommended height. When the former alternative is analyzed, it becomes too costly and failed to be the best, hence the final decision become extending the existed dam to the recommended height to increase the live storage capacity of the reservoir.

1. 4 Objectives of the Study

1.4.1 General Objective

- The main objective of this study was to extend (to heighten) Gefersa-I/II dam up to 5m effective height with 2m freeboard to increase the live storage capacity of the reservoir.

1.4.2 Specific Objectives

- To decide and select favorable and feasible Dam heightening type.
- To determine compatible Dam geometry and material with the previous dam and its foundation economically and technically.
- To evaluate and check the stability of the dam heightened under static and dynamic condition.

1.5 Description of the study area

1.5.1 Location of the study area

Addis Ababa is the capital city of Ethiopia and the place where African unity (AU), African Economic Commission and other international offices are situated. It is located in the central highlands of Ethiopia with in $9^o02'$ Latitude and 38^o 42'Longitude with an elevation ranging from 2000 to 2800 meters above mean sea level. The topography of the city consists of hills, valleys and rivers and many small streams. The temperature is fairly constant throughout the year between 20^oc and 25^oc during day time and between 7^oc and 11^oc during night. Average rainfall is 1200mm per year with the major rainy season from July to September.

The Gefersa Dam is located on the Addis - Ambo road approximately 18 km to north-west side of Addis Ababa city center. Specifically, it is found in Oromiya National Regional State Finfine Zuria Burayu City Administration. Gefersa-I/II dam is situated to the left of the main road, geographically found at $38^0 38'30''$ East Longitude and $9^0 4'15''$ North Latitude Gefersa-III is found upstream of Gefersa-I/II dam to the right of the main road Addis – Ambo. It is geographically located at $38^0 38'00''$ East Longitude and $9^0 4'45''$ North Latitude.

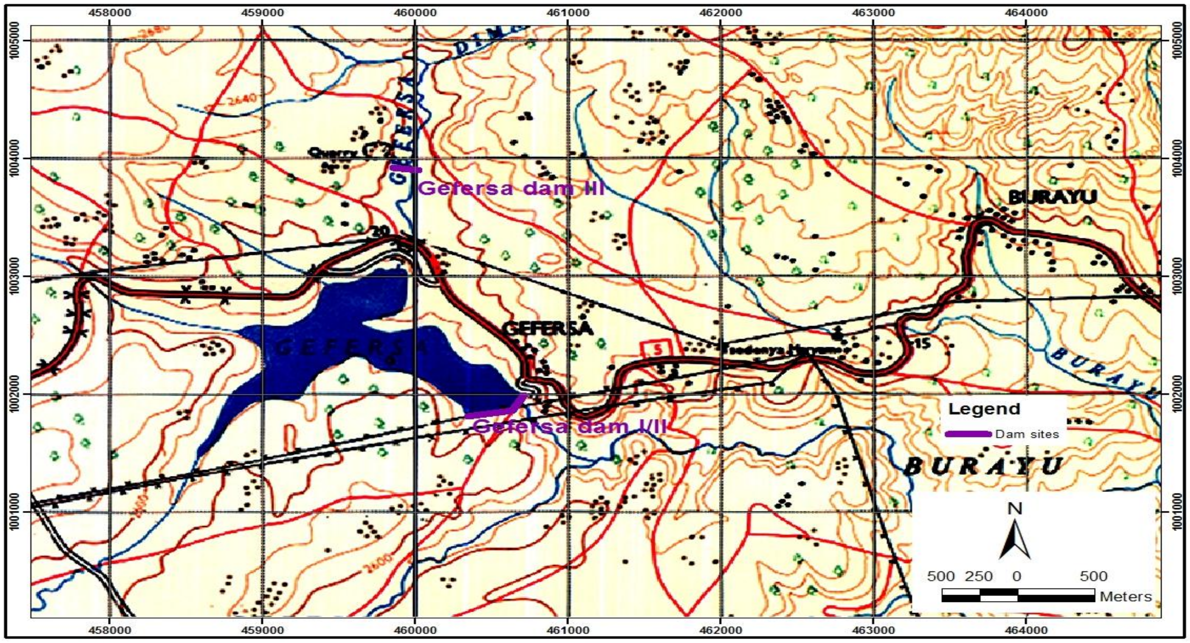

Figure 1:- profile of Gefersa-I/II and Gefersa-III dam

1. 6 Geology of the study area/dam site

1. 6.1 Regional Geology

The general geology of the area around the project site varies slightly from place to place. The project site is dominated by Foota basalt which is strongly weathered and jointed basalt, Wechecha-Yerer-Fuji ignimbrite and Quaternary olivine phiric basalt. It is fine grained, moderately jointed and dark grey to black in color. The upper part of the rock is highly weathered and altered to soil mass. Ignimbrite rock composed the geology of the area east and west of the project site. The ignimbrite rock is light grey in color, fine to coarse grained and columnar jointed at the top. To the south west around Salo area there is a dome shaped hill entirely composed of scoria rock. Further away towards east there is Tertiary basalt rock. The rock units around the relatively flat lying areas are overlain by residual soil deposit.

1. 6.2 Local Geology

The local geology of the project area has variation from place to place. At Gefersa I/II intake structure site the local geology is composed of site/rhyolite overlain by shallow residual/ back filled soil cover. The rock is light grey to creamy in color, moderately to highly weathered and intensively fractured. The rock is about 20 m thick and is underlain by pyroclastic material.

At Gefersa III dam site the geology is composed of thick residual soil deposit formed from prolonged time of weathering and decomposition of the underlying parent rock. The thickness of the soil reaches up to 24 m on the left abutment and the top about 8 m thick soil has organic content. Underlying the top soil horizon with high organic content is the residual soil mass characterized by variegated colors of brownish, red, whitish, and yellowish. The grading of the soil generally dominated with cohesive soil mass with sand (silty clay, clayey silt and sandy silt.

The reservoir area of Gefersa III dam generally is dominated by rolling and slightly sloping to flat land topography. The area is intersected by River and its tributaries. Rocks are observed on exposures at river and stream cuts and rarely on slightly sloping areas near rivers. Almost the entire reservoir area is covered by about 1-3 m thick residual and alluvial soil of sandy silty clay and sandy clayey silt composition.

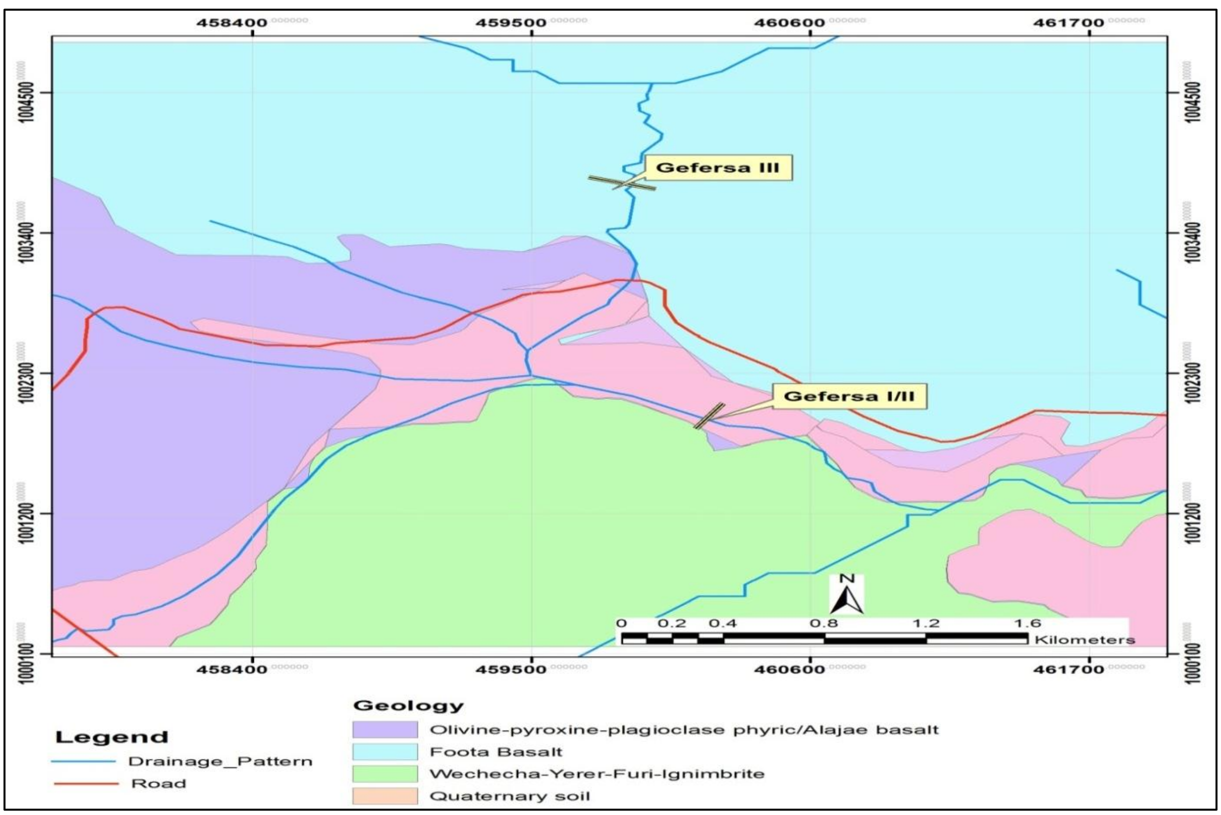

Figure 2: geological map of study area (from Assegid, 2007 1.7 Climate

The project area is part of the central highlands of Ethiopia with elevation ranging from 2000 to 2800 masl. The topography of the project site and its surrounding area consists of hills, valleys and rivers and many small streams. The temperature is fairly constant throughout the year between 20^{o}C and 25^{o}C during day time and between 7^{o}C and 11^{o}C during the night. The average rainfall is 1200mm per year with the major rainy season from July to September.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 General

Soil erosion causes worldwide environmental problems leading to degraded soil productivity and water quality, causes sedimentation in the reservoirs and increases the probability of flood as a result of reduction of flood storage capacity. A river is not only conveying water, but has many other functions. One of these functions is transporting of erosion products (boulders, gravel, sand, silt and clay) from its catchment to downstream direction. If the transport capacity of a river is affected by diversion of water from the river or by storing water in a reservoir, deposition of sediment may occur. If not properly taken care of, harmful sedimentation and erosion may occur. Many reservoirs are suffering from excessive sedimentation often due to the fact that either the upstream sediment supply was never considered or that the seriousness of this process was underestimated, because of the lack of sufficient data. Also a change in sediment yield is due to change in land use in the upstream catchment that can cause detrimental sedimentation.

2.2 Dam Heightening- the UK Perspective

This paper aims to illustrate the benefits of dam heightening as a means of increasing reservoir storage. Examples of several UK dams that have been designed to accommodate future heightening are detailed along with UK dam heightening projects that have been successfully carried out.

2.2.1 Advantage of Dam Heightening

When dam is heightened, the volume of extra storage is considerable compare to the increase in retention level. The cost of heightening work is significantly less than the cost of new dam to store the same volume. Another considerable benefit is that the reservoir infrastructure such as inlet works, spillway channel, access road, outlet work is already in place.

2.2.2 Case Studies in UK

Numerous UK dams have been designed to accommodate future heightening; several of these have been heightened. The following are UK dams involved in heightening.

2.2.2.1 Mullardoch Dam

Mullardoch dam is a concrete gravity dam located in the north of Scotland and is part of hydro-electric scheme. The dam was originally designed for a height of approximately 42m but a review of capital expenditure during construction resulted in it being stopped when its height was 35m. It was subsequently decided to complete the construction of dam to its full height (Roberts 1958).

The heightening works comprised a concrete slab on the downstream face of the dam which was held separate from the original dam by a series of ribs in the vertical plane so that 900mm wide slots were formed between the slab and the original dam.

2.2.2.2 Wayoh Dam

Wayoh reservoir was formed in 1876 to regulate compensation water to the Bradshaw Brook. The reservoir is impounded by a 19m high earth fill dam with puddle clay core. In 1959 construction commenced on heightening works to the dam so that it could be used for water supply (Jones 1962).

The works involved heightening the dam by 7.6m and alterations to the spillway and outlet works. The dam was heightened by steepening the upstream and downstream shoulders with two part way up the heightened dam. To provide a good key between the existing and new fill material the upper layers of material were removed and series of benches were formed in the existing dam.

The reservoir was lowered during the works which permitted the heightening works carried out to the upstream and downstream shoulders of the dam. The reservoir was also drawn down to allow modifications to spillway and outlet works to be carried out safely.

2.2.2.3 Ladybower Dam

Ladybower reservoir is impounded by a 43m high embankment dam constructed between 1936 and 1945. Remedial works have been carried out to the dam on five separate occasions since construction. On four occasions the works were carried out to address settlement, the other was the result of a review of the design flood routing.

The most recent works were carried out between 1998 and 2000 and comprised heightening the dam by 2.95m at a cost of 4.9million euro, (Jamieson 1999). The works involved placing about

200000m3 of rock fill to raise the crest of the dam and onto the downstream shoulder, construction of a new wave on the upstream side of the crest.

The impermeable zone of the dam was extended to the heightened crest level by means of a HDPE membrane that was keyed into the existing puddle clay core by a bentonite/cement slurry trench excavated into the top of the core

2.3 Bockhartsee Dam Heightening In Austria

The approximately 50m high Bockhartsee dam in Austria is located in a mountainous region and on a high elevation. Due to the current situation on the liberalized energy market and with an increasing demand for peak and regular hydro power energy as well as the necessity for a higher operation flexibility of existing pumped storage plants, Salzburg AG decided, among other measures, to enlarge the existing Naßfeld hydropower plant in the province of Salzburg, Austria.

2.3.1 The Bockhartsee Dam before Heightening

The existing Bockhartsee Dam is a rock fill type with a 60 cm thick central concrete membrane (diaphragm wall) as water barrier. In the initial construction phase between 1980 and 1982, the concrete membrane was built simultaneously with the dam fill in 7.5 m wide sections and in 4.3 m high steps. The vertical joints of the membrane were designed as settlement joints and sealed with a surface water stop tape as well as an internal rubber water stop. The horizontal construction joints were sealed with surface water stop tapes on the upstream side.

In the upstream dam shoulder only quarried rock (gneiss) was used as fill material while the downstream dam shoulder was zoned with moraine, mixed grain and quarried rock material. The rock and moraine materials were placed in about 1 m thick layers, the downstream drainage layer (mixed gravel) as well as the upstream transition zone was placed in 50 cm thick layers and the upstream fine grained zone in 30 cm thick layers. All dam shoulders (rock and moraine materials) were compacted by 15 tons vibratoring rollers. On the upstream diaphragm surface a thin, bitumen based sliding foil was applied. In finite element stress and strain analyses done after the first impounding of the dam and based on the initial short term monitoring results the following material properties for the dam shoulders were used:

The long term behavior of the Bockhartsee Dam before heightening over more than 20 years in terms of very low seepage and very small deformations was excellent and did not cause any problems during operation.

2.3.2 The Bockhartsee Dam Heightening

Based on a proposal of Prof. Dr. W. Schober, the Bockhartsee Dam was heightened by placing an angular retaining wall on top of the existing dam crest downstream of the existing concrete membrane. The first design approach was to raise the existing concrete diaphragm wall up to the new dam crest. Later on, the heightening was designed as an angular retaining wall independently from the existing diaphragm wall and connected with a special joint and sealer structure. This way it was possible to preserve the upstream slope and to operate the reservoir during the construction period. The total fill volume of the downstream dam shoulder was approximately 85,000 m³ and only solid rock from the cavern excavation was used. The construction at the dam and cavern excavation started in early May 2006 and the project was completed in September 2007.

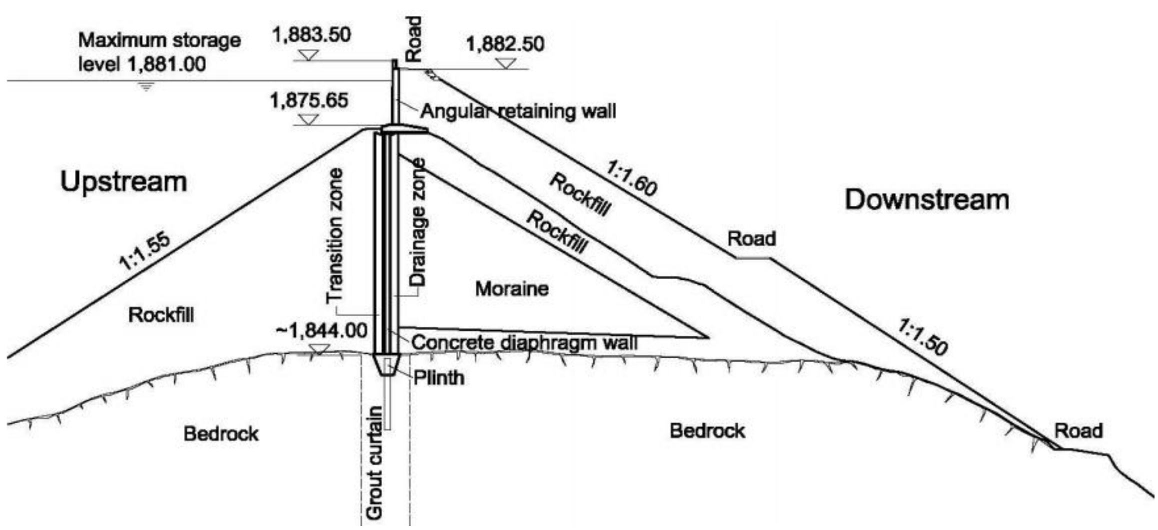

Figure 3: cross section of Bockhartsee dam after heightening

2.4 Review on annual average sediment deposition in Gefersa reservoir

Gefersa reservoir is one of the surface water sources providing clean drinking water to residents of the capital city Addis Ababa in addition to Legedadi and Dire reservoirs. The Gefersa catchment area (55.56 km2, or 5,556 ha) is located in Oromiya region under the control of AAWSA. Gefersa III reservoir, which is constructed in 1966, is located upstream of Gefersa I/II dam about 1km to the west. It is used as both storage and silt trap. The Gefersa River and its feeder streams are part of the Akaki river catchment. The reservoirs supply an average of 30,000m^3 of treated water per day to Addis Ababa city. The high rate of siltation is a major long-term problem for the reservoirs, as it severely affects their capacity and results in shortage of usable water for Addis Ababa. It also increases the water treatment costs .Based on the 1979 and 1998 bathymetric surveys, with the assumption of linear yearly siltation rate, there is 22,252m^3/year sediment outflows from the basin. In terms of soil loss from the catchment area, this constitutes a loss of 575 tons/km^2/year.

This study concentrates on of prediction sediment inflow to Gefersa reservoir using SWAT watershed model and assessing sediment reduction methods in the Gefersa catchment, and to simulate future trend under different land use scenarios based on the study of the master plan and field visits.

2.5 Review on dam type selection

The selection of type of embankment for any site is governed by many factors. Any engineer must encounter those factors while designing embankment for specific selected site. The overriding consideration is most case will be to construct an adequately safe structure for the lowest total cost. Usually the most economic design will be that which uses a construction materials source close to the dam, without excessive modification from the “borrow pit run’’ or “quarry run’’ material. Availability of construction materials, foundation condition, and topography of the site, climate condition, and spillway requirement are the major factors which govern the selection of embankment type for a certain site (geotechnical engineering of dams).

2.5.1 Availability of construction materials

Clearly the availability of suitable construction materials within economic haul distance is critical in the selection of the embankment type. The uniformity of the available earth fill will also influence design and the method of construction. If the borrow areas produce two different types of earth fill, the earth fill may be zoned into two parts (geotechnical engineering of dams).

A material whose composition is satisfactory for use in embankment construction is a suitable material. The moisture content of the material has no bearing upon such designation. In general, any mineral (inorganic) soil, blasted or broken rock and similar materials of natural or manufactured (i.e. recycled) origin, including mixtures thereof, are considered suitable materials. Consideration should be given to whether the material contains vegetable or organic matter, such as muck, peat, organic silt, topsoil or sod. Certain manufactured deposits of industrial waste, sludge or landfill may be determined to be unsuitable.

Inorganic, fine-grained soils are classified as either clays or silts. Clays and silts typically exhibit different engineering behavior with regard to compressibility, strength and permeability. The principal characteristics that distinguish fine grained soils from coarse grained soils for purposes of embankment dam design are that fine grained soils have lower permeability, lower shear strength, and higher compressibility. Soil plasticity serves as an initial indicator of the potential behavior of a clay or silt when placed in an embankment dam.

The most common index tests, the Atterberg limits, help to classify and characterize fine-grained soils according to the degree to which moisture changes impact soil consistency and behavior. Atterberg limits are water contents at defined transitions in soil consistency, as measured by standardized tests. The liquid limit and the plastic limit are the most commonly used Atterberg limits in engineering work. The liquid limit is the water content at the transition between a liquid and plastic solid state, and the plastic limit is the water content that defines the lower limit of the plastic solid state. The plasticity index (PI = liquid limit - plastic limit) indicates the magnitude of water content range over which the soil remains plastic.

