ENGINEERING GEOLOGICAL EVALUATION OF EMBANKMENT DAM FOUNDATION AND CONSTRUCTION MATERIALS CHARACTERIZATION OF FATO DAM SITE, CENTRAL ETHIOPIA

MASTERS OF SCIENCE IN ENGINEERING GEOLOGY

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Declaration

Here, I declare that this thesis is my original work and has not been presented in any other University for any degree award. All sources of materials used in my thesis work have been duly acknowledged.

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Advisor's Approval Sheet

This is to certify that the thesis entitled ''Engineering Geological Evaluation of Embankment Dam Foundation and Construction Materials Characterization of Fato Dam site, Central Ethiopia' 'Summited in partial fulfilment of degrees of Master of Science in Engineering Geology to School of Earth Science. It was carried out by Diribsa Chala with ID No GSR/3543/11 under my supervision. So here I recommend that the student's thesis work has fulfilled the requirements and can be submitted to the school.

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Examiners Approval Sheet

We, the undersigned members of the board of Examiners of the final open defenced by Diribsa Chala have read and evaluated his thesis entitled ‘’ Engineering Geological Evaluation of Embankment Dam Foundation and Construction Materials Characterization of Fato Dam site, Central Ethiopia’’. Therefore, the thesis has been accepted in partial fulfilment of degree of Masters.

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Abstract

Our country, Ethiopia, has constructed a number of dam projects, including the Renaissance Dam in Abay. The location of the present study (the Fato Dam site) was located in upper Guder, which was a sub-basin of the Abay basin. Geographically located between 360252E/979403N at the right abutment and 360386E/979644N at the left abutment. However, the location of construction materials was here and there within a less than 1km radius around the dam site, except for the sand proposed from the Senkelle area. The local geology of the site was composed of basaltic volcanic formations (vesicular, porphyritic, and aphanitic) from top to bottom. The tops of the abutments were composed of residual soils. Especially in the right abutment, the thickness of the soil reached around 30m from the surface.

The objective of the present study was the evaluation of the dam foundation and the characterization of the construction materials used for each zone of embankment. Evaluation of the foundation was mainly in terms of its seepage condition by plaxis software, with construction materials suitability in terms of location, quality, and quantity based on standards and finally recommend remedial measures for the seepage problems.

In order to meet the objective of the present study, literature reviews of different papers from both published and unpublished sources and reports of geotechnical investigations (previous work) of the site were used. In addition to these geophysical survey report, bore logging data and other in-situ data done by WWDSE in 2016 and Plaxis 2D software seepage analysis were performed to evaluate the seepage condition of the dam foundation and construction materials at the site.

From the borehole data and laboratory test results, the foundation soil was residual, dominantly cohesive soil. The soil of the foundation exhibits low to high plasticity in silty clay and also contains sand. The SPT results of the foundation were interpreted as medium to stiff consistence, while strength fell between 50 and 400 Kpa in the range of 4 to 15 N60 values for dominant in different depths. The permeability of the foundation soil was low to very low. The rock mass rating of the foundation fell from fair to good in its quality. The strength test results of the foundation rock ranged from 42-478 Mpa uniaxial compressive strength, and the permeability packer test result was above the required lugeon (<3LU) values. These were due to the deep, localized fractures and weathering conditions of the site.

From the gradation analysis of construction materials, the core material was suitable as it meets the standards. The sand material proposed for the filter and transition zone from the Senkelle area was not suitable because it contained a fine percent above the standards. In turn, locally available basalt is processed and used for filters and transition zones. Rock fills, riprap, and rock toe were also prepared from a basaltic quarry located near the site.

The foundation of the embankment dam had seepage problems, so improvement techniques, three rows of curtain grouting of depth 0.75 H (height dam above reservoir), and around the highly affected toe of the dam should be treated with dental treatment of Shasta's formula.