CHAPTER THREE

3. EXISTING CONDITION OF GEFERSA DAM

2.2 HYDROLOGY

3.1.1 General

This report presents the summery of different hydrological analysis conducted on of Gefersa River and its catchment for the design of Gefersa I/II dam intake rehabilitation and Gefersa III dam heightening works. The study was conducted in relation to different scenarios of Geferesa dam options. Availability of surface water resource, estimate of water for water supply and estimating maximum flood design are dealt with in this section. Gefersa River is the main tributary to Tinishu Akaki River, which is one of the main river in Akakai river system. The river up to the Gefersa III dam site has very small catchment area of about 35.5 Km2. The main Gefersa River originates from Intoto Mountains, north and east of Addis Ababa and Wechecha Mountain west of Addis Ababa. Gefersa River is formed from two rivers namely Menjaro (flowing from west side) and Dima River (from east side). Downstream of Gefersa I/II dam a gauging station installed known by Tinishu Akaki. The rainfall falling in rainy seasons comprises more than 90% of the annual total rainfall; out of this the major rainy season comprises about 75% of the total annual. The annual rainfall is in the range of 1260 mm. Maximum monthly recorded in August (330 mm). The amount of evaporation from open water surface is obtained using empirical equation and based upon the water surface temperature and the value of certain meteorological elements. Accordingly, the open water evaporation varies from 50 mm (August) to 170 mm (March).

Mean monthly flow of Tinishu Akaki River at the gauging station varies from minimum value of 0.147 MMC to maximum of 38.882 MMC. The annual available water at the Gefersa III dam site and its 90% dependability are 16.07 and 10.31 Mm3 respectively. The total sediment load including bed load, which will deposit in the Gefersa III reservoir throughout 50 years of dam life span, is equal to 0.393MCM. The reservoir sediment distribution analysis shows that the New Zero Elevation of reservoir is found to be located at a level of 2599.23m.a.s.l. However the existing intake level at Dam III is at 2597 m.a.s.l

therefore, for the simulation run 2597 m.a.s.l level have been considered as the minimum drawdown level. According to the Gefersa Water Supply Project operation schedule the Gefersa dam I/II reservoir will be operated throughout the year and water level will reach at its lowest starting from May to July i.e. on these three months the of Gefersa Dam I/II bottom out let, which is used for environmental release and sediment flash will be closed. And at the end of June the Gefersa Dam I/II reservoir will almost reach at it minimum draw down level (MDDL) I.e. at 2575.75m.a.s.l. And the Gefersa Dam III will supply the downstream Gefersa Dam I/II reservoir for month of July. Whereas, if the above condition altered (i.e. heightening of Gefersa III dam to FRL of 2613m.a.s.l) the Gefersa I/II dam bottom outlet gate closed the reservoir will attain reliability greater than 95%. Otherwise, the release from Gefersa I/II will continue for about seven months to attain the reliability level of 92%.
As per HEC-HMS rainfall-runoff modeling the inflow hydrographs of 1,000, 10,000 years return period and PMF at Gefersa III dam have a value of 152.8, 272.7 and 319.2 m3/s respectively. The routed outflows from the reservoir accordingly are 6.6, 89.4 and 138.3 m^3/s respectively.

3.2 Catchment Characteristics

Gefersa River is the main tributary to Tinishu Akaki River, which is one of the main rivers in Akakai river system. The river up to the Gefersa III dam site has very small catchment area of about 35.5 Km^2. The main Gefersa River originates from Intoto Mountains, north and east of Addis Ababa and Wechecha Mountain west of Addis Ababa. Gefersa River is formed from two rivers namely Menjaro (flowing from west side) and Dima River (from east side). Downstream of Gefersa I/II dam a gauging station is available known by Tinishu Akaki below Gefersa dam station. The length of Gefersa River up to Gefersa III dam and between Gefersa III and Gefersa I /II dam is 9.4 Km and 2.9 Km respectively. Rivers like Gefersa, Welenso, Burayu, Siren, Kersa and Fincha are major tributaries to Tinshu Akaki. Most of the study area from North to the South and West can be designated as elevated plateau with slope ranging from 0 to 20 %, whereas the protruding of Intoto mountain chain to the right side of the watershed is visible. The highlands of the area have rugged topography with slope of ranging from 20 to 70 %.

3.3 Hydro-Meteorological Data Availability

There are four (4) meteorological stations and a one (1) hydrological station is found to be relevant for the hydrological study of the project area. Except the Gefersa dam meteorological station the other are located far from the catchment area. According to the WMO specification the density of meteorological stations for the project boundary is satisfied due to the presence of Gefersa dam meteorological station. On Akaki River system there exist two hydrological stations viz. at Tilku Akaki and Tinishu Akaki rivers. The gauging station on Tinishu Akaki river is located downstream of Gefersa I/II dam. As per WMO recommendation the stream flow gauging station density of the mentioned river basin is within the limit i.e. one stream gauging station per 1000km^2.

3.4 Flow Availability Analysis

The Gefersa I/II and III dam are located on Gefersa River (tributary to Tinishu Akaki River). The Gefersa river flow at dam I/II and III were estimated by a semi distributed conceptual rainfall-runoff model (HBV model). The inputs to the model are rainfall, climate parameters of the nearby stations and flow record collected from Tinishu Akaki gauging station. Accordingly, the estimated monthly available water at the Gefersa dam I/II and III sites and there dependability is summarized and presented on table below.

Table 1: mean and 90% dependable flow at Gefersa I/II and III sites (Mm3/month

Month	**Gefersa dam III**		**Between Gefersa Dam III and Gefersa I/II**	
	Mean	**90% Dependability**	**Mean**	**90% Dependability**
Jan	0.356	0.296	0.133	0.111
Feb	0.234	0.17	0.097	0.07
Mar	0.224	0.096	0.084	0.036
Apr	0.267	0.092	0.103	0.035
May	0.295	0.146	0.11	0.055
Jun	0.452	0.195	0.175	0.075
Jul	1.871	1.124	0.698	0.42
Aug	6.52	4.242	2.434	1.584
Sep	3.359	1.836	1.296	0.708
Oct	1.142	0.985	0.426	0.368
Nov	0.825	0.681	0.318	0.263
Dec	0.53	0.453	0.198	0.169
Annual	16.074	10.316	0.506	0.324

3.4.1 Flood Frequency Analysis

In flood frequency analysis, the objective is to estimate a flood magnitude corresponding to any required return period of occurrence. And the flood frequency analysis at the project site was undertaken by using the annual maximum flow series are extracted from the daily flow record of Tinishu Akaki gauging station. Based on fitness test result the annual maximum flow of Tinishu Akaki River with different return period was estimated and transferred to the upstream Gefersa Dam I/II site (Table 2).

Table 2: Flood Magnitudes of Different Return Periods on Gefersa Dam III Site

location	Area (Km^2)	Return periods (years)						
		10	25	50	100	500	1000	10000
Tinishu Akaki at gauging station	131	90	123	150	180	263	305	482
Tinishu Akaki at Gefersa I/II dam	35	36	49	60	71	104	121	191

3.4.2 Spillway capacity determination using Flood Routing

Gefersa Dam I/II heightening shall be designed with spillways capable of passing extreme floods in order to minimize the risk of failure. The justification for adopting designs with a low probability of failure is partly because of the economic consequences of failure and partly because of the risk to life should a dam break occur. Reservoir routing was carried out for 1000, 10000 years and PMF using the above data as input and Modified Pulse method. The Elevation Storage data for Gefersa Dam I/II reservoir at FRL and the outflow from spillway has been calculated based on equation below.

The outflow from spillway has been worked out using the following equation:

$Q = CdLH^{1.5}$

Where, Q = is the outflow discharge in m3/sec,

Cd =is the coefficient of discharge (2.17)

L = the length of spillway in m (30.0m)

H = the depth of flow located at the spillway crest level of 2613. 00m.a.s.l

Accordingly, the result of the Reservoir routing i.e. the peak out flow and also maximum water level is given on table 2 below.

Table 2: Maximum outflow and Water Levels of the Gefersa Dam I/II Reservoir

Return Periods	Maximum Out flow (m3/sec)	Maximum Water Level (m.a.s.l)
1000	6.6	2613.1
10000	89.4	2614.2

PMF	138.3	2614.6

3.5 Sedimentation

3.5.1 Estimation of Sediment Yield

The sediment data is obviously required to plan the size of the storages, for apportioning the active and inactive storage spaces and for determining the revised geometry of the storage after certain design life. The average annual sediment production from a catchment is dependent on many factors such as climate, soil type, land use and topography. The daily sediment load of the gaging station on Tinshu Akaki was analyzed for establishing conventional sediment-discharge rating curve given below (Figure 2-1).

$Q_s = aQ_w^{b}$ ……………………………………………………………………………….(2.1)

Where, **Q**s is the suspended sediment concentration in tons/day

Qw is the daily discharge in m3/s respectively

And ‘a’ and ‘b’ parameters varying depending upon the hydro meteorological characteristics of the watershed and estimated through regression analysis.

The method employed for calculating the result of the analyses showed that the specific sediment yield rate at Tinishu Akaki station is 232 Tons/Km2, which is equivalent to an annual sediment transport of 32 thousand tons/year.

3.6 Gefersa Dam I/II Intake Structure Foundation Site

3.6.1 General

Geo-technical evaluation of Gefersa Dam I/II intake site is done by means of exploratory core drillings. And on the basis of local geological setting and investigation revealed from borehole drilled at Gefersa I/II site and extrapolation of the previous drilled bore holes the proposed intake structure foundation site lies on rhyolite/andesite rock mass underlying shallow overburden soil mass.

3.6.2 Foundation of Gefersa Dam I/II Intake Site

Geo-mechanics Classification (Rock Mass Rating System) was used for classifying and interpreting the quality of the foundation rocks of the proposed site. Accordingly, Bieniawski classification method was employed for the foundation rock mass, the results are given in the table below.

Table 2: rock mass classification, based on Geo-mechanics classification method

Item	Rock type	UCS (Mpa)	RQD (%)	Spacing of Discontinuities	Condition of Discontinuities	Ground Water	Total	Rock mass Class
Zone-1 Ratings	Rhyolite /Andesite	4.67	27	45mm	<1mm separation, Slightly rough surfaces	wet	34	IV
Zone-1 Ratings	"	12.70	35	30mm	<1mm separation, Slightly rough surfaces	wet	36	IV

Accordingly, the rock mass at Gefersa dam I/II fall on Rock Mass Class IV, which is poor rock. Laboratory test of rock samples revealed that intact rock nature possesses specific gravity of 2.44 to 2.57, and UCS ranging from 4.67 – 12.70Mpa. As can be inferred from Table 3-10 below, the rock mass at dam site has poor quality, with estimated cohesion between 170 to 180 Kpa, angle of internal friction ranging from 220 to 230, and modulus of deformation in the range of 3 to 4.5GPa.

3.7 Geometry of Existing Dam

The existing Gefersa I/II dam was constructed using homogeneous clay soil as major construction material. To pass the phreatic line through the foundation of the dam, fine gravel material is provided with 45m length with 0.8m thickness. The total bed and top width of the embankment are fixed as 149.35m and 5m respectively. The total height of the existing dam is around 19m above bed of the river. The upstream and downstream side slope of the embankment is 1V:2.58H and 1V:2.27H respectively. Figure 5 shows the detail of the detail of the dam section.

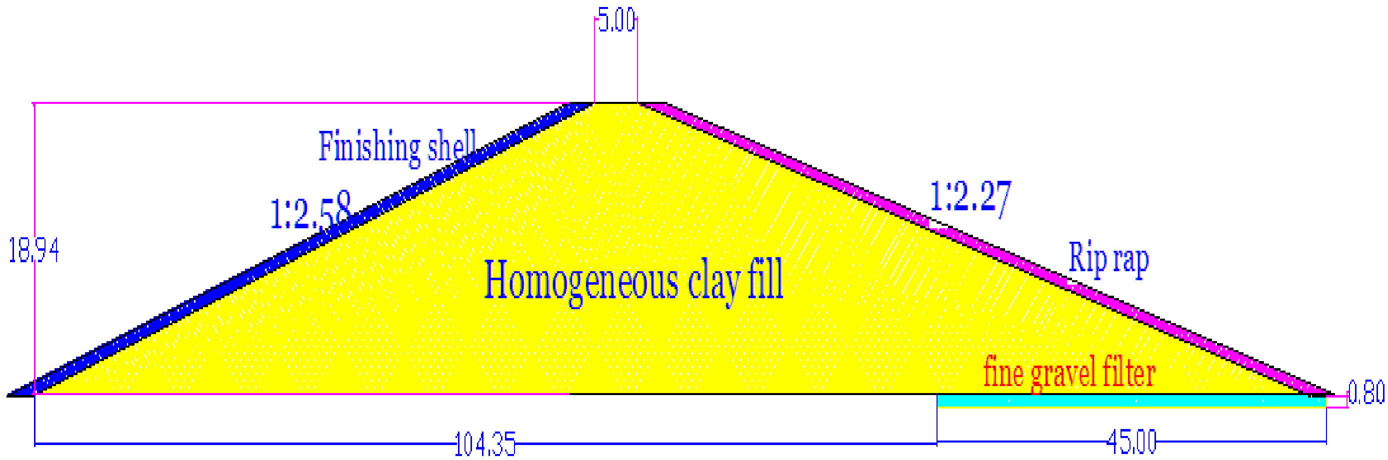

Figure 4: existing geometry of Gefersa I/II dam

3.8 Gefersa Dam I/II Heightening Design

3.8.1 Dam Geometry

3.8.1.1 Dam Heightening Determination

To decide on the dam magnitude of Gefersa Dam I/II heightening the following procedure applied. This phase considers raising the dam crest level considering hydrologic study to provide a sufficient reservoir capacity. In this stage a 5m effective height and 2m free board (i.e. 7m) dam heightening including has been found to provide a sufficient 3 million meter cube of water storage reservoir.

43.8.1.2 Crest Width

The crest width adopted for the rehabilitation dam considers the following factors:

- Minimum allowable seepage distance through the embankment at normal reservoir water level.
- Road way requirement.
- Practicability of construction.

A minimum crest width should provide a safe seepage gradient through the embankment at the level of maximum reservoir, according to USBR standard of embankment design under DS-13(2)-9.

A minimum of 3m crest width is required to allow compaction with normal rollers. A crest width greater than 6m or 8m is seldom required. (R. Fell, 2005)

According to the US ARMY CORPS guideline (EM1110-2-2300), the minimum top width should be between 25-40 feet depending on the height of the dam. This considers the ability to carry a public highway, road and shoulder widths (US Army Corps, 1994).

Considering the above guidelines, previous dam crest width performance and checking the seepage requirement the rehabilitation dam will have a 7m crest width.

3.8.1.3 Slopes and Berms

Upstream and downstream slopes are main geometrical determinants for stability. The flatter the slope the more the design is conservative. Berms can be provided to providing level surface for construction and maintenance of the dam section and reducing the surface erosion in case of downstream slope and breaking the continuity of the slope.

Since all the fill is in the downstream side, the heightened dam will use from the previous dam crest of 2m width as an upstream berm. The 7m fill starting from the existing dam crest will have a slope of 1: 3.0(V: H) at the upstream face. The downstream side has one berm width of 7m and the downstream slopes are 1:1.45 and 1: 4.17 (from top to bottom).

3.8.1.4Crest Shift

The new heightened dam crest center is shifted 25m to downstream. This simplifies the work methodology in minimizing works in the upstream face and flexibility in lowering the upstream stored water.

3.8.1.5 Dam Design Analysis

The dam heightening material type and geometry are selected and analyzed considering material availability, compatibility with the previous dam and requirement of less workmanship. The static and dynamic analysis of dam stability is satisfied. The factor of safety to be satisfied is adopted from US Army corps of Engineers for static stability (DS-13(4)-5), where end of construction stability lower acceptable value is 1.4 and where steady state seepage lower acceptable value is 1.5. (US Army Corps, 1987) But for the earthquake considering cases all factor of safeties have been checked to be greater than 1 as recommended by the guideline US Army Corps of Engineers (1984b).

The existing downstream protection ripraps and a 0.5-1.0 m of top clay soil will be cleared from the downstream face (the clearance of the existing top soil will be determined by the supervisor at the time of construction). Clearing of some specific depth of the existing soil is necessary to avoid weak junction as a result of decay of organic material. The new downstream fill starts at 2m away from the upstream edge of the existing dam crest. As a result a 2m width of berm will be created in the upstream side.

3.8.1.6 Geometry of the New Heightened Dam

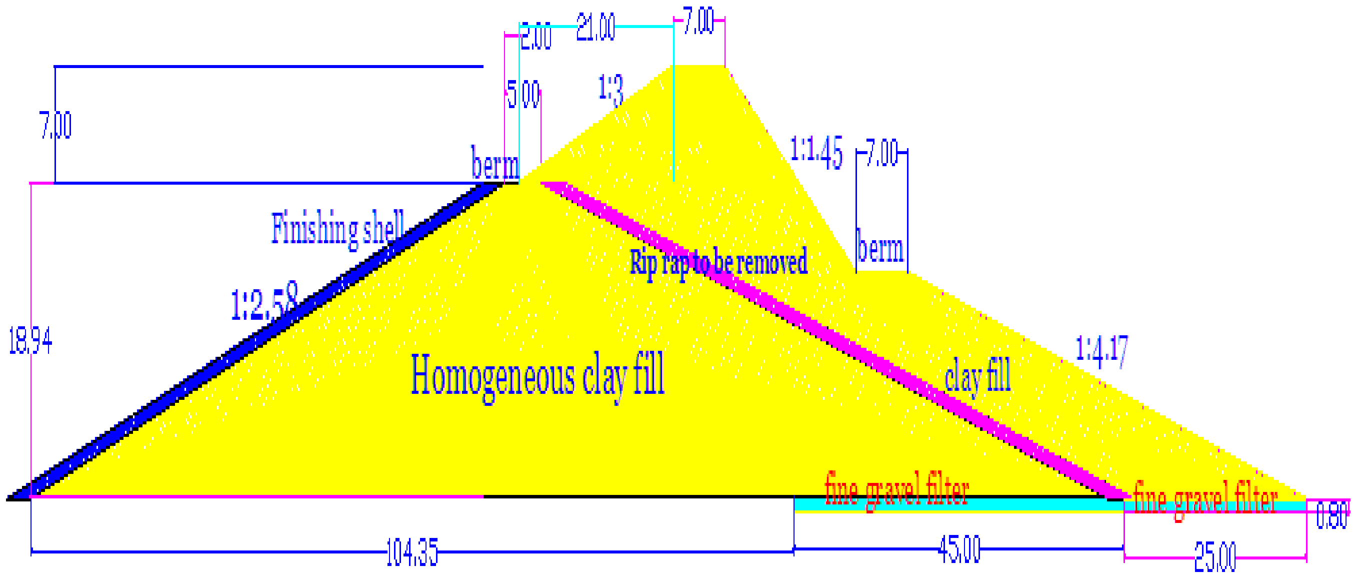

Figure 5: geometry of the heightened Gefersa I/II dam

Sudden draw down is one of the main cases to be considered for upstream slopes in case of fine material fill dams. This is due to the slower rate of release of pore water pressure in fine material fills. The upstream supporting water will be removed quickly, while pore water pressure is developing in case of sudden drawdown. As a result the minimum acceptable number of days of drawing the reservoir water down must be given for safety purpose. After analyzing in such a way that the minimum days allowed for rapid drawdown is 20 day lowering water level from 23m (Full supply level) to 13m. The lower upstream slope stability factor of safety occurs at the last days of draw down. But the lower factor of safety determined is 3.017, which is above the minimum to be satisfied 1.2, according to USBR manuals.

CHAPTER FOUR

AAWSA INCEPTION REPORT

Geotechnical Investigation Study

Draft Feasibility Report

1. INTRODUCTION

1.1 Overview

Ethiopian Construction Design and Supervision Works Corporation, ECDSC (the former Water Works Design and Supervision Enterprise, WWDSE) has been entered into a contract agreement with Addis Ababa Water and Sewerage Authority (AAWSA) to conduct feasibility study and detailed design of intake (Gefersa I /II) and dam (Gefersa-III Dam) structures. The geotechnical investigation also covers construction materials investigation for planned dam height extension work.

Gefersa Dam is located on the Addis Ababa - Ambo Asphalt road approximately 18 km north-west of Addis Ababa city center. Specifically, it is found in Oromiya National Regional State FinfineZuriaBurayu City Administration. Gefersa-I/II dam is situated to the south of the main road, geographically at 38^{o}38'30'' East and 9^{o}4'15'' North. Gefersa-III is found upstream of Gefersa-I/II dam north of the main Addis Ababa – Ambo asphalt road. It is geographically located at 38^{o}38'00'' East and 9^{o}4'45'' North.

The project area is part of the central highlands of Ethiopia with elevation ranging from 2000 to 2800 m.a.s.l. The topography of the project site and its surrounding area consists of hills, valleys and rivers and many small streams. The temperature is fairly constant throughout the year between 20^{o}C and 25^{o}C during day time and between 7^{o}C and 11^{o}C during the night. The average rainfall is 1200mm per year with the major rainy season from July to September.

As part of the contract agreement and project scope, geotechnical investigation has been planned and implemented at structure sites of the project salient features: dam site, Intake, and reservoir area. The investigation also included assessment of construction materials for dam fill and appurtenant structures construction.