Key Words: Permeability, Seepage, Plaxis 2d, core, filter

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List of Abbreviations

AASHTO ------------------American Association of State Highway and Transportation Officials

ASCE ---------------------American Society of Civil Engineers

CB -----------------------Coarse Boundary

CH ---------------------High plasticity Clay

CL ------------------------Low plasticity Clay

$D_{15}B$ ------------------------- 15 percent of base soil size

$D_{15}F$ ----------------------------15 Percent of Filter size

ECDSWC --------------------------Ethiopia Construction Design and Supervision Works Corporation

FB -------------------------Fine Boundary

ICODS ---------------------------Interagency Committee on Dam Safety

ICOLD --------------------------International Commission on Large Dam

LLF1 -Master--------------------Lower Limits of Fine Filter

LLF2 -----------------------------Lower Limits of Coarse filter

MH -------------------------------High Plasticity Silty soil

ML ‘ ---------------------------Low Plasticity Silty soil

MoWIE ------------------------------Ministry of Water, Irrigation and Energy

NMC -----------------------------Natural Moisture Content

OMC -----------------------------Optimum Moisture Content

RMR -----------------------------Rock Mass Rating

RQD ------------------------------Rock Quality Designation

ULF1 -------------------------------Upper Limits of Fine filter

ULF2 ----------------------------------Upper Limits of Coarse filter

USCS ---------------------------------Unified Soil Classification System

USBR ----------------------------------Unite State Bureau of Reclamation

USSD ---------------------------------Unite State Society on Dams

VES ---------------------------------Vertical Electrical Sounding

WWDSE --------------------------------Water Works Design and Supervision Enterprise

CHAPTER I INTRODUCTION

1.1 Background

A dam project is usually built in the valley of a river for the purpose of industrial needs, agricultural development, the generation of electric power, controlling flooding problems, and the protection of the groundwater aquifer. Dams are also used for recreation purposes in addition to the above purposes. Based on construction styles, there are different types of dams. The major types of dams are gravity, arch dam, buttress, embankment, and composite dam. The selection of a dam site is based on many parameters, such as geology, topography, availability of suitable construction materials, climate, hydrology, etc.

The present study was focused on the Fato dam site, which was located in Oromia Regional State, West Shoa, Ethiopia. Fato Dam is an embankment type of dam, and its current status is under construction for irrigation purposes. The maximum height of this dam is 40.6 m, with a crest length of 283m. According to the (WWDSE, 2017) detail design report of Fato Dam, the gross storage capacity of the reservoir is up to 57.71 Mm3. The project was initiated by the Federal Democratic Republic of Ethiopia (FDRE) Ministry of Water, Irrigation, and Energy (MoWIE) which had been involved in a number of water and land resource development project studies with the objective of expanding and intensifying water resource use for irrigation and other development sectors. As one part of this development, the Upper Guder Multipurpose Project had been designed with the aim of achieving power generation, food security, and income generation for the local farming community. Accordingly, the Ethiopian Construction Design and Supervision Works Corporation (ECDSWC) Water and Energy Design Supervision Works Sector had entered into an agreement with the Ministry of Water Irrigation and Energy (MoWIE) to conduct a feasibility study and detail design of the Fato dam project, which had large-scale irrigation of about 6000ha. However, according to the feasibility study results, only 4947ha should be irrigated by a sprinkler irrigation system from Fato Dam.

The scope of the study done by ECDSWC included geotechnical investigations on all parts of the dam site, such as the spillway/intake structure, reservoir area, power house, and irrigation pipe route. The investigation has also partly involved the assessment of construction materials for dam fill and appurtenant structure construction.

The site investigation covered geophysical resistivity imaging and vertical electrical sounding (VES), core drilling, in-situ testing, piezometer installation, groundwater level monitoring, sampling, and laboratory analysis activities.

In the present study, the main emphasis was given to two main components of the Fato dam. The first one was the evaluation of the foundation part of the dam using engineering geological principles, and the second issue was the characterization of construction materials. So the standard procedure was adopted for the engineering, geological evaluation of the foundation, and characterization of construction materials for each part of the dam.

1.2 Location and Accessibility of the Study area

The Fato Dam site is located in Oromia Regional State, about 150km West of Addis Ababa. Geographically, the site is located between 360252E/979403N at the right abutment and 360386 E / 979644 N at Left Abutment. The irrigation command area of the project lay on the right bank of the Fato River. The Fato River is the upper Guder River, which is located in the Guder sub-basin. The project shares two Woreda administration boundaries, namely Toke Kutaye and Dire Inchini. The altitude of the project area ranges between 2150 to 2460 m. The Fato River was selected as the main source of water for the proposed irrigation dam project.