The methods of geotechnical investigation conducted at the project site has involved geophysical surveying, rotary core drilling, insitu testing in boreholes, groundwater level monitoring, Pit/trench excavations, collection of representative soil, rock and water samples, and laboratory analysis of samples. Information collected from existing documents review and from current study were analyzed and interpreted to characterize the engineering performance of proposed construction sites of the project.

The factual data are prepared with proper formats and presented with this report as annexes: Analyzed in-situ tests, engineering geological and discontinuity logs of boreholes, excavated test pit and trench logs, and detailed laboratory analysis data for soil and rock samples tested.

The details on executed geotechnical investigation activities, adopted standard procedures, geotechnical evaluation of studied sites, the findings of the study, and forwarded recommendations over identified geotechnical problems and/or information gaps are presented with this report.

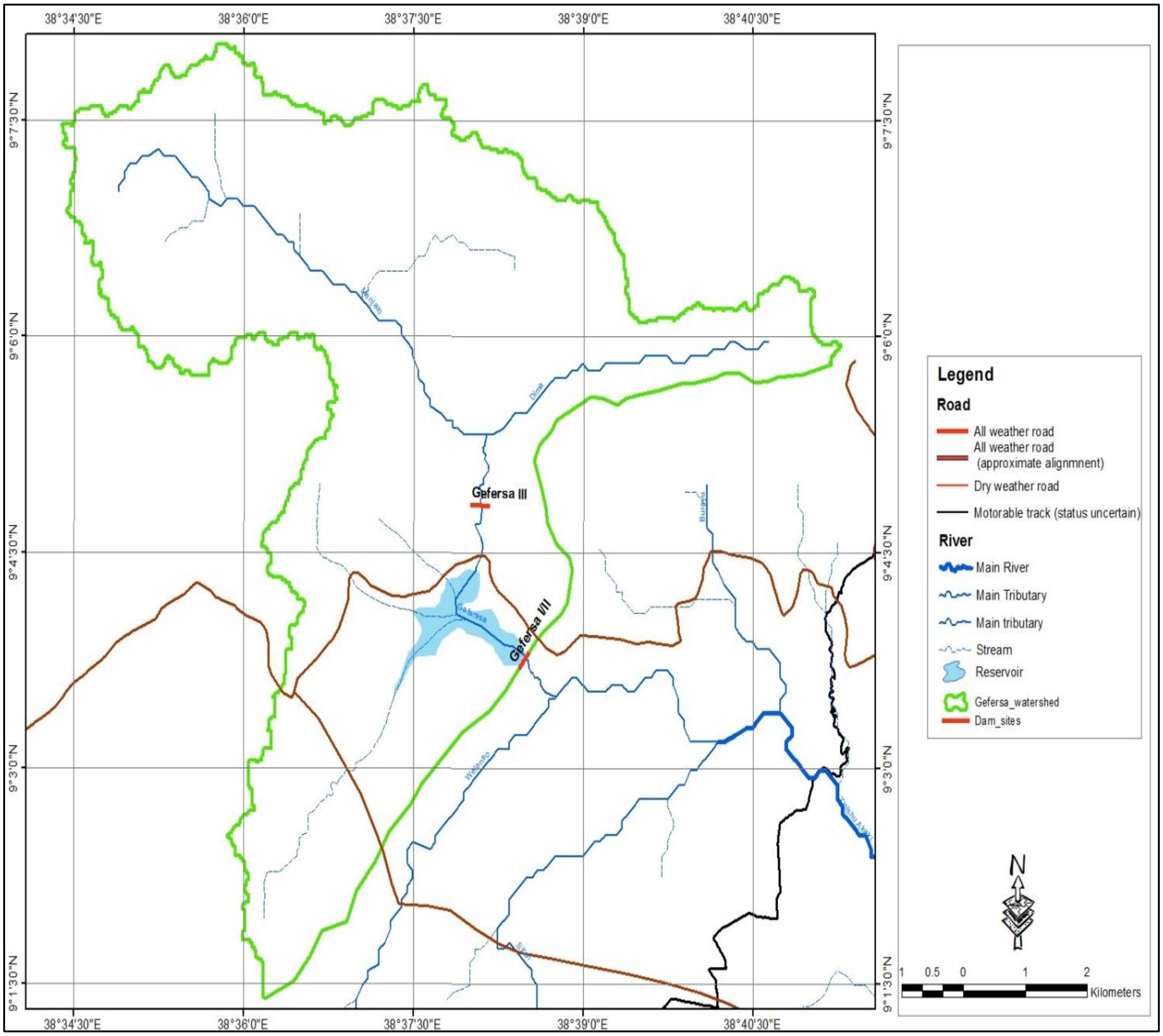

Figure 1.1: Location Map of the Project Site

1.2 Objectives and Scope

1.2.1 Objectives

The current geological and geotechnical investigations are planned to be executed for feasibility studies and detail design stages embodying the following objectives:

- Understanding the surface and subsurface stratification, geological structures, and engineering classification of earth materials composing the foundations of engineering structures;

- Suitability of the site for the proposed engineering structures with respect to identified and anticipated geotechnical aspects and workability;
- Assessment of the bedrock conditions and evaluation of engineering properties of the ground materials that compose major project structure sites up to engineering reasonable depths/extents, for rating parameters such as: bearing capacity, deformability characteristics, erodibility potentials, differential settlement potentials, volume change characteristics etc.
- Identification of suitable construction materials with sufficient quantity at reasonably close proximity to the engineering structure sites.
- Foreseeing construction challenges and suggesting remedial measures during and post-construction phases.

1.2.2 Scope

The geological and geotechnical investigation covers the following activities:

- Conducting foundation investigation for the proposed intake and dam sites.
- Assessment of reservoir water tightness.
- Construction material assessment for proposed project demand
- Preparation of the geotechnical investigation report along with maps, drawings, logs, cross sections and recommended measures against identified geotechnical problems

The geotechnical investigation follows sequential and standard methods and procedures to meet desired objectives over the execution of each scope of the study.

1.3 Methodology

The geotechnical investigation of the project was carried out adopting integrated investigation methods sequentially. The investigation was approached through desk work, field work and laboratory studies of representative samples from which information generated are analyzed and interpreted for the geotechnical characterization and evaluation of proposed construction sites and construction materials.

Desk Work

The desk work at the early stage of the investigation was necessitated to gather information mainly on the regional geologic setting, geodynamics and geomorphology, and the nature of proposed project. The desk work objectives were met through review of previous geological and geomorphological studies, interpretation of satellite imageries, review of project area previous works and project contract documents. Information collected from the desk work were relevant in investigation Planning and preliminary understanding of studied sites. Generally, the following were part of the desk work review:

- Geology of Addis Ababa City - NC37-10/X and Y and NC37-6/E and F, (Assegid, 2007).
- Addis Ababa-Nazareth Volcanic: A Miocene-Pleistocene Volcanic Succession in Ethiopian Rift (Haile Sellasie Girmay and Getaneh Assefa, 1989).
- Engineering geological mapping of Addis Ababa (EIGS, 1990)
- Geotechnical investigation of Gefersa dam Rehabilitation (WWDSE, 2000)
- Project documents: Contract document, geotechnical investigation, topographic maps, and drawings.

Field Work

The geotechnical investigation has used both direct and indirect methods of investigation to collect geological, engineering geological and geotechnical information of surface and subsurface materials comprising project sites. The field work has involved the following methods and activities:

- Surface outcrop description and measurements
- Geophysical Survey (vertical Electrical sounding and Electrical imaging)
- Geotechnical core drilling
- Insitu testing: Standard penetration Test (SPT) and permeability tests (falling head, packer tests),
- Collection of representative soil rock and water samples for laboratory testing
- Excavation of test pits/trenches
- Insertion of auger holes
- Logging information from borehole drilling, insitu testing, pit excavation and auguring.

- Preservation of drilling core samples in standard core boxes

Laboratory studies

Required laboratory tests were conducted on samples obtained from the field work. Soil and rock samples were analyzed for index and engineering properties required and possible. Full water quality testing was made to water samples obtained. The laboratory tests were carried out in the corporation laboratory service at Addis Ababa.

2. REVIEW OF PREVIOUS STUDIES

The project area is located in the southwestern part of Addis Ababa. Addis Ababa area has undulating morphology intersected by Big Akaki, Kebena, Little Akaki Rivers and their tributaries. The top of Entoto Ridge forms the water divide of the Nile and Awash River Basins where the project area lies within Awash River Basin.

Available data pertaining to the geological conditions of the project site were collected and reviewed. The following documents were available for review of the previous works and preparation of this inception report:

- Geology of Addis Ababa City - NC37-10/X and Y and NC37-6/E and F, (Assegid, 2007).
- Addis Ababa-Nazareth Volcanic: A Miocene-Pleistocene Volcanic Succession in Ethiopian Rift (Haile Sellasie Girmay and Getaneh Assefa, 1989).
- Engineering Geological Mapping of Addis Ababa (EIGS, 1990)
- Geotechnical Investigation of Gefersa Dam Rehabilitation (WWDSE, 2000).

The geology of Addis Ababa City (Assegid, 2007) shows that the main geological formations which composed the relatively low lying gently sloping to flat land areas of Addis Ababa including south western, central and north eastern south part of Addis Ababa are ignimbrites, pyroclastic and recent sediments. The ignimbrite is fine to coarse grained and show columnar jointing. The south western part of Addis Ababa is composed of lower ignimbrite and pyroclastic. It is fine to medium grained and show columnar jointing. The western part of Addis Ababa area is dominantly composed of olivine basalts, trachyte and trachy basalts with intercalated pyroclastic rocks. The southern and eastern part of Addis Ababa is composed of olivine basalts, layered basalts, and trachyte and trachy basalts with pocks of pyroclastic rocks and welded tuffs.

According to the study conducted by Haile Selassie Girmay and Getaneh Assefa (1989), the geology of Entoto Ridge is composed of basalts, trachyte, rhyolites, ignimbrites, tuffs and associated pyroclastic materials. Trachyte and rhyolites with associated pyroclastic materials (Entoto Silicic) dominate the top and foothills of Entoto Ridge whereas the relatively flat land area of Addis Ababa including south western Addis Ababa, Sululta, Nefas Silk, Filwoha, Ayat, Sendafa and Lege Tafo localities is dominated by ignimbrites, tuffs and related pyroclastic. The area around Bole international airport is covered by aphanatic basalt. Further south, the geology of Addis Ababa area is composed of the Nazreth group.

The work by Kebede Tsehayu and Tadesse Haile Mariam (1990) were conducted for engineering geological mapping of Addis Ababa area. According to the work, the geological formations were mapped in to rock units and soil units. The mapped rock units in the area are the trachyte and rhyolite, ignimbrite tuff, trachy basalt and basalt whereas the soil units being alluvial, fluvial, lacustrine and residual sediments.

3. The geotechnical investigation report of Gefersa Dam rehabilitation (WWDSE, 2000) reveals that the geology of the GEOLOGIC SETTING

3.1 Regional Geology

The general geology of the area around the project site varies slightly from place to place. The project site is dominated by Foota basalt which is strongly weathered and jointed basalt, Wechecha-Yerer-Fuji ignimbrite and Quaternary olivine phiricbasalt. It is fine grained, moderately jointed and dark grey to black in color. The upper part of the rock is highly weathered and altered to soil mass. Ignimbrite rock composed the geology of the area east and west of the project site. The ignimbrite rock is light grey in color, fine to coarse grained and columnar jointed at the top. To the south west around Salo area there is a dome shaped hill entirely composed of scoria rock. Further away towards east there is Tertiary basalt rock. The rock units around the relatively flat lying areas are overlain by residual soil deposit.

3.1.1 Quaternary olivine basalt

This basalt is grey color on fresh out crop and becomes reddish brown up on weathering. Most of the time the rock out crops in boulder form. In hand specimen phenocrysts of olivine and pyroxene are clearly seen. Sometimes the rock is vesicular and the vesicles are filled by secondary zeolite and quartz. This rock uncomfortably overlies the Wechecha-Yerer-Furi ignimbrite and the Wechecha-Yerer-Furitrachyte and trachy basalt. Thin section studies of samples from this unit show about 60% plagioclase, 20% pyroxene, and 5% opaque minerals and 15% ground mass. The groundmass is made up of feldspar, pyroxene, opaque and other unidentified minerals.

In some locality minor rock type composed of 15% plagioclase, 20% olivine minerals and with 60% groundmass is also observed. The groundmass is composed of micro-phenocrysts of pyroxene, plagioclase laths, and olivine and opaque minerals. This rock type exhibits pikiliotic texture.

3.1.2 Wechecha-Yerer-Furi ignimbrite

It is grey, which contains fragments of ignimbrite, rhyolite and pumice with sanidinephenocrysts. It is fine to medium grained in texture. At the top part, it shows columnar jointing whereas at the base it shows layering. In the lower most part and most top part of this unit there are pyroclastic deposit which contains phenocrysts of sanidine. The pyroclastic layers are multiple with each layer separated by thin paleosoil. Thin section studies of this rock show a composition of 52% glass (ash) 18% sanidine crystals with accessory minerals of 4% magnetite and 5% hematite. The secondary minerals observed are 3% augite, 5% rock fragments and 5% quartz. It has a flow texture and also eutaxitic by major constituents which are glass or ash. But the ash flow/ tuff which is found at the lower most and at the upper most part of the unit shows 69% quartz with 10% opaque and 5% hematite accessory minerals. Micro phenocrysts of 5% quartz, 3% sanidine, 3% plagioclase and 5% glass are also observed. This pyroclastic deposit shows layering of a millimeter thickness with major constituent showing radiating and fibrous texture.

3.1.3 Foota basalt

This basalt is dark grey on fresh out crops. Upon weathering, it has developed a laterite with maximum thickness of 2 m and locally it shows spheroidal weath of samples of aphanitic textured basalt shows about 99% ground mass and less than 1% plagioclase. The where the ground mass made up of olivine, pyroxene and feldspars. It is characterized by sub horizontal layering. Locally the rock is affected by 2 joint sets. It shows alternating layers of vesicular basalt with either porphyritic basalt or aphanitic basalt. Thin section studies on a number.

3.1.4 Wechecha-Yerer-Furi trachyte and trachy basalt

This trachyte and trachy basalt is light grey to dark grey often to greenish grey. Weathered surfaces show various colors. It has aphanitic to vesicular texture. At its lower part it shows columnar jointing and is affected by two set of joints. At some places it shows layering. Mostly the trachyte and the trachy basalt are found alternatively layered with the trachyte being dominant. The base of the unit is dominated by trachy basalt to basaltic rock type.

Thin section studies on samples from this unit shows a composition of 20-60% plagioclase, 5-12% olivine minerals and 15-70% ground mass. The groundmass is made up of olivine, pyroxene, plagioclase and opaque minerals. The plagioclase minerals lath-like crystals. The pyroxene and olivine minerals are weathered to form secondary minerals of augite and idding site. The rock shows poikilitic texture. Moreover, a porphyritic trachyte was locally observed and is composed of 30% feldspar (sanidine), 10% pyroxene minerals and 60% groundmass. Micro phenocrysts of mainly feldspar and pyroxene form the groundmass.

3.2 Local Geology

The local geology of the project area has variation from place to place. At Gefersa I/II intake structure site the local geology is composed of andesite/rhyolite overlain by shallow residual/ back filled soil cover. The rock is light grey to creamy in color, moderately to highly weathered and intensively fractured. The rock is about 20 m thick and is underlain by pyroclastic material.

At Gefersa III dam site the geology is composed of thick residual soil deposit formed from prolonged time of weathering and decomposition of the underlying parent rock. The thickness of the soil reaches up to 24 m on the left abutment and the top about 8 m thick soil has organic content. Underlying the top soil horizon with high organic content is the residual soil mass characterized by variegated colors of brownish, red, whitish, and yellowish. The grading of the soil generally dominated with cohesive soil mass with sand (silty clay, clayey silt and sandy silt.

The reservoir area of Gefersa III dam generally is dominated by rolling and slightly sloping to flat land topography. The area is intersected by River and its tributaries. Rocks are observed on exposures at river and stream cuts and rarely on slightly sloping areas near rivers. Almost the entire reservoir area is covered by about 1-3 m thick residual and alluvial soil of sandy silty clay and sandy clayey silt composition.

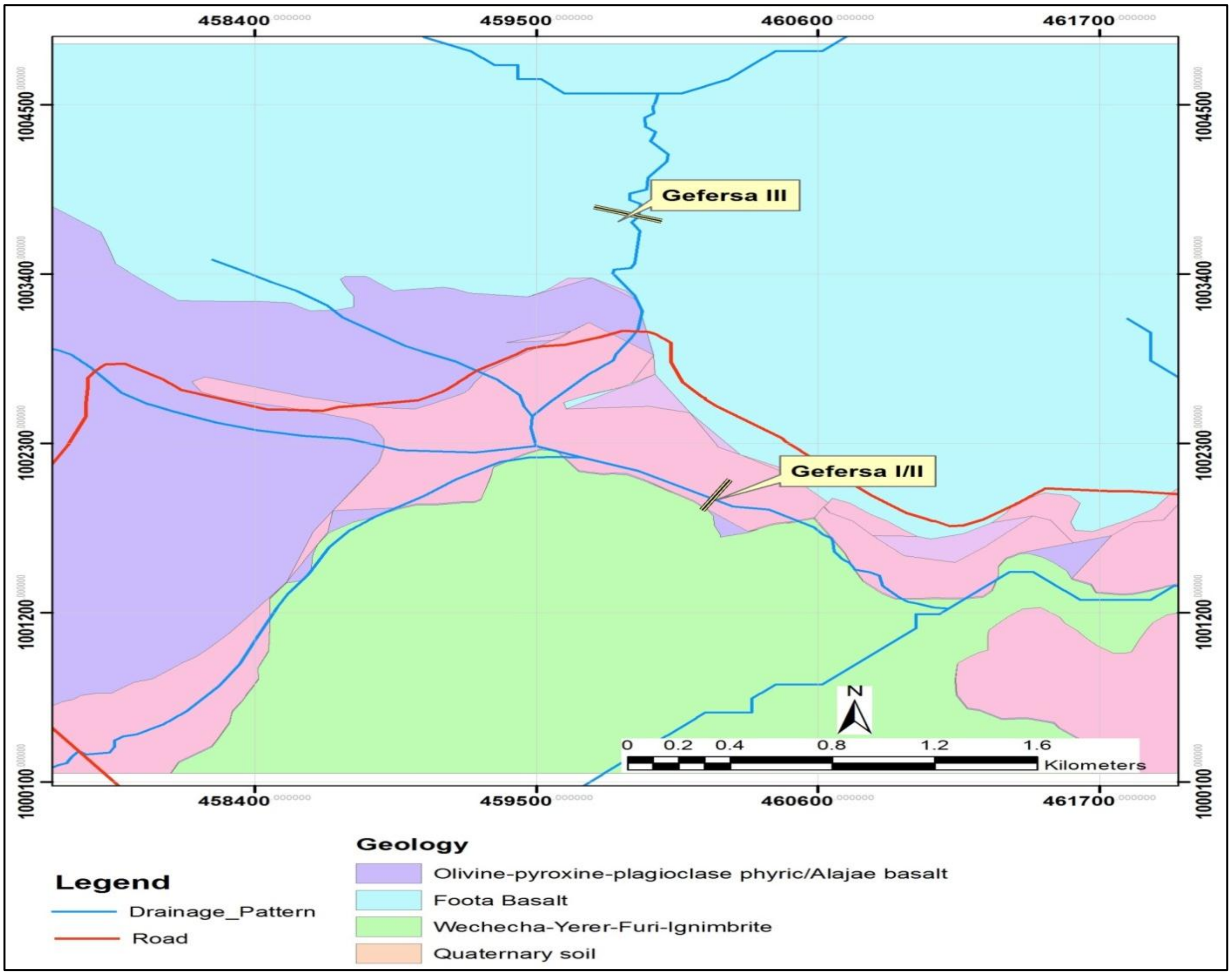

Figure 3.1: Geological Map of the Project Area (Extracted from Assegid, 2007)

4. Geotechnical Investigation

The site investigation has covered geophysical surveys, geotechnical core drilling, insitu testing, sampling and Laboratory testing activities. The activities were carried out in accordance to project scope and following standard procedures.

4.1 Geophysical Survey

Resistivity geophysical surveying, using Vertical electrical sounding and electrical imaging methods, was carried out for proposed project headwork construction sites at Gefersa III rehabilitation project. The survey was conducted in order to meet such objectives: to map the subsurface geologic condition based on interpretation of resistivity information measured, to locate the extent and nature of geologic structures or weak zones in foundation, groundwater table indication, and to help on interpolation of information between geotechnical boreholes Electrical resistivity values measured from the field work were analyzed and interpreted using appropriate software, technical standards and calibration to actual condition geologic information. Results are presented in tabulated summaries and in a form of imagine sections and graphs for corresponding imaging lines and VES points tested.