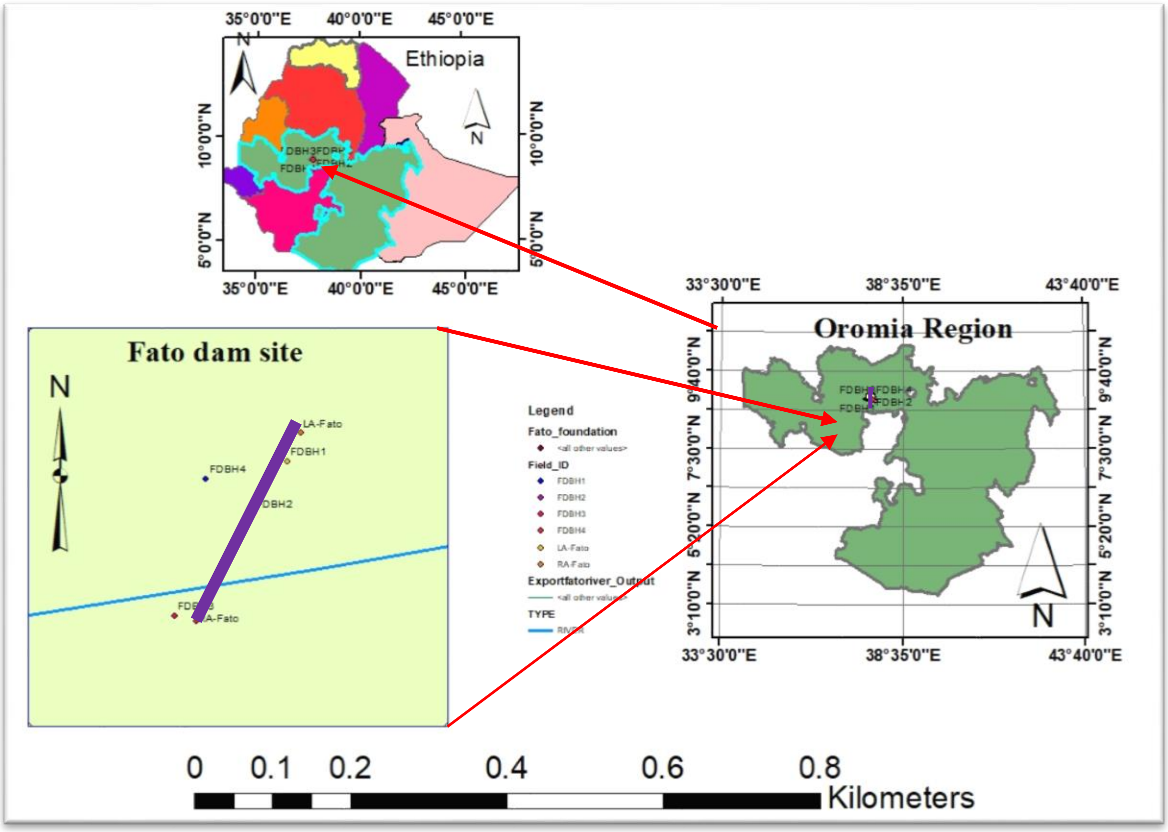

Figure 1.1 Location map of Study Area

In terms of accessibility, the project area is accessible through 135 km from Addis Ababa-Ambo-Guder asphalt road and Guder-Gebeta 15 km all weather gravel road accessible to the project site.

1.3 Statement of the problem

So many earth and rock fill dams are built over the world; however many of them do not function as initially proposed. This is because of the uncertainty in geological investigations. So such problems are reduced by more and more investigation, from site selection to service time. Seepage through the foundation within the dam body and abutment is one major problem with water moving through the embankment and foundation materials (ICODS, 2015). This may cause leakage through the foundation if no systematic investigation and improvement are done.

According to (Foster et al, 1997) greater percentage of the world's embankment dam failures were the result of internal erosion and piping. In the present study, the area was located in the central Ethiopian plateau, composed of geological formations of tertiary volcanic and quaternary superficial sediment deposits. The geological sitting consists of pervious volcanic rocks associated with different geological structures that affect the suitability of the dam site.

According to WWDSE (2016), the geophysical survey indicated the presence of a weak zone at the left abutment, so they recommended further investigation to evaluate the condition associated with the site and its influence on the dam foundation and dam structures. The present study also considered the characterization of construction materials. The characterization of fill materials was done by correlating them with foundation seepage. Therefore, the present study applied engineering geological principles to evaluate the foundation condition and characterization of the construction material of the Fato dam site by using Plaxis 2d software.

1.4 Objective of the study

1.4.1 General Objective

The major objectives of the present study were Engineering Geological evaluation of foundation condition, characterization of construction materials and give engineering geological judgment based on the results of the study.

1.4.2 Specific Objectives

- ✓ To determine important engineering geological properties of rocks and soils present in the foundation area of the dam and construction materials
- ✓ Characterization of construction material for each zones of embankment.
- ✓ To analysis Seepage condition of foundation and construction materials and recommend remedial measures.

1.5 Research Questions

Research questions that should be answered through this study are:

- ✓ What types of rocks and soils found at dam foundation?
- ✓ What were the major geological problems that influence the Fato dam foundation?
- ✓ What types of remedial measures were recommended to overcome the site problems?
- ✓ What was the overall suitability of construction materials for each zone of the dam parts?

1.6 Significance of the study

The present study was based on an integrated geological and geotechnical approach based on available data of the site. The evaluation of the dam foundation and construction material characterization was done. The aim of the study was to develop knowledge of the researcher on the dam project. The project authority and enterprise can use this work as additional input for the assessment of the project. It may also be able to be used as a source of information for future researchers who intend to work on the same geological sitting or the same study area. In other case the study is used for the improvement of the researcher's knowledge and skills as well as for the fulfillment of the M.Sc. academic degree.

1.7 Scope and Limitation of the study

The present study covers only the evaluation of the foundation part of the dam and the characterization of construction materials. The study was performed at the foundation and near the foundation of the dam in order to evaluate the suitability of the dam foundation in terms of seepage. In the case of construction materials, its suitability in terms of quality and quantity, as well as its distance from the dam site, were analyzed. Primary and secondary data are mainly used in this study with the permission of the project owners and clients due to financial constraints.

1.8 Scheme of the study

The structures of this study arranged into seven (7) chapters as follows:

Chapter I: gives the general information of introduction of the study, background, statement of the problems, location, and objective of present study and significance of the study.

Chapter II: Literature review provides basic information from different sources regarding on the present study.

Chapter III: General overview of study area and this is deals with geology, physiographic, hydrology (hydrogeology), Soil, land use land cover and climate condition.

Chapter IV: Focused on the main Methodology followed, Material used for the present project.

Chapter V: Describes the general engineering geological evaluation of Dam Foundation of Fato dam.

Chapter VI: Discusses about characterization of construction materials and seepage analysis by plaxis software.

Chapter VII: Deals with the conclusion and recommendation part of the present study. Finally reference list and appendixes are found at the end portion of the paper.

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CHAPTER II LITERATURE REVIEW

2.1 General

A dam is a hydraulic structure that is constructed in order to store water for different purposes. Irrigation, power generation, industries, animals, and human drinking water supply are some of the purposes of dam construction. Since dams are used for such purposes, evaluation of the actual condition of the site is required in order to monitor and manage the risks of failure that occur during and after the construction phase.

The risk of dam failures is more danger to people than any other engineering structure (USBR, 2002). After the dramatic failures of the Saint Francis dam occurred at midnight in 1928 near Los Angeles (California), the assumption on dam engineering greatly changed (USBR, 2002). Today, millions of researchers are working on dam engineering because this time is the time that requires more water resources since the population of the world is increasing dramatically. As the population increases, the requirement of basic needs also increases.

Different studies focus on the investigation of dams during design, construction, and service time to evaluate the site condition for mega projects like dams, which is a pre-requisite element (Nigatu Fekadu, 2006; Rashid and Mohsen Haeri, 2016). According to Gebremedhin Berhane and Walraevens, 2013 most countries Projects faced problems with water supply after completion. This is mostly caused by a lack of appropriate evaluation and remedial measures at the site of the water supply project.