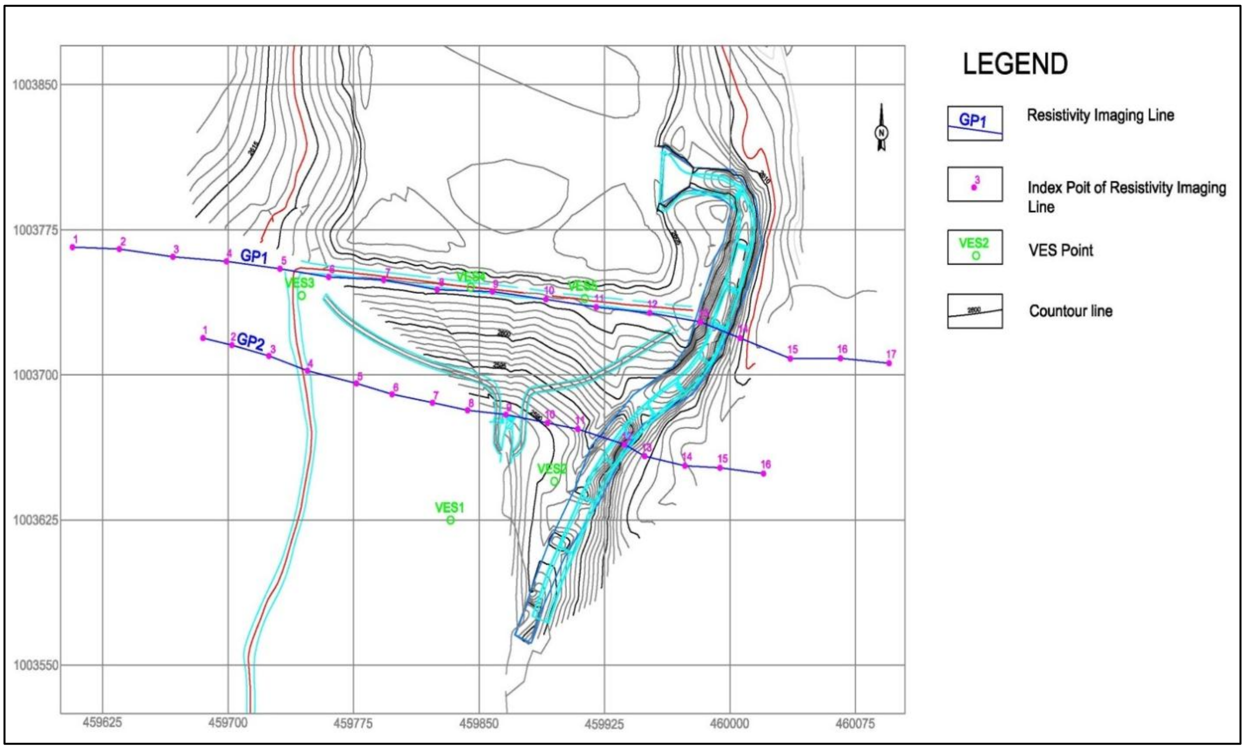

Figure 4.1: Geophysical investigation survey lines

A total of 2 resistivity imaging survey lines with a total length of 865 m were conducted. The location, length of survey line and depth of investigation are summarized in Table-1 below for corresponding electrical imaging survey lines.

Table 4.1: Coordinates and work volumes of resistivity imaging survey lines

No.	Survey	Coordinates (UTM)	Length of	Depth	Remark

	Line	Starting		Ending		survey line	of investigation	
		Easting	Northing	Easting	Northing	(m)	(m)	
1	GP1	459607	1003766	460095	1003706	508	40	Dam axis
2	GP2	459685	1003719	460020	1003649	357	30	Downstream
Total						**865**		

A total of 5 VES points were surveyed. Table2 below presented information on eachVES point location and the corresponding depth penetrated.

Table4.2: Coordinates and work volumes of VES points

No.	VES point	Coordinate		Length of AB/2	Remark
		East	North	(m)	
1	VES1	459833	1003625	66	Downstream
2	VES2	459895	1003645	45	Downstream
3	VES3	459744	1003741	100	Dam axis
4	VES4	459845	1003745	100	Dam axis
5	VES5	459913	1003740	100	Dam axis

Soil and rock resistivity information, collected from resistivity imaging and vertical electrical sounding survey, were analyzed and interpreted using ground computer tomography and win resist software's respectively and corresponding resistivity sections (2D for imaging and 1D for Vertical Electrical Sounding survey) are produced for each survey line/point. The geophysical survey findings were useful to preliminary understanding and interpretations of the geological configuration of the proposed site.

Generally, the geophysical survey, conducted at Gefersa III site, has revealed the following:

- According to the resistivity imaging section at the dam axis, the relatively low resistivity values are reflected at the sub parts marked with elliptical block around electrode numbers 40, 100 and 115, which is considered to be the response of highly weathered basalt.
- On the resistivity imaging section, the relatively low resistivity values reflected at the sub part marked with elliptical block between electrode numbers 54 and 94 located on the dam axis may be due to clay and highly weathered basalt.
- As shown on the resistivity imaging section, the relatively high resistivity values are reflected at the blocks between electrode numbers 1 and 33, 44 and 53 and 104 and 111 as well, and it is interpreted to be the response of highly/moderately fractured basalt.

- On the resistivity imaging section found at downstream of the Dam axis, the relatively low resistivity values less than 14Ω·m reflected at the sub parts marked with elliptical block around electrode number 23, 65, 90 and 110 may be due to the highly weathered basalt.
- On the resistivity imaging on the downstream side of the dam axis, the resistivity values except the sub part marked with elliptical block are higher than 15Ω·m, and it is expected to be the response of top bed (between electrode numbers 8 and 31) and highly/moderately fractured basalt (between electrode numbers 36 and 52 as well as between electrode numbers 69 and 78).

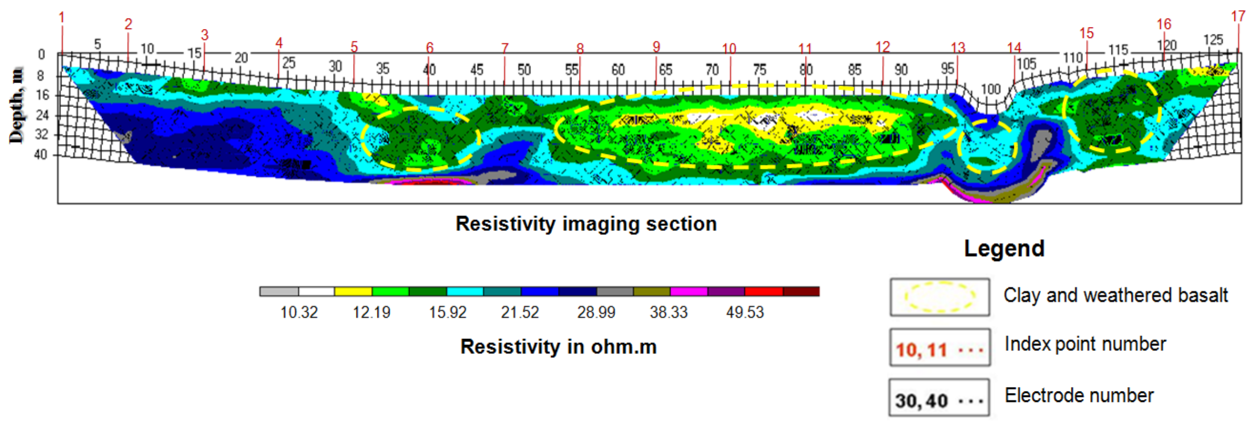

Figure 4.2 Resistivity imaging section along survey line GP1

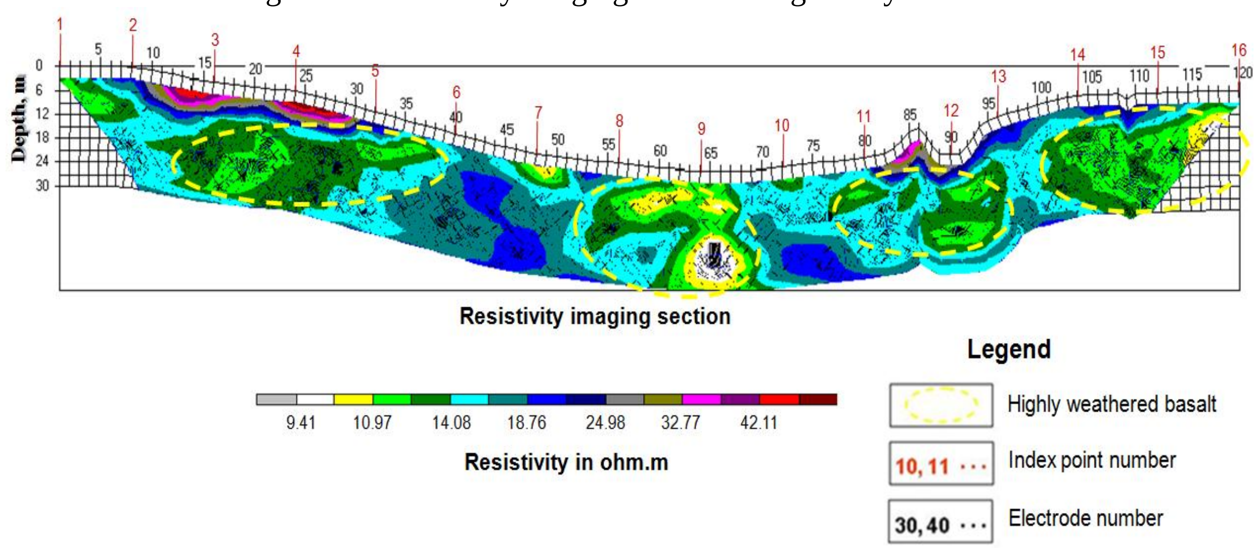

Figure 4.3: Resistivity imaging section along survey line GP2

4.2 Geotechnical Core Drilling

Geotechnical core drilling at the current project structure sites is associated with diverse purposes including:

- To obtain subsurface material stratification up to reasonable depths,
- To conduct in-situ tests on subsurface soil and rock mass,
- To understand the groundwater condition of the area, and
- To obtain samples of subsurface materials for direct physical examination and laboratory studies.

For the current project 3 boreholes (1 borehole at Gefersa I/II intake structure and 2 boreholes at Gefersa III) were completed as planned for Gefersa Rehabilitation project. Accordingly, a total of 3 boreholes with a total depth of 76 m were drilled along the proposed dam axis and the intake structure sites.

The Geotechnical Core Drilling and extraction of soil and rock core samples was conducted in accordance with ASTM D2113-99: Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation, by using rotary core drilling method. Machenza wire- line drilling rig of the Corporation was deployed to perform the drilling operation. Boreholes were drilled with diameter range of 76 to 96mm that led to recover 48 to 63 mm diameter core samples. Standard Penetration Tests and permeability tests (falling head and packer tests) were carried out in boreholes as appropriate. Undisturbed soil samples were collected from cohesive soil layers using Shelby sampler.

Recovered core samples from the drilling works are preserved properly in wooden core boxes designed to standards and in accordance with technical and contract agreements. The average drilling Total Core Recovery (TCR) in each borehole exceeded 80 percent recovery.

Geological and engineering geological information were collected from the drill cores and recorded with appropriate log formats for each borehole (Annex-1). Geological and discontinuity logs have been prepared for each borehole. Furthermore, drill core samples were photographed using high resolution digital camera as preserved in core boxes.

Table 4.3: Coordinates and drilled depth of the boreholes

S.No	BH-ID	Easting	Northing	Elevation (m)	Drilled Depth(m)	Remark
1.	GI/II-BH-1	460140	1001811	2590	35.0	Intake
2.	DG-35	459975	1003733	2606	30.0	Dam axis
3.	DG-36	459013	1003686	2593	11.0	Downstream

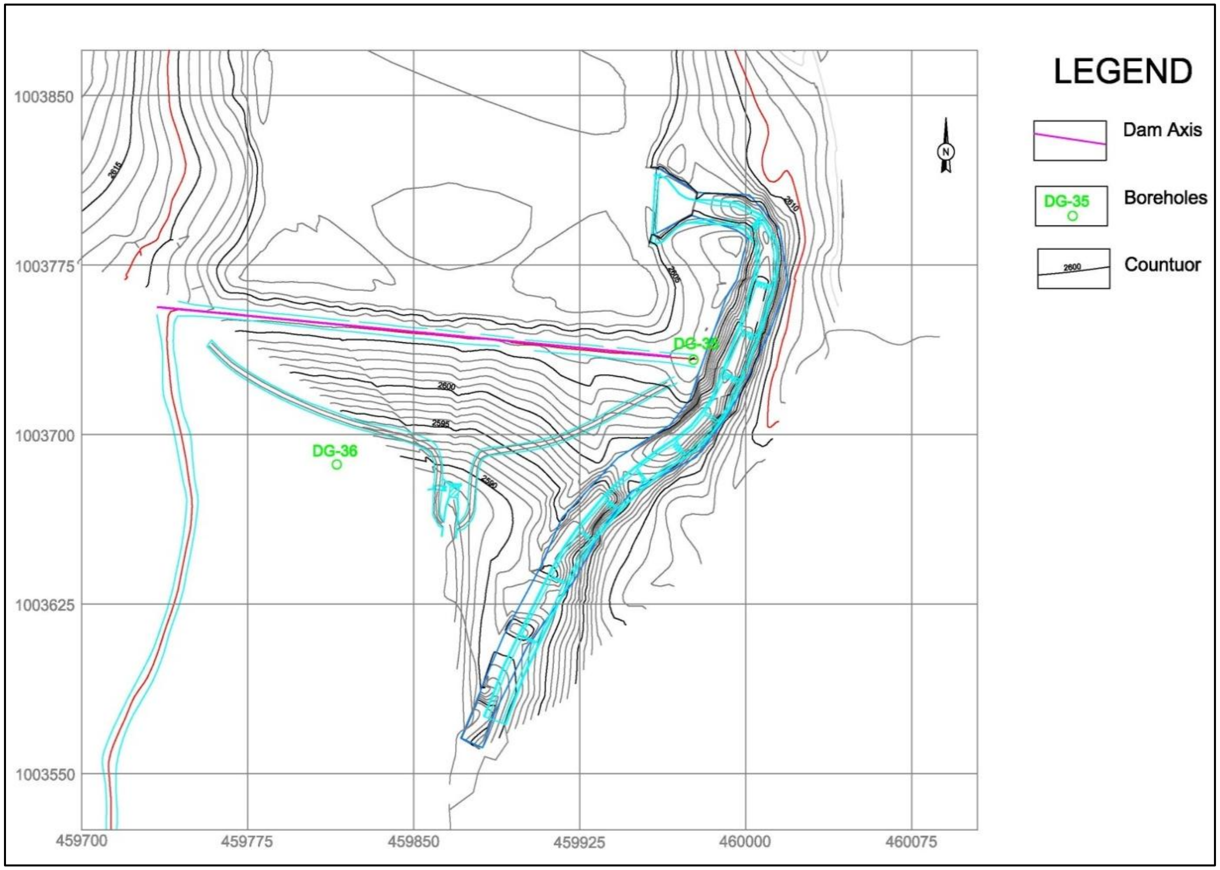

Figure 4.4: Investigation Borehole Location map at Gefersa III

4.3 In-situ Testing

In-situ strength/state of compactness and permeability characterizing tests were carried out to soil and rock mass intercepted in boreholes where the actual condition found is in accordance to the particular test type requirements. Standard Penetration Test, falling head permeability test and constant head permeability tests were conducted in borehole where appropriate.

4.3.1 Standard Penetration Test (SPT)

Standard penetration test (SPT) was done on soil layers encountered in boreholes. Standard SPT hammer with trip hammer release mechanism and imparting 60% hammer efficiency was used for the testing at all times. Regular standard penetration tests (SPT) were generally undertaken starting from depth of 1.5 m. The regular testing interval for a particular soil zone was kept 1.5-3.0 m. Sampler tube was in use during the operation, and disturbed samples recovered from the testing sampler tube were examined. The measured field penetration resistance/ number of blow counts (N) was considered for the number of blows required for the last 30 cm penetration of SPT rod in to the soil stratum.

Penetration was considered refusal where no penetration was observed for 10 successive blows or where more than 50 blows were required for the 30 cm penetration length or sum of blow counts of 45cm penetration required more than 100 blows. Numbers of blows were recorded for every 15 cm penetration. The measured penetration resistance (N) could not be directly used for further geotechnical evaluations as the theoretical input energy is believed to be affected by several factors. The hammer efficiency, diameter of borehole, drill rod length, the type of sampler involved, and the over burden pressure are known factors to be considered in the approximation of the amount of energy which is transferred to the actual penetration driving energy.

Table 4.4: SPT Result Summary

BHID.	**SPT No.**	**Depth (m)**	**Soil Description**	**Penetration Resistance**				
				N_1	N_2	N_3	N	N_{60}
DG-35	1	17.65– 18.10	Residual Clay	13	14	13	27	21
	2	20.0 – 20.45	“	6	7	8	15	12
	3	22.55 – 23.0	“	4	10	21	31	24
	4	24.50– 24.95	“	9	18	27	45	35
DG-36	1	1.50 – 1.95	“	4	4	4	8	6
	2	3.45 – 3.90	“	2	4	4	8	6
	3	10.20– 10.65	“	7	13	13	26	20

4.3.2 Falling Head Permeability Test

A total of 5falling head permeability tests were carried out in geotechnical auger holes to assess the water tightness of the reservoir area of Gefersa Dam III. The falling head auger hole permeability tests were conducted on soil layers.

The test procedure was undertaken in accordance to the British Code of practice for site investigation BS5930 (1999) and U.S. Bureau of Reclamation Procedures.

Falling head permeability testing was performed by filling the auger hole with clean water and measuring the fall of the water level from the top of the auger hole. The water head drawdown in the hole was recorded for periodic time intervals.

The test was kept continuous until the falling water level shows equilibrium condition or sufficient numbers of readings were collected.

The collected information was therefore analyzed using proper format and standards (Annex 2). The general borehole variable head permeability computation formula, provided in BS5930 (1999), was adopted for determination of the tested section permeability value:

$$\boldsymbol{k = \frac{A}{F(t_2 - t_1)} \log_e \frac{H_1}{H_2}}$$

k = The permeability of soil;

H_1 = The variable head measured above groundwater table at time t_1 after commencement of test;

H_2 =The variable head measured above ground water table at time t_2 after commencement of test;

A = The cross-sectional area of borehole

Table 4.5: Summary of in situ infiltration test result of Gefersa III reservoir area

Auger ID	Location			Test Depth (m)	In situ Permeability (cm/sec)
	Easting	Northing	Elevation		
GAH-1	459814	1004248	2605	1.5	4.57×10^{-5}
GAH-2	459855	1004319	2602	"	7.45×10^{-5}
GAH-3	459979	1003878	2610	"	2.35×10^{-4}
GAH-4	460080	1004161	2609	"	1.34×10^{-4}
GAH-5	460040	1004454	2608	"	5.14×10^{-5}

4.3.3 Constant head Test

A total of 4 constant head permeability tests were carried out in geotechnical boreholes where appropriate. The standard practice of the test procedure and plant arrangement was applied for the testing operation:

- Borehole drilling to desired depth of test section,
- Washing of test section, for removal of drill cuttings, until the returning water from borehole appeared clean
- Lowering proper size packer to the top of the test section, and inflated with required sealing pressure magnitude
- A pressure of 0.2bar/m was maintained throughout the test which is equivalent to 20m head.
- Water application in to the test section constantly.
- Recording of the total intake and the time taken

The testing unit packers were inflated to the working pressure to ensure a proper seal giving allowances over counter acting hydrostatic pressure in the borehole (applied plus column head). The coefficient of permeability k is then calculated as:

$$K = q/FH_c$$

$$F = \frac{2\pi L}{\log_e[(\frac{L}{D}) + \sqrt{\{1+(\frac{L}{D})^2\}}]}$$

Where:

- K = coefficient of permeability of soil
- q = intake rate
- H_c = Head
- F = intake factor

Table 4.6:Summary Constant head permeability test result

Borehole ID	Test No.	Test section (m)		Test section length	Coefficient of permeability
		From	to	L (cm)	k (cm/sec)
DG-35	1	15	20.45	545	3.06×10^{-06}
	2	20	25.45	545	2.85×10^{-06}
	3	25	30	500	1.37×10^{-06}
DG-36	1	6	11	5	1.36×10^{-06}

Table 4.7: Summary of executed activities in boreholes

No.	**Activities in Borehole**	**Unit**	**Borehole ID No.**			**Total Tasks**
			GI/II-BH-1	DG-35	DG-36	
1	Constant head Test	No.	-	3	1	4
2	SPT Test	No.	-	5	3	8
3	Rock core Sample	No.	2	1	1	4
4	Undisturbed Shelby sample	No.	-	1	-	1
5	Core Soil Sample	No.	-	2	1	3
6	SPT spoon soil sample	No.	-	7	3	7
7	Drilled Depth	m	35	30	11	76
8	Core photo	No.	6	5	2	13
9	LithologicLogging	m	35	30	11	76

4.4 Test Pit Excavation

Test pits were excavated manually at provided location of dam foundation & dam impervious zone material potential areas. A total of 6 test pits, with a maximum of 5 meters depth, were excavated.

Excavations were inspected closely and actual information recorded on proper test pit log formats. Required quantity of representative both undisturbed and disturbed samples were collected from pit and trench walls and tested in the laboratory for determination of purpose based index and engineering properties of soil samples. Every excavation had been backfilled properly after completion of the activities planned.

Table 4.8:List of test pit excavated at dam foundation and impervious material sites

Test Pit ID	Location	Coordinate		Excavated Depth (m)	Remark
		Easting	Northing		
GTP-1	Foundation	459687	1003761	5.0	
GTP-2	“	460027	1003730	4.0	
GCBTP-1	Clay borrow	460040	1003811	2.0	
GCBTP-2	“	460191	1003787	2.0	
GCBTP-3	“	460167	1003798	2.0	
GCBTP-4	“	460133	1003845	2.0	

4.5 Sampling and Laboratory Study

Representative rock core, undisturbed and disturbed bulk soil samples were collected from core drilling recovery, and excavated test pit/trench walls respectively. The required laboratory tests of such sample were analyzed based on corresponding ASTM and BS standards.

Table 4.9:List of soil samples collected from test pits

S. No	Sample ID	Site	Sample Depth (m)	Sample Type
1.	**GTP-1-1**	Foundation	2.40 - 2.50	Undisturbed
2.	**GTP-1-2**	“	4.0 – 4.10	“
3.	**GTP-2-1**	“	3.0 – 3.10	“
4.	**GTP-2-2**	“	1.30 – 1.40	“
5.	**GTP-1**	“	2.10 – 4.10	Bulk Disturbed
6.	**GTP-2**	“	0.80 – 2.50	“
7.	**GCBTP-1**	Clay borrow	0.50 – 1.50	“
8.	**GCBTP-2**	“	Surface Outcrop	“
9.	**GCBTP-3**	“	“	“
10.	**GCBTP-4**	“	“	“

Table 4.10:List of soil samples collected from boreholes

1.	**DG35-SS-1**	Dam axis	2.0 – 2.56	Small Disturbed
2.	**DG35-SS-2**	“	6.40 – 7.0	“
3.	**DG35-SPT-1**	“	17.65 – 18.10	SPT Sample
4.	**DG35-SPT-2**	“	20.0 – 20.45	“
5.	**DG35-SPT-3**	“	22.55 – 23.0	“
6.	**DG35-SPT-4**	“	24.50 – 24.95	“
7.	**DG35-UD-1**	“	21.95 – 22.55	UD Sample
8.	**DG36-SPT-1**	“	1.50 – 1.95	SPT Sample
9.	**DG36-SPT-2**	Downstream	3.45 – 3.90	“
10.	**DG36-SPT-3**	“	10.20 – 10.65	“
11.	**DG36-SS-1**	“	5.55 – 6.0	Small Disturbed

Table 4.11:List of rock samples collected from proposed quarry site

1.	**HRS-1**	Boost - 1	Rock	Surface Outcrop
2.	**ACRS-1**	Rock Quarry	"	"
3.	**ACRS-1**		"	"

4.5.1 Tests on Rock samples

The intact rock samples were studied in the laboratory for determination of index and engineering properties.

Index and engineering properties of intact rock samples were studied at such tests performed to determine: specific gravity, water absorption, bulk unit weight and strength (Uniaxial Compression and Point load) of rock specimen. The details of collected rock core samples, the type of laboratory tests conducted, and corresponding laboratory test results tabulated summary are presented below.

Table 4.12: Laboratory test result of intact rock samples from Intake site

Borehole No.	Depth(m)	Specific gravity	UCS(Mpa)	Porosity (%)
BH-1	11.46 – 11.66	2.57	4.67	6.23
	25.66 – 26.0	2.44	12.70	3.69

4.5.2 Tests on Soil Samples

The soil samples of current investigation were collected from walls of test pits and trenches excavated at provided material source areas for dam impervious core zone and embankment

The laboratory study of collected samples included tests required for the soil classification and engineering properties characterization as to the construction purpose required. Such tests performed had embodied: grain size analysis (sieve and hydrometer), Atterberg limits, bulk unit weight, specific gravity, natural moisture content, Linear and volumetric shrinkage, free swell, standard and modified compaction/proctor, shear strength (direct shear), consolidation (odometer), unconfined compression, dispersion (double hydrometer), permeability and chemical tests (organic content, Cl^- and $S0_4^-$). All tests were carried out in the Research, Laboratory and Training center of Ethiopian Construction Design and Supervision Works Corporation.

Table 4.13: Summary of Laboratory test results samples collected from clay borrow

Parameters	GTP-1	GTP-2	GCBTP-1	GCBTP-2	GCBTP-3	GCBTP-4
	2.10 – 4.10m	0.80 – 2.50m	0.50 – 1.50	Surface outcrop	Surface outcrop	Surface outcrop
	Lab No:503/09	Lab No: 504/09	Lab No: 505/09	Lab No: 506/09	Lab No: 507/09	Lab No: 508/09
Grain size Analysis						
Clay (%)	48.08	32.42	44.66	50.03	57.07	56.52
Silt (%)	35.54	61.71	41.82	48.45	39.59	41.74
Sand (%)	16.38	5.87	13.52	1.52	3.34	1.75
Gravel (%)	-		-	-	-	-
Atterberg Limits						
Liquid Limit (%)	61.15	53.40	51.36	50.90	57.0	56.70
Plastic Limit (%)	37.16	32.60	35.99	33.24	34.98	37.62
Plasticity Index (%)	23.99	20.80	15.77	17.66	22.02	19.08
Free Swell %	50.0	49.0	47.5	37.5	35.0	42.5
Specific gravity	2.73	2.63	2.74	2.71	2.84	2.61
Linear Shrinkage, %	10.89	11.6	9.96	12.5		13.11
NMC	49.72	30.30	-	-	-	-
Consolidation,cc	0.200	0.220	-	-	-	-
Compaction						
MDD(gm/cc)	1.179	1.308	1.361	1.398	1.524	1.357
OMC (%)	44.70	34.0	33.42	32.85	25.70	35.20
Dispersion	ND	ND	ND	ND	ND	ND
Cl^-(meq/lit)	1.19	1.32	1.06	1.19	1.06	1.06
SO_4(meq/lit)	3.47	2.68	7.04	2.60	5.67	1.93
Direct shear						
C(kPa)	43.0	74.33	77.0	53.0	35.67	32.0
Ø (^{0})	10.76	19.55	13.77	9.09	13.77	11.86
Triaxial (UU)						
C(kPa)	26.27	52.06	65.50	63.34	43.34	31.66

Ø (^{0})	14.09	13.08	17.71	13.03	17.59	12.76
Organic Content (%)	0.16	0.30	0.54	0.17	0.10	0.22
Permeability(cm/s)	$4.52x10^{-7}$	$1.91x10^{-6}$	$4.42x10^{-7}$	$3.27x10^{-8}$	$1.69x10^{-6}$	$2.86x10^{-8}$

Table 4.14: Summary of laboratory test results of soil samples

BH ID	**Sample ID**	**Sample Type**	**Sample Depth(m)**	**Grain Size Analysis**			**Atterberg limits**			**Bulk unit weight**	**Sp. gr.**	**Free Swell**	**Consolidation**	**UCS, Kpa**	**Direct shear**		Permeability
				Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	P.I (%)						**C**	**Ø**	
DG35	DG35-SS-1	Disturbed	2.0 – 2.56	2.9	52.45	41.65	55.15	23.02	30.13	1.98	2.8	62.5	-	-	-	-	-
	DG35-SS-2		6.40 – 7.0	29.42	59.13	11.45	52.8	37.35	15.45	1.59	2.88	42.5	-	-	-	-	-
	DG35-SPT-1	Undisturbed	21.95–22.55	1.98	67.52	30.5	57.2	34.02	23.18	1.65	2.58	46.71	0.22	46.71	-	-	-
	DG35-SPT-2	SPT	17.65–18.10	82.82	17.18	-	34.62	NP	-	2.44	2.86	-	-	-	38.7	18	
	DG35-SPT-3		20.0 – 20.45	55.88	42.21	1.91	54.6	36.56	18.04	1.75	2.86	-	-	-	-	-	-
	DG35-SPT-4		22.55 – 23.0	17.54	79.57	2.89	-	-	-	1.81	2.82	-	-	-	-	-	-
	DG35-UD-1		24.50–24.95	4.48	35.46	60.06	66.8	45.02	21.78	1.35	2.72	-	-	-	-	-	-
DG36	DG36-SPT-1	Disturbed	5.5 – 6.0	4.92	43.32	51.76	63.48	34.65	28.83	-	2.67	57.5	-	-	-	-	-
	DG36-SPT-2	SPT	1.50 – 1.95	10	65.59	24.41	51.25	30.32	20.93	1.86	2.76	42.5	-	-	-	-	-
	DG36-SPT-3		3.45 – 3.90	6.15	28.51	65.34	76.15	38.43	37.72	1.77	2.7	72.5	-	-	-	-	-
	DG36-SS-1		10.20–10.65	3.4	70.72	25.88	64.3	37.92	26.38	1.67	2.67	57.5	-	-	-	-	-
GTP-1	GTP-1-1	Undisturbed	2.40 - 2.50	16.38	35.44	48.08	61.15	37.16	23.99	-	2.73	10.89	0.2	45.75	-	-	$1.19x10^{-6}$

	GTP-1-2		4.0 - 4.10	-	-	-	-	-	-	-	-	-	0.345	-	53	23.27	8.5x10^{-7}
GTP-2	GTP-2-1	Undisturbed	3.0 - 3.10	5.87	61.71	32.42	53.4	32.6	20.8	-	2.63	49	0.175	258.9	-	-	5.14x10^{-7}
	GTP-2-2		1.30 - 1.40	-	-	-	-	-	-	-	-	-	0.22	-	42.7	21.55	2.37x10^{-6}

5 ENGINEERING CHARACTERIZATION

5.5 Gefersa I & II Intake Structure Foundation Site

5.5.1 Foundation Rock Mass Characterization

On the basis of local geological setting and investigation revealed from borehole drilled at Gefersa I/II site and extrapolation of the previous drilled bore holes the proposed intake structure foundation site lies on rhyolite/andesite rock mass underlying shallow overburden soil mass. GI/II BH-1 is located some 20m upstream of the dam with reference to the proposed intake structure location. The Rock mass characterization made for the dam site is applicable to the intake foundation in conjunction with produced geologic cross-sections and local geological setting mapped. Further site specific intake foundation characterization requires planning and implementation of core drilling at the proposed site.

Intact rock properties are derived and characterized based on laboratory studies of samples collected from field work. The rock mass is characterized using the Geomechanics rock mass rating parameters which considers the intact rock property and effect of discontinuities, degree of weathering and the groundwater condition. Such properties of the rock mass were obtained from measurement and examination of drilling recovered core samples.

Plate 5.1: Photo showing the intensively fractured rock mass at the intake foundation

5.5.2 Foundation Rock Mass Classification

Geomechanics Classification (Rock Mass Rating System) was used for classifying and interpreting the quality of the foundation rocks of the proposed site.

The following six parameters were used to classify the foundation rock mass using RMR system; they were collected in the field and from laboratory test results.

1. UCS of rock material
2. Rock Quality Designation (RQD)
3. Spacing of discontinuities
4. Condition of discontinuities
5. Ground water condition
6. Orientation of discontinuities

The intake foundation rock mass falls in one structural region, Based on geological, fracture density and evenness as displayed by weighted RQD values. This region is assumed to be more or less uniform in certain geotechnical features.

Accordingly, Bieniawski classification method was employed for the foundation rock mass, the results are given in the table below.

Table 5.1: Rock Mass Classification, based on Geomechanics Classification Method.

Item	**Rock type**	**UCS (Mpa)**	**RQD (%)**	**Spacing of Discontinuities (mm)**	**Condition of Discontinuities**	**Ground Water**	**Total**	**Rock mass Class Number**
Zone-1 Ratings	Rhyholite/ Andesite	4.67	27	45mm	<1mm separation, Slightly rough surfaces	wet	34	IV
Zone-1 Ratings	“	12.70	35	30mm	<1mm separation, Slightly rough surfaces	wet	36	IV

Therefore, the rock mass at Gefersa dam I/II fall on Rock Mass Class IV, which is poor rock. Laboratory test of rock samples revealed that intact rock nature possesses specific gravity of 2.44 to 2.57, and UCS ranging from 4.67 – 12.70Mpa. As can be inferred from Table 5.3 below, the rock mass at dam site has poor quality, with estimated cohesion between 170 to 180kPa, angle of internal friction ranging from 22 to 23 degrees, and modulus of deformation in the range of 3 to 4.5GPa.

Table 5.2: Laboratory test result of intact rock samples from Intake site

Borehole No.	Depth(m)	Specific gravity	UCS(Mpa)	Porosity (%)
BH-1	11.46 – 11.66	2.57	4.67	6.23
	25.66 – 26.0	2.44	12.70	3.69

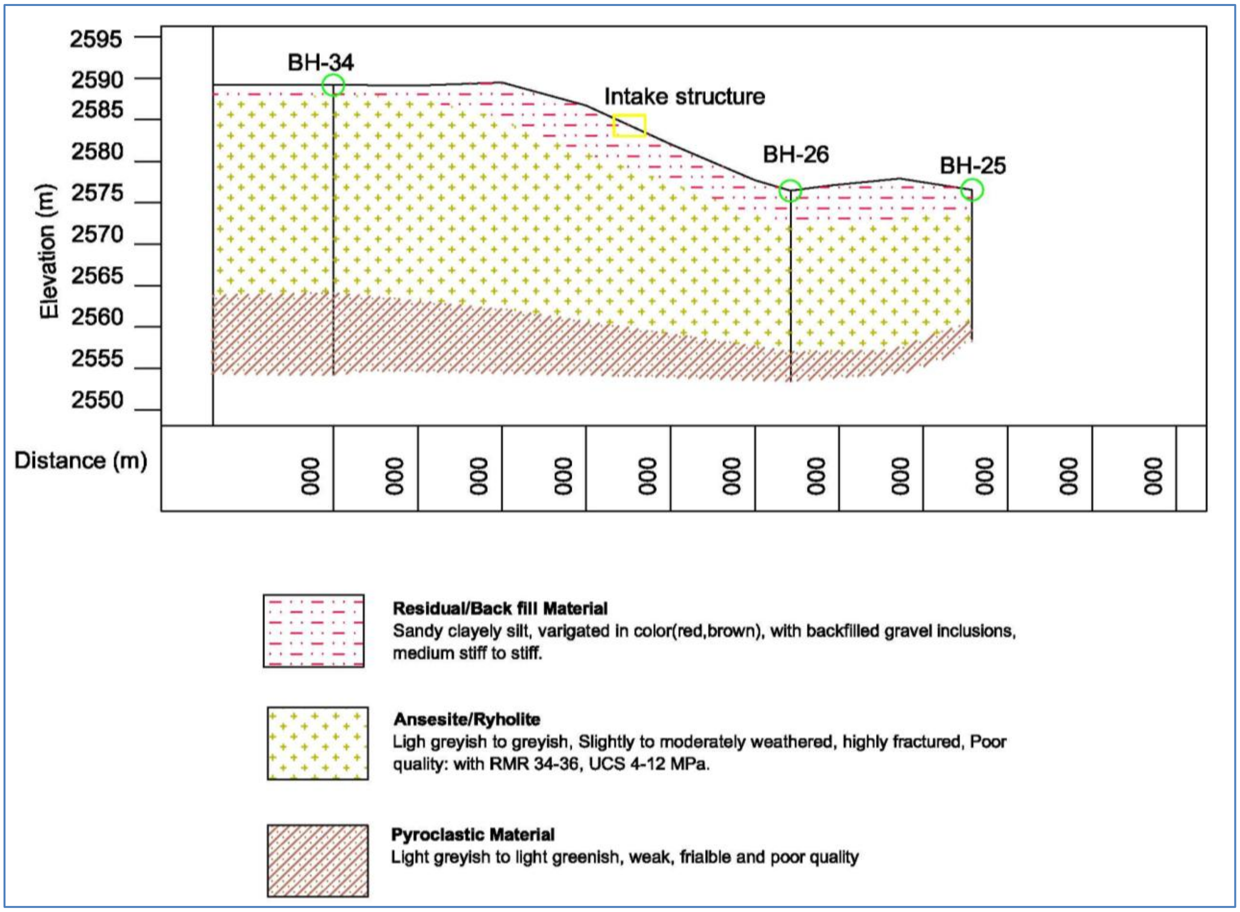

Figure 5.1: Engineering Geological Cross-section at the Intake Structure site

Table 5.3: Summary of dam site rock mass properties

BH-ID	Depth	Intact Rock Strength		Rock Mass Class Properties					
		UCS	Point load	RMR	Class	Quality	C	Ø	E_d
BH-1	11.46 – 11.66	4.67	-	34	IV	Poor	1.7	22	3.98
BH-1	25.66 – 26.0	12.71	-	36	IV	Poor	1.8	23	4.46

Where, C = Cohesion

Ø = Angle of Internal friction and

E_d= Modulus of Deformation

Table 5.4: Correlation of RMR with rock mass shear strength Parameters (adopted from Bieniawski 1989)

RMR	100 - 81	80 - 61	60 - 41	40 - 21	<20
Rock class	I	II	III	IV	V
Cohesion, Kpa	>400	400 – 300	300 - 200	200 - 1000	<100
Ø in degrees	>45	45 - 35	35 - 25	25 - 15	<15

5.5.3 Bearing Capacity Analysis

The foundation site is characterized by light grey to creamy in color, moderately to highly weathered, intensively fractured, weak Rhyolite. Rock strength tests were done to characterize the strength of the rock mass.

For estimation of the safe load bearing capacity, the rock has been evaluated in accordance with the Hoek-Brown Criterion, 2011, using minimum values measured in situ and Roc-Lab software developed for this purpose by Rock Science. The criterion starts from the properties of intact rock and then introduces factors to reduce those properties on the basis of the characteristics of the rock type, jointing and weathering; and then estimate acceptable equivalent frictional angle and cohesive strength for a given rock mass. The criterion correlates the parameters as:

Sigci = Intact Uniaxial Compressive Strength

GSI = Geological Strength Index

mi =Material constant

D = Disturbance factor

Ei = Intact modules

c = Cohesion

Ø = Frictional angle

The rock mass constituting the foundation area can be described based on UCS results from laboratory and field outcrop observation as weak, closely spaced jointed and slightly weathered.

Sigci = 8.68, can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer

GSI = 56, inter locked, partially disturbed mass, with multi-faceted angular blocks formed by 4 or more joint sets

Mi = 25, for Rhyolite

D = 0

Ei = Mrx UCS, this relationship is used when no direct values of the intact modules are available or where completely undisturbed sampling for measurement of Ei is not available.

The criterion estimates Mr. for Rhyolite as 400. Then; Ei = 3468MPa. The Hoek - Brown Criterion results: Mohr-Coulomb Fit

c = 0.617Mpa

Ø = 40.25 degrees

Rock Mass Parameters: σ'_{cm}= 2.66MPa (see figure below), which is the global rock mass strength of the rock mass, evaluated from the following formula using the inputs (Ø and c).

$$\sigma'\mathrm{cm} = \frac{2c'\cos\text{Ø}'}{1 - \sin\text{Ø}'}$$

Hence taking the global rock mass strength of 2.66 and considering a safety factor of 3 a load bearing capacity of 0.886Mpa (88.66Kpa) can be taken as the safe bearing capacity for the foundation rock of the rhyolite layer found at Gefersa dam I/II intake structure site.

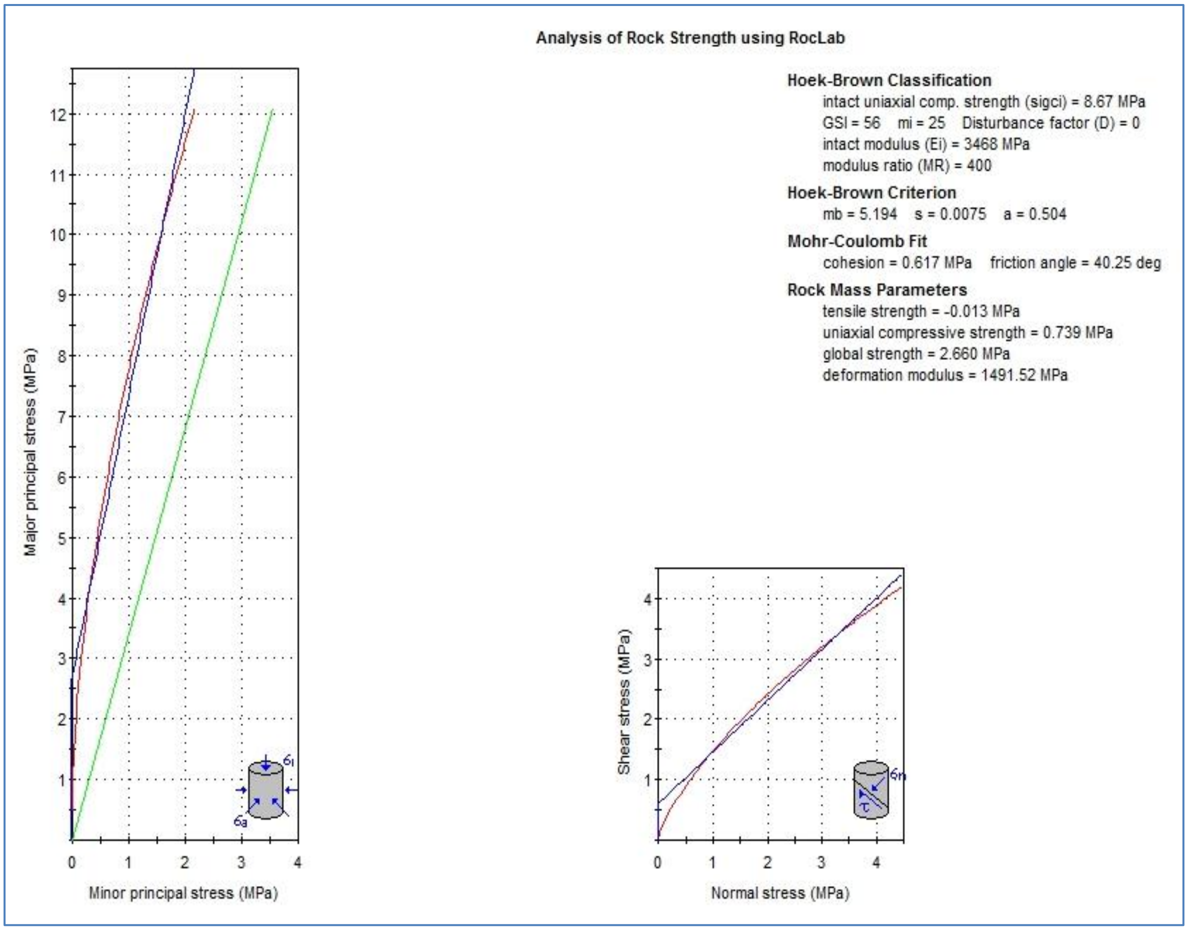

Figure 5.2: Load Bearing Capacity Analysis Result of the Rhyolite

5.6 Gefersa III Dam

5.6.1 Foundation Soil Characterization

The residual soil at Gefersa III Dam site is formed from prolonged time of weathering and decomposition of the underlying parent rock. Interpolation of geological information between executed geotechnical core drilling boreholes revealed considerable thickness variation of the soil mass within project area; where there is up to 24 meters thick soil cover on the left abutment. The upper most portion of the residual soil (0-8m) is rich in organic matter and plant roots forming a dark brown top soil layer. Underlying the top soil horizon is the residual soil mass characterized by variegated colors of brownish, red, whitish, and yellowish. The grading of the soil generally dominated with cohesive soil mass with sand (silty clay, clayey silt and sandy silt), with medium to high plasticity.

All the soil samples collected from boreholes and test pits are classified as MH/OH which is inorganic silts of high plasticity/ organic silts of high plasticity.

The soil mass possess a bulk unit weight in the range of 1.35to 2.24 mg/cc. Standard penetration test results on the residual soil deposit has revealed medium stiff to stiff consistency (N60 ranging 6 to 35) in its natural state occurrence on most portion. The elastic modulus of the residual soil mass ranges between 3600 to 12300 Kpa as estimated from SPT data on the basis of relation Es = 300 (N+6) , after Bowles (1988).

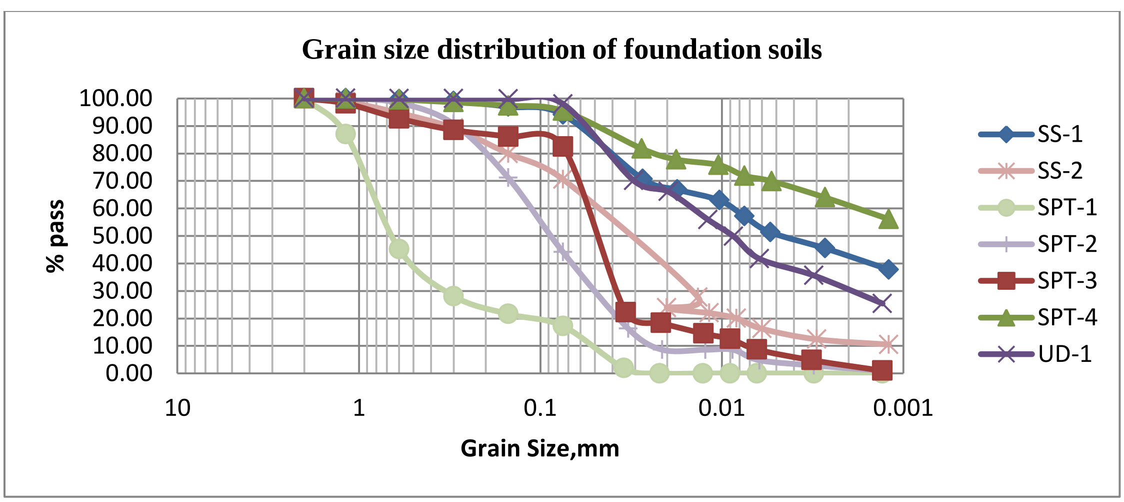

Figure 5.3: Grain size distribution curve of foundation soil samples

Table 5.5: Summarized engineering property of soil samples collected from foundation site

Sr. No.	Sample ID	Sample Depth (m)	Soil Gradation	LL	PI	Clay content,%	Soil Group	Activity	Plasticity	Compre ssibility	Degree of Expansion
1	DG35-SS-1	2.0 – 2.56	Clayey SILT	55.15	30.13	41.65	CH	Inactive	High	High	High
2	DG35-SS-2	6.40 – 7.0	Clayey sandy SILT	52.8	15.45	11.45	MH	Normal	Medium	High	Low
3	DG35-SPT-1	21.95– 22.55	Clayey SILT	57.2	23.18	30.5	MH	Normal	High	High	Medium
4	DG35-SPT-2	17.65– 18.10	Silty SAND	34.62	-	-	MH			Low	
5	DG35-SPT-3	20.0 – 20.45	Silty SAND	54.6	18.04	1.91	MH	Active	High	High	Low
6	DG35-SPT-4	22.55 – 23.0	Sandy SILT	-	-	2.89	MH				
7	DG35-UD-1	24.50– 24.95	Silty CLAY withsand	66.8	21.78	60.06	MH	Inactive	High	High	Medium
8	DG36-SPT-1	5.5 – 6.0	Silty CLAY withsand	63.48	28.83	51.76	MH	Inactive	High	High	Medium
9	DG36-SPT-2	1.50 – 1.95	Sandy clayey SILT	51.25	20.93	24.41	MH	Normal	High	High	Medium
10	DG36-SPT-3	3.45 – 3.90	Silty CLAY withsand	76.15	37.72	65.34	MH	Inactive	High	High	High
11	DG36-SS-1	10.20– 10.65	Clayey SILT	64.3	26.38	25.88	MH	Normal	High	High	Medium
12	GTP-1	2.10-4.10	Sandy silty CLAY	61.15	23.99	48.08	MH	Inactive	High	High	Medium
13	GTP-2	0.80-2.50	Clayey SILT with sand	53.4	20.8	32.42	MH	Inactive	High	High	Medium
14	GTP-1-1	2.40 - 2.50	Sandy silty CLAY	61.15	23.99	48.08	MH	Inactive	High	High	Medium
15	GTP-1-2	4.0 - 4.10		-	-	-					
16	GTP-2-1	3.0 - 3.10	Clayey SILT with sand	53.4	20.8	32.42	MH	Inactive	High	High	Medium
17	GTP-2-2	1.30 - 1.40		-	-	-					

Plate 5.2: Photo showing residual soil at dam site

5.6.2 Bearing capacity analysis

The allowable bearing pressure is the maximum net intensity of loading that can be placed on the soil without any shear failure or the risk of excessive settlement. It is therefore, the smaller of the net safe bearing capacity (shear failure criterion) and the safe bearing pressure (settlement criterion) that has to be considered. Consequently, taking in to account the material properties of the project sites, the foundation ground need to be analyzed taking or considering their strength and engineering characteristics.

A uniform settlement is usually of little consequence in buildings, but a differential settlement can cause severe structural damages.EBCS-7 (1995) recommends permissible total settlements of 50mm and 75mm on sandy and clayey soils, respectively. Differential settlements between adjacent columns up to 20mm are acceptable. In calculation of the allowable bearing capacities of the soils both the total and differential settlement values have estimated to be within tolerable limits (25mm). Therefore, the calculated allowable bearing pressures are including the settlement criteria.

The determination of allowable bearing pressures is discussed based on the strength of the sub-surface formations from in situ test results. Field observation and measurements on different soil layers are the main data used in the analysis of the allowable bearing pressures of the foundations.

After adjusting the N-values based on the above formula, the design N-values are used to determine the allowable bearing capacities of soil layers at each foundation site.

Table 5.6: The allowable bearing capacity at the dam foundation of Geferesa Dam III

BH ID	**Layer No.**	**Depth (m)**		**N**	**N'**	**SPT N' Equivalent UCS, KPa**	**SPT N' Equivalent Cu=UCS/2, KPa**
		from	**to**				
DG-35	1	17.65	18.10	27	21	189	95
		20.00	20.45	15	12	108	54
		22.55	23.00	31	24	216	108
		28.50	28.95	45	35	315	158
	Mean		**From SPT N'-Value (Layer - 1)**			**207**	**104**

5.6.3 Foundation Permeability

The water tightness condition assessment of the dam foundation requires understanding of the permeability of the rock and soil mass composing the dam foundation and abutments, the mechanism of interconnection of reservoir water to leakage path, and the tolerable limit of seepage in design.

In-situ falling head and constant head tests were carried out at proposed dam site. The degree of permeability of the foundation and abutments is characterized adopting classification of degree of permeability suggested by Bell F.G. (2007) for soil mass.

Table 5.7: Degree of Permeability of soils (Bell F.G., 2007)

Coefficient of permeability	Degree of Permeability	USBR Classification Rating
Greater than 10^{-0}	Very High	Pervious
10^{-2} to 10^{-0}	High	

10^{-4} to 10^{-2}	Medium	
10^{-6} to 10^{-4}	Low	Semi-pervious
10^{-8} to 10^{-6}	Very low	Impervious
Less than 10^{-8}	Impermeable	

Analysis of the test results generally revealed that the residual soil mass at the weir foundation is characterized by very low permeability to impervious nature. The residual soil at dam site forms semi-pervious to impervious foundation whereas the rock mass forms a pervious foundation in general. The presence of inter-site faulting and exposure of the fractured bed rock along river channel is anticipated to form a potential connection of reservoir water to the pervious foundation and abutment zones under the dam. Furthermore, a weak zone on left abutment (likely be a geologic structure) could form a potential leakage zone at the reservoir area.

5.7 Reservoir Water Tightness

5.7.1 Stability Evaluation

The Gefersa III dam site reservoir area has slightly undulating morphological setting. The reservoir area generally is dominated by rolling and slightly sloping to flat land topography. The area is intersected by River and its tributaries. Rocks are observed on exposures at river and stream cuts and rarely on slightly sloping areas near rivers. Almost the entire reservoir area is covered by about 1-3 m thick residual and alluvial soil of sandy silty clay and sandy clayey silt composition.

The majority of the reservoir area is well covered with grass and utilized for subsistence farming and grazing land for cattle by inhabitants of the area. With regard to stability of Gefersa III dam site reservoir area, no scars and any other instability problems are observed except minor erosion of soil at the sides of the rivers and their tributary streams.

5.3.2 Seepage Analysis

Geotechnical investigation has been conducted to assess the seepage/ leakage condition of the reservoir areas of Gefersa III dam. The investigation included auger hole excavation for visual observation and sampling of the soil strata, in situ permeability testing were conducted at the excavated auger holes

From observation at test pits and other natural exposures the soil blanket of the reservoir area has thickness ranging between 1m to more than 3 m except at specific localities such as river and stream cuts where massive weathered tuff and basaltic rock has been exposed on surface.

The in situ falling head tests show that the silty clay soil has average infiltration rate ranging from 1.34 x $10^{-4}$to 7.45 x 10^{-5}cm/s. Thus it is concluded from the field investigation and laboratory test results that the reservoir area of Gefersa Dam III is covered by sandy clayey silt/ sandy silty clay soil blanket of low permeability.


Plate 5.3: In situ infiltration tests in the reservoir area of the project area

5.8 Construction Material Assessment

Construction materials suitable for construction of the proposed structures have been investigated in detail. The investigation was performed for the purpose of engineering characterization, sampling and volume estimation of the materials suitable for construction of the different structures included in the project. The construction materials assessed and investigated included:

- Impervious clay for dam clay core, and
- Rock quarry for coarse aggregate and rock fill,

5.8.1 Impervious Clay for Dam Clay Core

Reddish brown lateritic soil is abundant in and around the project area. The soil is observed covering parent rock material around the top and sides of hills. The thickness of soil layer varies from 1.5 m to 3 m on average. Three sites were identified as potential clay borrows to be used for core material. To characterize the physical and engineering properties of the lateritic soil deposits selected for core materials 5 samples were collected and sent to the central laboratory. The reddish brown lateritic soil layer is underlain by yellowish brown transitional soil layer and the parent rock material underneath is exposed at places.

Plate 5.4: Photo showing potential clay borrow area

5.8.2 Characterization of clay core material

Permeability

Laboratory permeability test was done on remolded soil samples and a very low permeability value of about 1.91 x 10^{-6} to 2.86 x 10^{-8} cm/sec was obtained. The permeability values indicate that all the core materials are found to be impervious.

Grain size Analysis

A total of 6 samples were collected and analyzed for evaluating the quality of soils which could be used as impervious core material. Two types of impervious core materials were identified and investigated for clay core: the dark brown clay (black cotton soil) and light brown/reddish clay.

Based on the laboratory test results the light brown impervious material has 32.42 to 57.07 % clay, 35.54–61.71 % silt and 1.75–16.38 % sand.

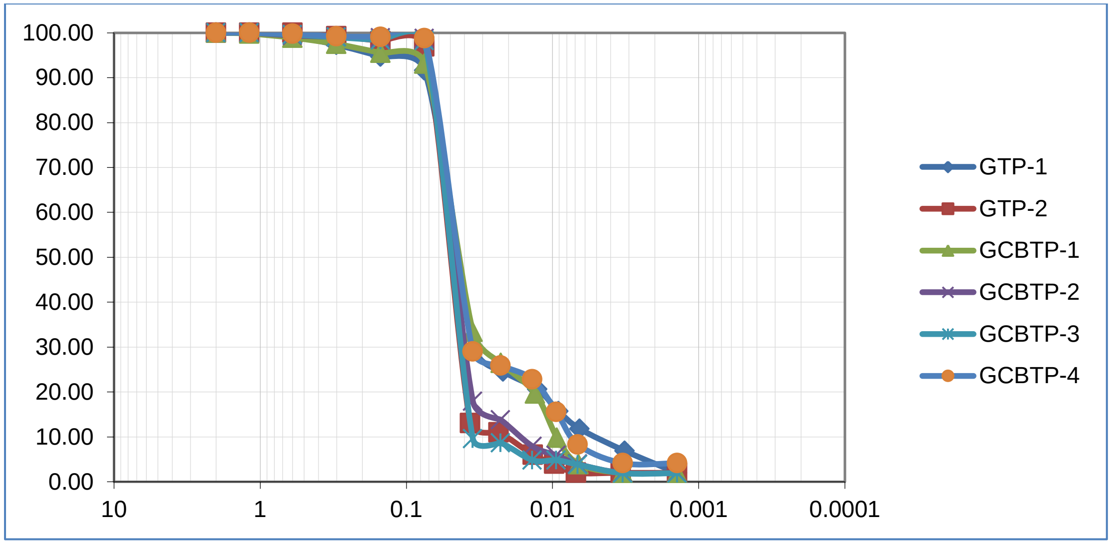

Figure 5.4: Grain size Distribution Curve of Impervious Clay Material

Atterberg Limit

According to the standard set by Murthy (2002) values obtained for the selected samples provided plasticity index values higher than 17, except GCBTP-1, which imply that the soil material is highly plastic.

Plasticity index = 0 % - non-plastic

< 7 % - low plastic

7-17 % - medium plastic

> 17 % - high *(source: Murthy, 2002)*

Generally the results show that most of the light brown soils falls under highly plastic ranges while the light brown soils are classified as Medium to Highly Plastic. According to USBR (1991), the tested soil samples falls MH-OH, which is inorganic silt of high compressibility.

Compressibility decreases with decreasing liquid limit. The higher the liquid limit of clay, the more compressible it will be during compaction. Accordingly, all soil samples collected from core material potential sites have marginally high to high compressibility with liquid limits of greater than 50 %.

Liquid limit < 35% - low compressibility

35 - 50 % – medium compressibility

>50 - high compressibility (Source: Purushothama, 2008)

Swelling Potential

The swelling potential of the soil samples can be calculated according to empirical formula of Anderson (1981) in which the calculated swelling potential(S) is correlated with the degree of expansion.

$$S = 0.23PI - 3.12$$

Where, S= Swelling Potential,

PI = Plasticity Index

Table 5.8: Relationships between Swelling Potential and Degree of Expansion (Anderson, 1981)

S. No	**Degree of expansion**	**Plasticity Index (PI)**	**Swelling Potential (S)**
1	Low	20	1.5
2	Medium	20 – 31	1.5 – 4.0
3	High	31 – 39	4.0 – 6.0
4	Very High	>39	> 6.0

Accordingly the dark brown clay has a high to very high degree of expansion, while the reddish brown clay has a Medium to High degree of expansion.

Erodibility

Samples collected from impervious clay core borrow areas were undergone for erodibility test with pin hole and double hydrometer methods and results shows that all the materials are non-dispersive (ND).

Direct Shear

Both cohesion (c) and angle of internal friction (ø) contributes to the shear strength of soils. Understanding shear strength is the basis to analyze soil stability problems such as lateral pressure on earth retaining structures, slope stability and bearing capacity

Direct shear test has been conducted on representative samples as shown in the table below. The result showed that cohesion of the soil samples ranges between 15 to 66kpa and internal friction angle varies from 11 to 26 degrees and hence has fairly good resistance to shearing forces

Organic Content

The presence of significant quantity of organic matter has effect on the strength and compressibility of soils. The soils with organic matter are weaker and more compressible than soils having the same mineral composition but lacking in organic matter.

The organic content of soil samples was evaluated and the value ranges between 0.16 to 0.54 %. It is insignificant and has no effect on the strength and compressibility of the soils of the project area.

Table 5.9: Summarized engineering property of soil samples collected from clay borrow

Sr. No.	Sample ID	Sample Depth (m)	Soil Gradation	LL	PI	Clay content ,%	Soil Group	Activity	Plasticity	Compressi bility	Degree of Expansion	Permeabil ity, cm/s	Degree of permeability
1	GCBTP -1	0.50-1.50	sandy silty CLAY/ sandy Clayey SILT	51.36	15.77	44.66	MH	Inactive	Moderate	High	Low	4.42×10^{-7}	Impervious
2	GCBTP	surface	Silty CLAY	50.9	17.66	50.03	MH	Inactive	High	High	Low	3.27×10^{-8}	Impervious

	-2												
3	GCBTP -3	surface	Silty CLAY	57	22.02	57.07	MH	Inactive	High	High	Medium	$1.69x10^{-6}$	Marginally impervious
4	GCBTP -4	surface	Silty CLAY	56.7	19.08	56.52	MH	Inactive	High	High	Low	$2.86x10^{-8}$	impervious

5.8.3 Rock Quarry Rip Rap and Masonry

Rock appropriate for masonry and riprap was assessed and located at closer proximity from the dam site and the investigation was based on the following three main criteria;

1. A source which can produce hard, dense and durable to withstand destructive forces during placing, wave action, weathering, servicing, etc.
2. Nearby site to reduce haulage distance.

Ignimbrite rock is abundant in the project area. The widely jointed, moderately to slightly weathered ignimbrite rock is the only material used for rock production for rip rap. Sites selected for rock quarry for the production crushed coarse aggregates are used for the production of rip rap for the proposed dams.

Plate 5.5: Proposed ignimbrite rock quarry for rip rap and masonry structures

Laboratory tests conducted on ignimbrite samples show that the rock is suitable as masonry stones as well as rip rap. The moderately weathered ignimbrite rock has point load value of 1.45-1.76Mpa and low water absorption values.

Table 5.10: Summary of laboratory test results of the sand and basalt quarry materials

Parameters	**GRS-1 (Rock)**	**GRS-2 (Rock)**
	Lab No:515/09	Lab No:516/09
Point Load (Mpa)	1.45	1.76
Specific gravity	2.58	2.50
Porosity, %	17.44	12.80
Water Absorption, %	9.0	9.1

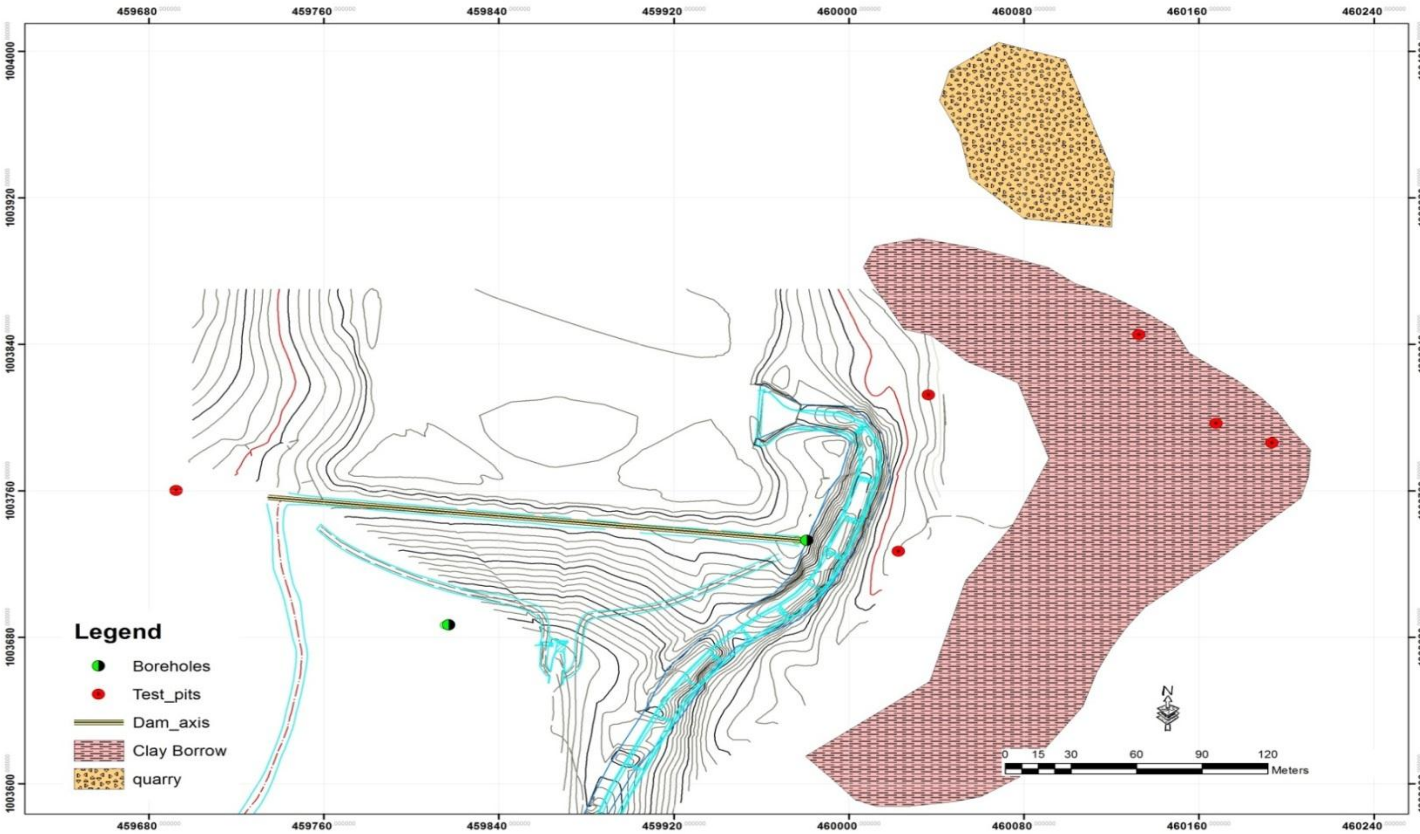

Figure 5.5: Location map of Proposed Construction Material Potential sites

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusion

- The project area is located North West of Addis Ababa at Gefersa I/II and Gefersa III Dam sites. The project area in general has a rugged topography. Gefersa River and its tributaries intersect the area adding in the ruggedness of the project areas.
- The geology of Gefersa I/II intake structure site is composed of andesite/rhyolite overlain by thin residual/ back filled soil cover. The rock is light grey to creamy in color, moderately to highly weathered and intensively fractured. The rock is about 20 m thick and is underlain by pyroclastic material.
- The geology of Gefersa III dam site is composed of thick residual soil deposit formed from prolonged time of weathering and decomposition of the underlying parent rock. The thickness of the soil reaches up to 24 m on the left abutment and the top about 8 m thick is rich in organic content. Underlying the top soil horizon with high organic content is the residual soil mass characterized by variegated colors of brownish, red, whitish, and yellowish. The grading of the soil generally is dominated by cohesive soil mass with sand (silty clay, clayey silt and sandy silt) classified as MH/OH according to USCS.
- The reservoir area of Gefersa III dam generally is dominated by rolling and slightly sloping to flat land topography. Rocks are observed on exposures at river and stream cuts and rarely on slightly sloping areas near rivers. Almost the entire reservoir area is covered by about 1-3 m thick residual and alluvial soil of sandy silty clay and sandy clayey silt composition classified as MH according to USCS.
- Data obtained by interpolation of the previous boreholes to the current drilled borehole the andesite/rhyolite rock at the intake structure (Gefersa I/II) site has experienced moderate to high weathering and intensive fracturing. Laboratory test of rock samples revealed that the intact rock nature possesses specific gravity of 2.44 to 2.57, and UCS ranging from 4.67 – 12.70Mpa. The rock mass at the intake site has poor quality (with RMR in the range of 34-36%), estimated cohesion between 170 to 180 Kpa, angle of internal friction ranging from 22 to 23 degrees, and modulus of deformation in the range of 3 to 4.5GPa.

- Taking the global rock mass strength of 2.66 and considering a safety factor of 3 a load bearing capacity of 0.886 Mpa (88.66 Kpa) can be taken as the safe bearing capacity for the foundation rock of the rhyolite layer found at Gefersa dam I/II intake structure site.
- The soil mass at Gefersa III dam site possess a bulk unit weight in the range of 1.35to 2.24 gm/cc. Standard penetration test results on the residual soil deposit has revealed medium stiff to stiff consistency (N60 ranging 6 to 35) in its natural state occurrence on most portion. The elastic modulus of the residual soil mass ranges between 3600 to 12300 kPa as estimated from SPT data. The allowable load bearing capacity of the soil mass as estimated from SPT data is 104 kpa.
- The residual soil at Gefersa III dam site forms semi-pervious to impervious foundation whereas the rock mass forms a pervious foundation in general. The presence of inter-site faulting and exposure of the fractured bed rock along river channel is anticipated to form a potential connection of reservoir water to the pervious foundation and abutment zones under the dam.
- Almost the entire reservoir area is covered by about 1-3 m thick residual and alluvial soil of sandy silty clay and sandy clayey silt composition. The in situ falling head tests show that the silty clay soil has average infiltration rate ranging from 1.34 x 10-4 to 7.45 x 10-5cm/s. Thus it is concluded from the field investigation and laboratory test results that the reservoir area of Gefersa Dam III is covered by sandy clayey silt/ sandy silty clay soil blanket of low permeability.
- With regard to stability of Gefersa III dam site reservoir area, no scars and any other instability problems are observed except minor erosion of soil at the sides of the rivers and their tributary streams.
- Clay for impervious requirement of the dam core and ignimbrite for masonry and rip rap requirement of the project is available in close vicinity of the project site.

6.2 Recommendations

The following recommendations have been made

- The foundation formation for the intake structure is moderately weathered and intensively fractured andesite/ rhyolite rock mass of poor quality (with RMR of 34 – 36%). The rock mass has low load bearing capacity and is permeable due to intensive weathering and fracturing.

- Thick residual soil deposit covers Gefersa III dam site. It has low load bearing capacity. The soil is semi pervious to impervious. The soil deposit forms the foundation formation for the dam/ weir.
- The reservoir area is covered by soil blanket. Only selected sites at stream cuts needs treatment to improve the water tightness of the reservoir area.
- Clay core material and ignimbrite rock for masonry work and rip rap are available in the close vicinity of the project site. Coarse aggregate requirement of the project can be satisfied by aggregate supplied in the surrounding area of the project site.

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CHAPTER FIVE

5.0 MATERIAL AND METHODOLOGY

5.1 Materials Used

Geostudio 2007 is playing a great role to perform this modeling. The modeling includes seepage analysis, slope stability analysis, and deformation analysis under static and dynamic condition.

5.2 Methodology

5.2.1 Dam analysis

5.2.1.1 Seepage Analysis

Ensuring the embankment safety against eternal erosion, piping and excessive pore pressure is the basic task while designing embankment dam.

Steady state and transit seepage analysis is conducted to estimate the amount of seepage flow through the embankment. Finite element method is the basic principle while using seep/W in geo-studio software to conduct seepage analysis.

5.2.1.2 Slope Stability Analysis

The type of fill materials to fill the geometric section of the embankment and the foundation condition are the basic factors which affect the stability of the embankment. Shear strengths, unit weights, and pore-water pressures are the main inputs for slope stability analysis. The stability of Gefersa Dam is analyzed using Slope/W from Geo-Studio software.

The main objective of slope stability analysis is to determine the factor of safety for different slip surface and the analysis is conducted for different loading conditions.

The loading conditions are:-

- Upstream and downstream slopes under steady state seepage condition
- Upstream slope under sudden drawdown condition; and
- Upstream and downstream slopes during construction and end of construction condition.
- Upstream and downstream slopes during dynamic condition

Slope stability analysis is conducted using slope/W based on the principle of limit-equilibrium. Morgenstern-Price method is applied to obtain the factor of safety of the slip surfaces. This is because Morgenstern-Price method satisfy both force and moment equilibrium condition.

5.2.1.3 Loading Conditions

The factor of safety for different loading condition is listed in figure below.

Table 11: minimum factor of safety for different loading condition

Case	Loading Condition	Critical Slope	FoS
1	End of construction	Upstream	1.3
		Downstream	1.3
2	Sudden drawdown	Upstream	1.3
3	Steady state seepage	Upstream	1.5
		Downstream	1.5

4	Steady state seepage with earthquake	Upstream	1.1
		Downstream	1.1

5.2.1.3 Deformation Analysis

Failures by slope instability constitute about 6% of failures and 37% of incidents of embankment dams (foster et al 1998, 2000). A number of these incidents were prevented from becoming failure by early detection of the impending failure from visual inspection or from deformation monitoring, and remedial measure is taken by lowering the reservoir and/or construction of stabilization works.

In general, there are two types of stress analyses that are used in the evaluation of existing and proposed embankments. These are the total stress analysis and the effective stress analysis. The total stress analysis is used in the design of embankments for loading conditions during construction, rapid drawdown, and earthquake. The effective stress analysis should be used only in cases where the soils behave drained and piezometer data are available.

The analysis of deformation requires consideration of stress conditions imposed during construction and the stress-strain relationship of the material used in the embankment construction. In zoned embankment construction it is also important to consider both the total and effective stress conditions, and the interaction between different material zones in the analysis.

CHAPTER SIX

6.0 RESULTS AND DISCUSSION

6.1 Results of Seepage Analysis

As seep/w result indicates asphalt concrete is very effective material used as an obstacle to drop down the seepage line to prevent piping through the body of the dam. It is one of the tasks of design and construction to make the structure functional in the sense that the water is properly drained away and the quantity of drained water is tolerable and small. As the seep/w result shows, the amount of seep water to the body of the dam is estimated as $1.952*10^{-7} m^3$/sec per unit length of the dam. This unit discharge will be multiplied by the total crest length of dam to calculate the total seepage.

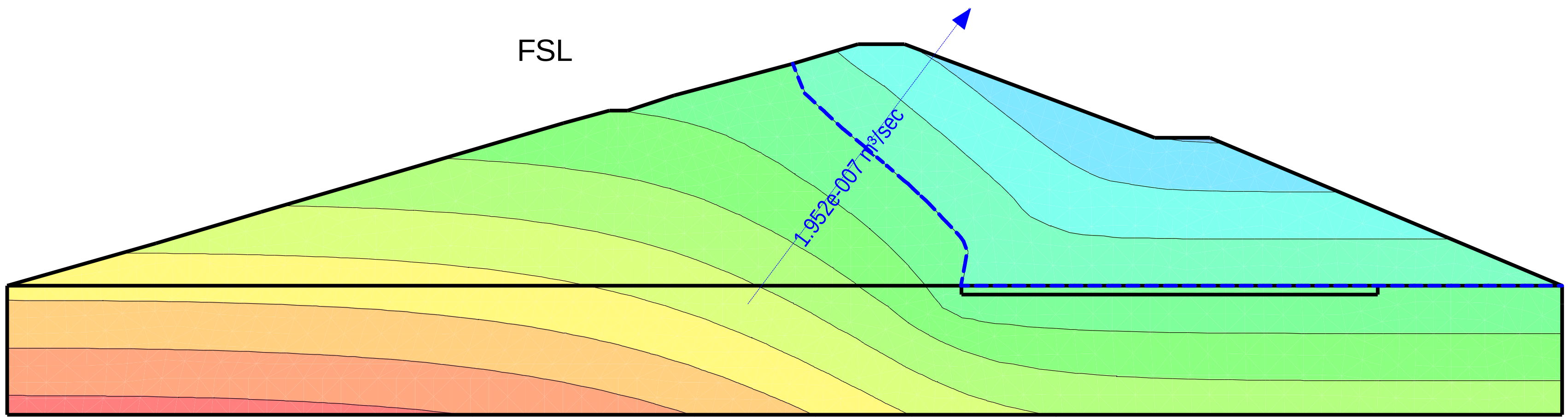

Figure 6: steady state seepage analysis

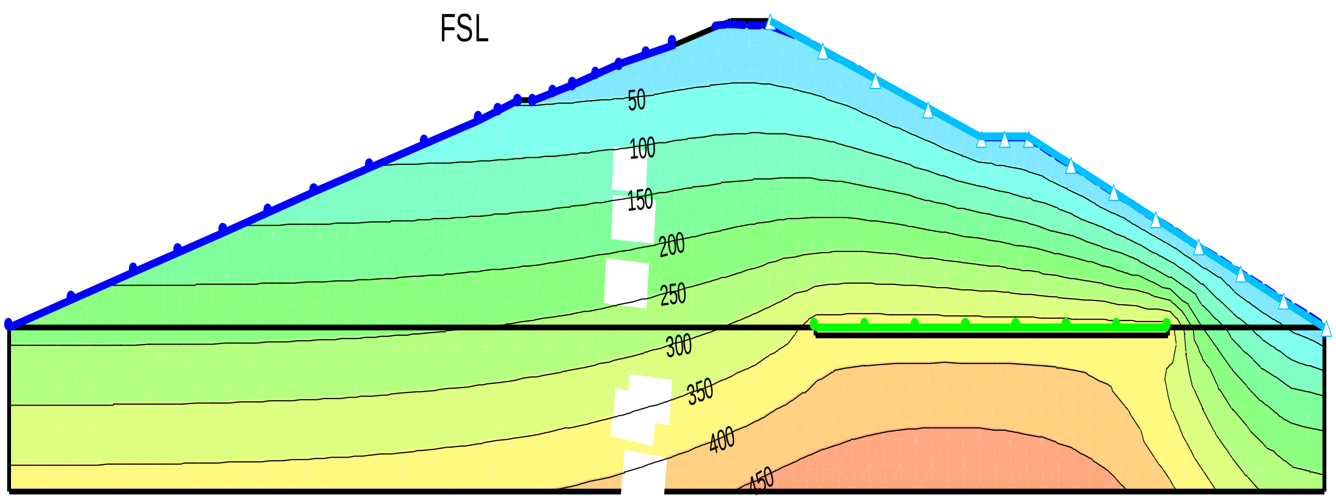

Figure 7: pore water distribution under steady state seepage

Pore-water distribution is also minimum at the top and maximum at the bottom along with the depth of water increment. The graphical representation of vertical and horizontal velocity and hydraulic gradient are presented below. As the graph show their value displayed centrally as expected.

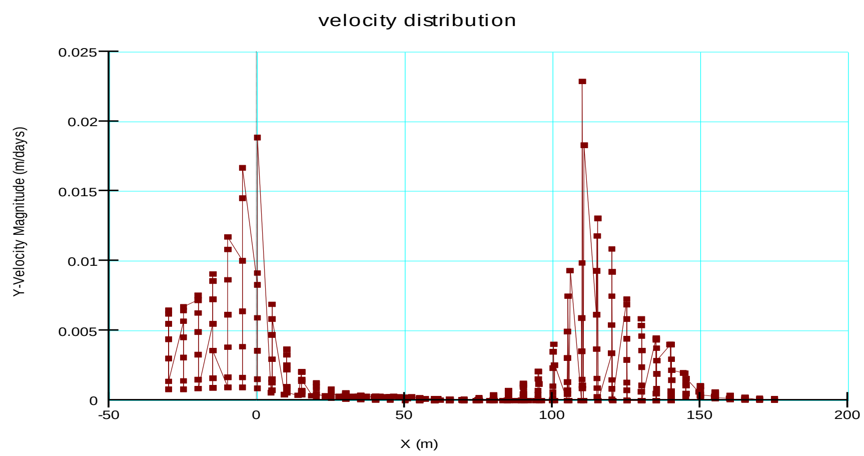

Figure 8:- y-velocity vs x-coordinate through the body and foundation

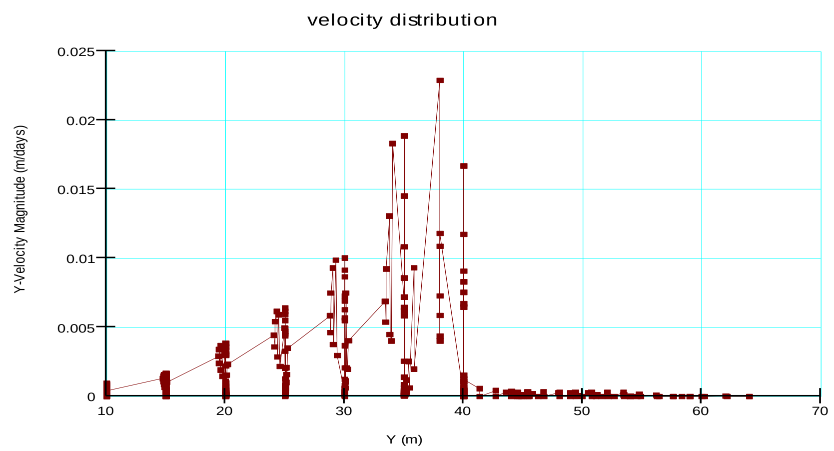

Figure 9:- y-velocity distribution vs y-coordinate through the body and foundation

The y-velocity distribution determines how fast the seep water to the foundation of the dam rather than to the body of the dam. As the above figures shows that the maximum velocity in the direction of x and y-axis are 0.023m/sec and 0.024m/sec respectively.

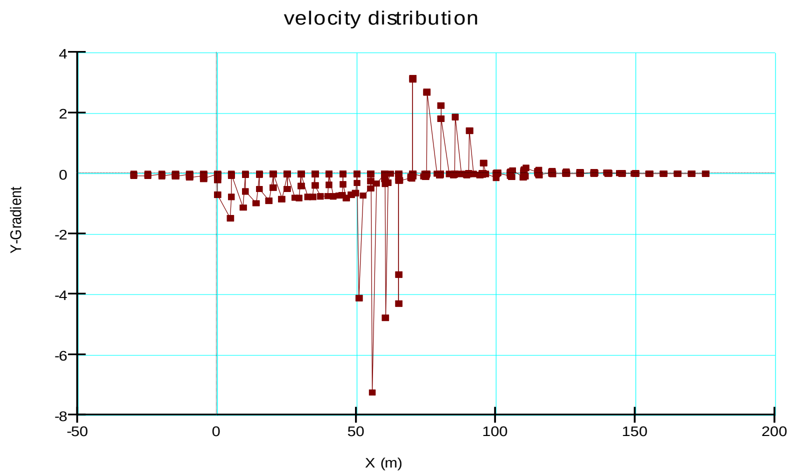

Figure 10:- y-gradient vs x-coordinate through the body and foundation

The y gradient of seep water to the body of the dam have steep slope (7.7) near the center of the dam at which the seep water dropped to the foundation with high gradient rather than crossing the body of the dam.

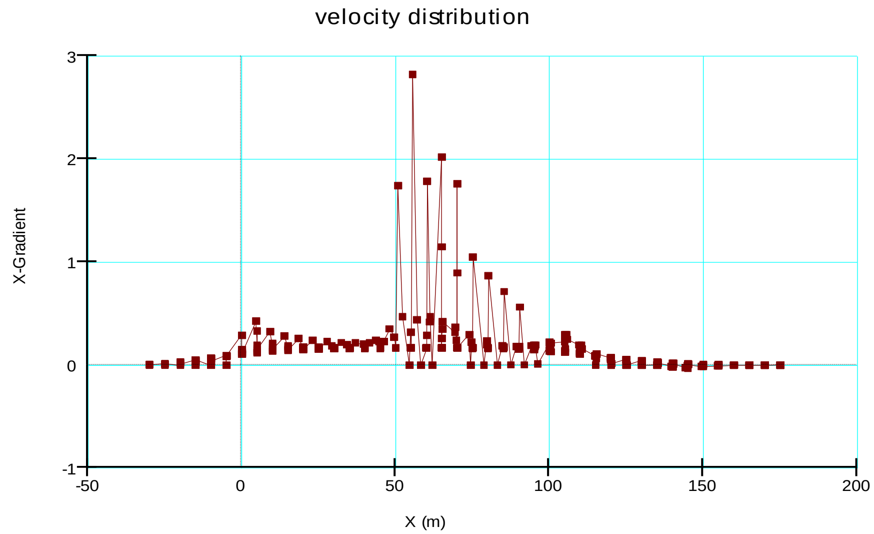

Figure 11:- x-gradient vs x-coordinate through the body and foundation

The x-gradient of seep water to the body of the dam have steep slope (2.8) near the center of the dam at which the seep water dropped to the foundation with high gradient rather than crossing the body of the dam.

6.2 Results of slope Stability Analysis

The factor of safety of the dam slope in different loading condition (steady state seepage, end of construction, during rapid draw down, and before and after earth quake) is calculated using Geo-studio 2007 software. The computed factors of safety under the different loading conditions against their slope failures are summarized in table below.

Table 12:- computed factor of safety for different loading condition

	FoSmin	Computed factor of safety		status
Loading condition		Upstream	Downstream	
Steady sate seepage	1.5	9.546	3.029	ok
During rapid drawdown	1.3	3.178	-	ok

The computed critical slip surfaces with their corresponding factors of safety are shown in Figures below. Slope stability is controlled by the shear strengths of the dam materials and the foundation (Coulomb, 1776). With a zoned dam with earth core on a foundation of equal or higher shear strength, the critical slip plane touches the foundation line. In any cases if the foundation is stiffer than the embankment the slip plane may not touch the foundation line depending of the rigidity of the foundation.

Since reservoir operation is made, the upstream body of the dam is subjected to frequent stress and strain. Earthquake demands free draining materials all over the upstream shell at a permeability of k ≥ 10-2 m/s (Kutzner, 1997). Therefore, the upstream shell material is selected as rock fill. The stability analysis before and after earth quake is done by using quake/w software and the result below shows, the dam satisfies all the requirements of USBR recommendations.

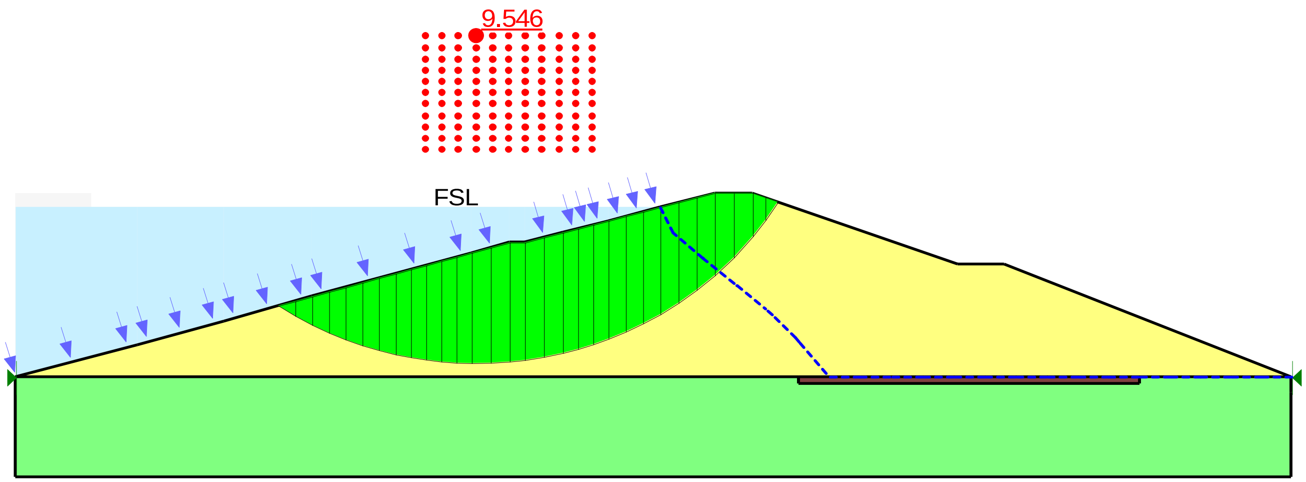

Figure 6: slope stability analysis under steady state seepage (upstream face)

As figure 13 shows that the minimum factor of safety for upstream face slope stability analysis under steady state seepage condition is about 9.546. Hence the slope is stable under steady state condition.

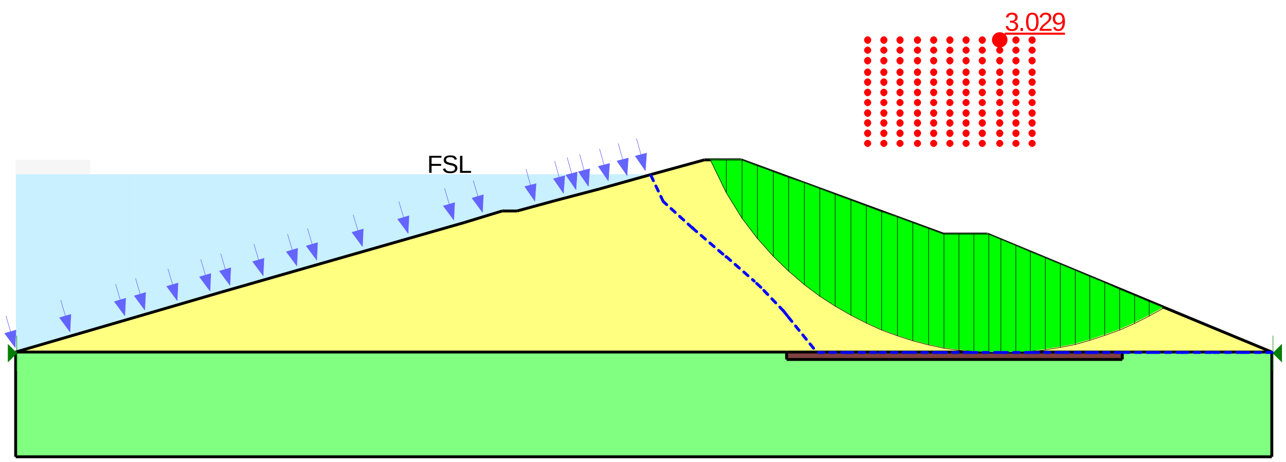

Figure 7: slope stability analysis under steady state seepage (downstream face)

To be the dam slope is safe, the downstream slope also has to fulfill the standard. As the figure 14 shows the minimum factor of safety is around 3.029 which greater than the minimum factor of safety (1.5). Therefore the dam is safe against slope instability during steady state condition.

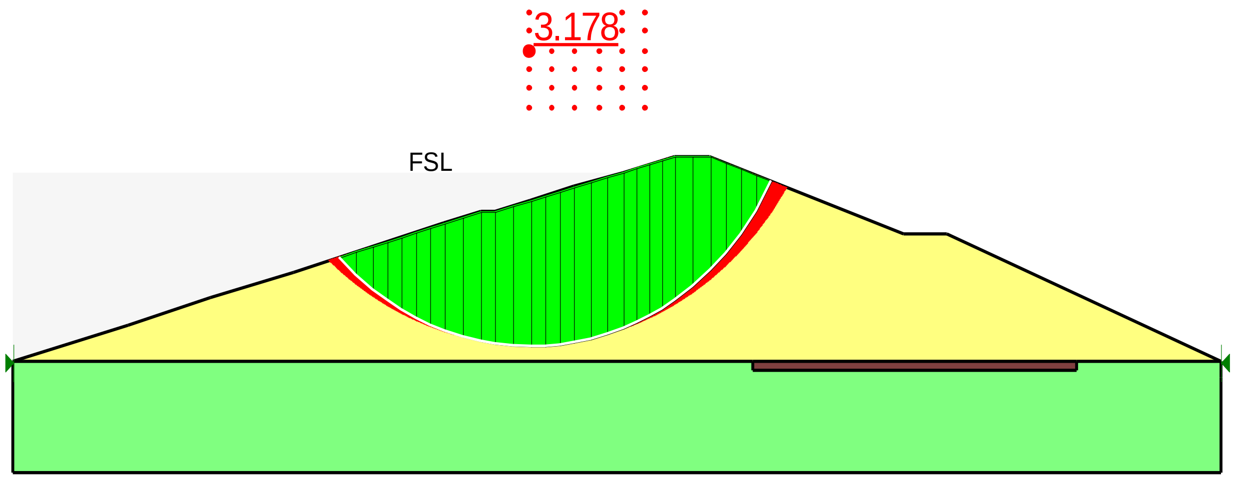

Figure 8: slope stability analysis during rapid draw down (upstream face)

Embankment slope may fail during reservoir drawdown; hence analysis has to be conducted while slope stability is done. As the result of the model (Geo-studio software) indicate the upstream slope during rapid drawdown is safe with factor of safety 3.178.

6.3 Results of Deformation Analysis

Using Geo-studio software the magnitude and distribution of vertical and horizontal distribution in each stage of construction and at the end of construction is presented below.

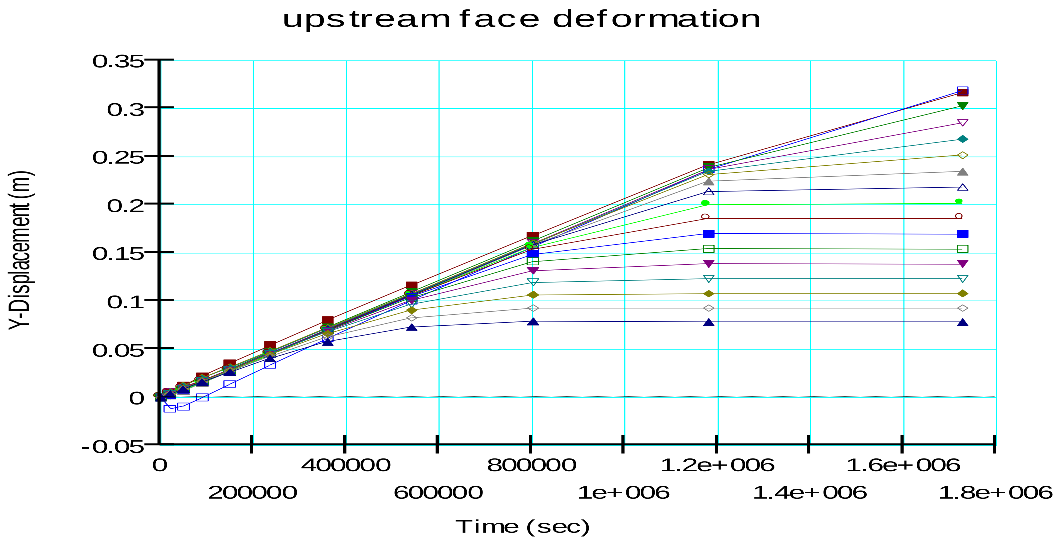

Figure 9:- ground surface settlement (at the end of construction)

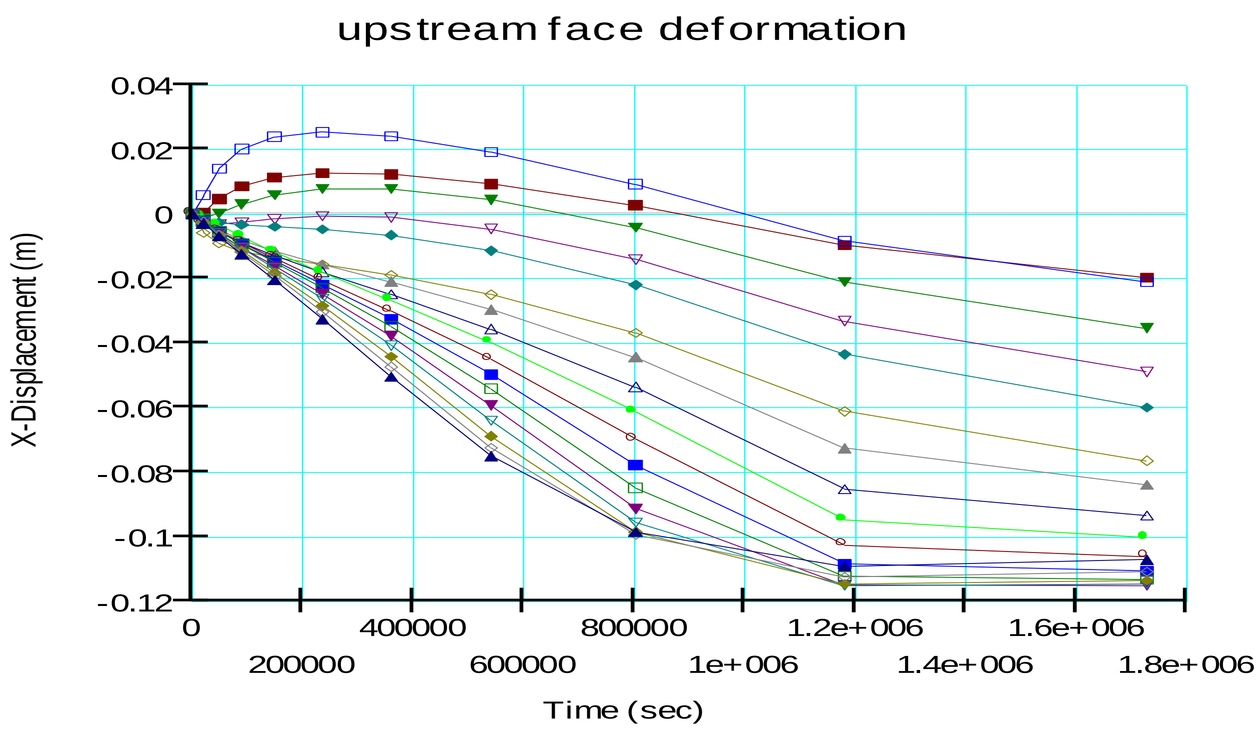

Figure 10:- ground surface settlement (at the end of construction)

As the impervious layer ranges from 0.5 m at the top to 1.5 m at the bottom, the horizontal displacement must be at most less than 0.5 m in order not to have cracking through the core. The maximum vertical displacements were 0.33m at the end of construction conditions. As the graph of horizontal settlement shows the minimum and maximum settlement are 0.021m and 0.185m respectively. Since the maximum settlement is less than 0.5m the embankment is safe against crack.

6.3.1 Stress and pore-water distribution

The stress and pore-water distribution are also presented below. The maximum value is 450kpa/m at the bottom and minimum value of 50kpa/m at the top as it expected.

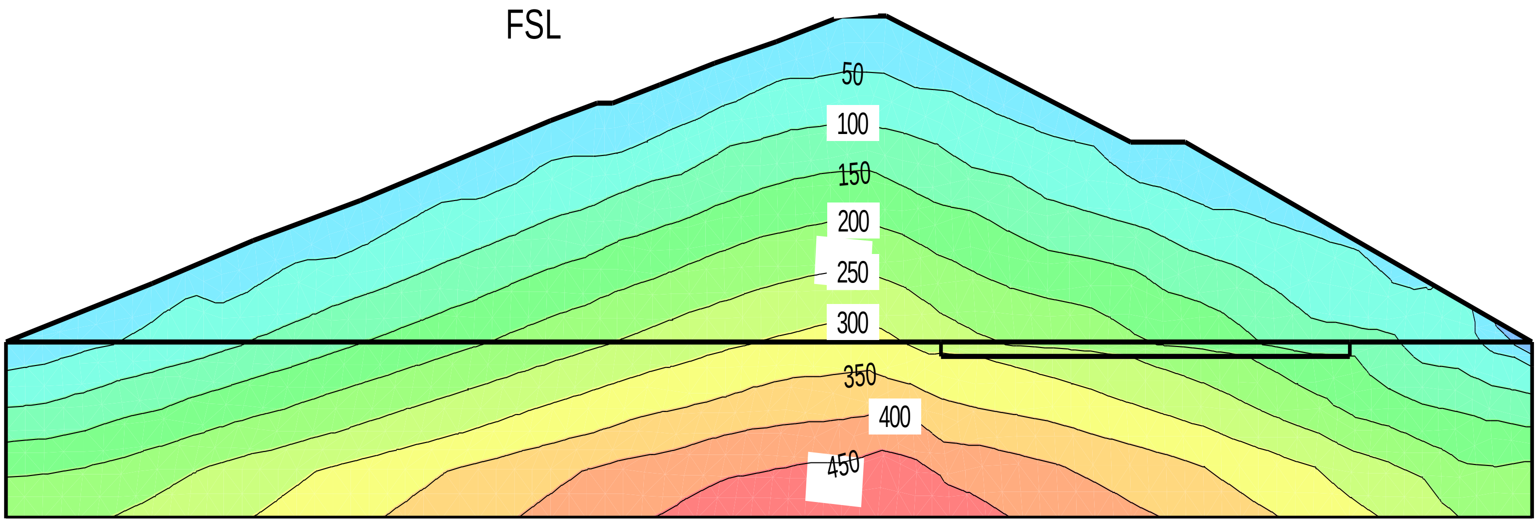

Figure 11:- stress distribution over the dam

Obviously the pore water distribution is increase as the depth of reservoir increase hence the maximum pore-water pressure is 240kpa/m at the bottom and the minimum pore water pressure is about 0kpa/m at the top.

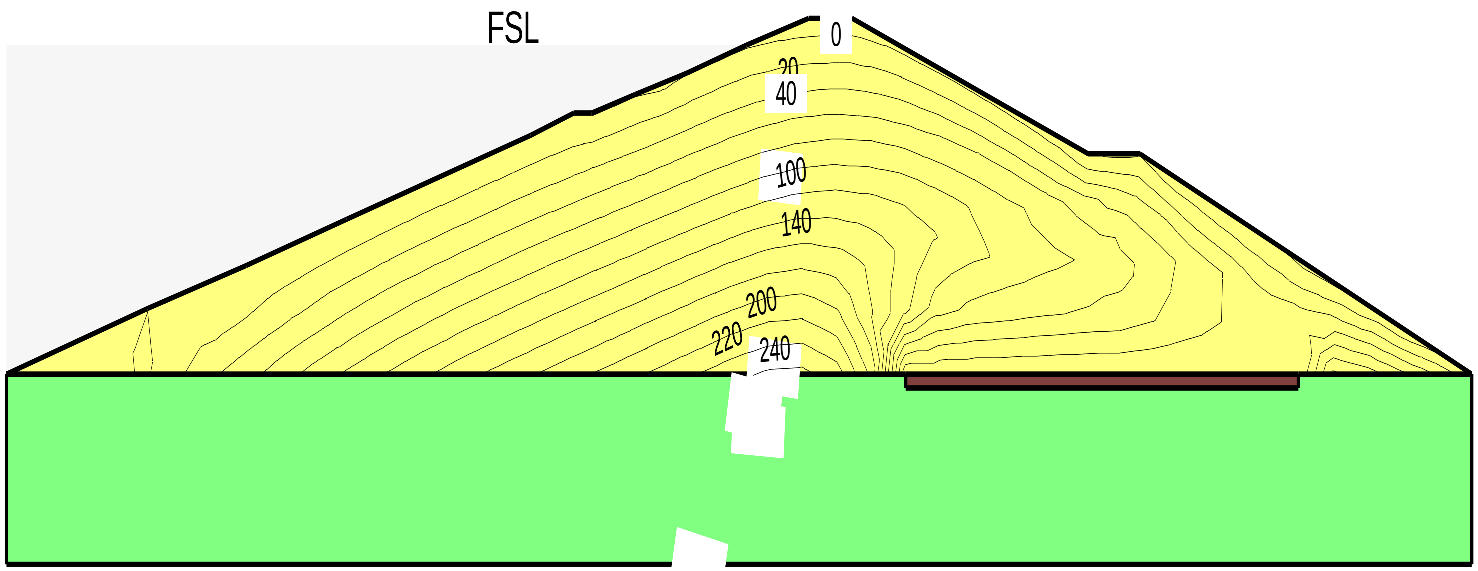

Figure 12:- pore-water distribution over the dam

CHAPTER 7

7.0 CONCLUSION AND RECOMMENDATION

7.1 Conclusion

This study mainly focus on selecting the best compactable material with the existing Gefersa I/II dam and analyzing the stability of the extended dam.

This study was highly guided on AAWSSA's inception report which include detail of material investigation and material laboratory test results. This document (AAWSSA inception report) included as one chapter(see chapter four).

Based on condition of the upstream face of existing dam, most of the fill for the heightning part is done at the downstream face of the dam with homogeneous material(clay soil) as the existing dam, hence heightning is started at the crest of existing dam by leting 2m berm as working space. To fill the total volume for heightning, the dam shift 25m to downstream side. To reduce deformatio of on the dam and to increase safety for construction berm is provided at the downstream face of the dam.

Exess seep water to the body of the dam is critical broblem in embankment dams which create piping and sloughing at the downstream face of the dam. It is obivious that piping failur is the major way of failur for most embankment dams. Hence the seep water from the reservoir to the body of the dam has to be minimized to prevent failur of embankment dam due to piping. According to seep/w analysis the seep water to the baody of the dam is around $1.952*10^{-7}m^3$/sec per unit length of the dam.

The slope stability is major issue while analysing embankment dam. Beyond piping failur, embankments also fail due to slope unstability. For this specific case study the slope of the dam become safe for all loading conditions.

Deformations of an embankment dam start occurring during the construction of the dam. These deformations are caused by the increase of effective stresses during the construction by the consecutive layers of fill material and also by effects of creep of material. As the analysis results that were found from Geostudio-2007 indicate that the maximum vertical and horizontal settlement is 0.33m and 0.185m respectively. Hence those settlements are less than allowable limit (0.5m) the embankment is free from crack problem.

7.2 Recommendation

As the written document on the Gefersa reservoir shows, a number of remedial actions are taken to eliminate the problem of sedimentation from the main reservoir. In 1954 rehabilitation is done as the first remedial action, but the problem is continued. After a decade Gefersa-III dam is also built at the upstream face of the main dam to trap the sediments before enter into the reservoir, again this action also couldn’t be break through solution. For the third time AAWSA propose to heighten the existing dam as a solution. But it seems not the best solution. Hence research has to be conducted which come up with a breakthrough solution.

Before construction is started, the rip rap and top clay soil at the downstream face of the existing dam has to be removed to compact the fill material with existing material.

Since most of the fill for heightening is loaded on the downstream face of the dam, the crest of existing dam is shifted to the downstream side by 25m. Due to this reason the pipe through the filter of existing dam was covered by the fill material, hence it has to be extended to the allowable length for well-functioning of the pipe.

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