Dam site selection is mainly based on both engineering and economic considerations for the project. The main parameters that influence the choice of dam types are: topography, catchment characteristics, morphology of the river valley, geology, availability of construction material at a reasonable distance, climate, and flood regime. Based on construction materials, dams are categorized into concrete dams and embankment dams. Gravity dams, arch dams, and buttress dams are made of concrete, while embankment dams are built of rocks and soil fill.

2.2. Embankment Dams

When earth and rock fill are used as the main construction materials, the dam is known as Embankment dam. Currently, this type of dam is widely used over world because of its economics and simplicity of construction. Especially embankment dams are favourable on wide bases when it requires huge amounts of concrete to construct gravity dams. Embankment Dam uses every natural material from clay size to big stone with minimum processing, so it is cost-effective because it is built from locally available materials (Bisrat Alemayehu, 2014; GSJ, 2018; Mulatu Tumoro, 2010; USSD, 2011).

According to USACE (2004), there are two main requirements for embankment dams. The first one is a technical requirement, which includes issues relating to:

- ✓ Foundation and abutment stability both under dynamic and static conditions.
- ✓ Seepage must be controlled in all parts of the dam.
- ✓ Freeboard must be sufficient to prevent overtopping and settlement of the foundation and embankment.
- ✓ Spillway and outlet also must have the capacity to prevent overtopping of the embankment.

The second requirement is linked with the administrative work of the project, such as environmental assessment, monitoring, maintenance, evaluation, an action plan, etc. In addition to this, the foundation materials must be properly prepared and treated during construction (USSD, 2011).

Many researchers classify embankment dams into two main types. These are earth and rock-filled embankment dams. Embankment dams can also be classified based on the method of construction and characteristics of the fill.

Based on construction method, embankment dams are classified as: (i) hydraulic fill, in which materials were transported and filled by mean water pressure through the channel route. (ii) Semi hydraulic fill dam was when materials were transported to the site mechanically but were placed in fill with the water jets. (iii) Another type of embankment dam was rolled fill, and it was built with compaction equipment and earth materials transported by earth mover machines. (iv) A rock fill dam was also one category of embankment dam in its construction; it comprised loose rock fill dumps, impervious facings upstream, etc. (v) A composite dam is the type of embankment dam that incorporates the above classes in its nature.

Based on the characteristics of the fill materials, embankment dams are classified as either homogeneous fill dams or zoned dams. A homogeneous fill dam was required when one type of fill material was available and used to construct the entire dam. It was used in areas of flattened slopes that were low to moderate in terms of height. Seepage is controlled by decreasing the velocity of the percolating water.

Different embankment dams are built using several design approaches (Fell, 2005). A zoned dam is a rolled-fill dam that consists of several zones in which permeability increases from the core toward the outer slopes. The number of zones is determined based on the availability and types of borrowed materials. The stability of the zoned dam is mostly due to the weight of the heavy outer zones.

According to WWDSE (2016), the final option for the embankment dam type for the Fato Dam site was a zoned rock fill dam with a core made of natural sealing clay material. The selection was done based on the available construction materials near the dam site.

2.3 Challenges in Embankment Dam Construction

General

Embankment dams can be faced different problems, starting from the initial construction stages to its service time. Some of the challenges of embankment dams were linked with the appropriateness of the site and how well it was investigated to understand the condition of the site at all. Dams and other mega project may have been affected by challenges emanating from earthquakes, leakage conditions, deformation, settlement, movement of faults around dam foundations, etc., which may affect the embankment dam project (Sigtryggsdóttira & Snæbjörnsson, 2019).

2.3.1 Main Causes of Embankment dam failures

The failure of the embankment dam was much greater than that of other concrete dams. Embankment dam failures were caused by different factors. One is due to poor-quality materials used in its construction. So it is more susceptible to failure than the gravity dam if improper design, construction, and maintenance are done. Major causes of failure are piping, internal erosion, overtopping, liquefaction, and sliding (ICODS, 2015; Kunitomo, 2000; US Bureau of Reclamation, 2002; USSD, 2011). ICOLD (2012) classifies dam failures into three main groups. These are hydraulic failures, seepage failures, and structural failures. These and other causes of the failure of embankment dams are discussed as follows:

2.3.1.1 Seepage failure

Uncontrolled flow through the dam foundation and parts of the dam leads to dam failure. Most earth dam failures happen due to seepage. According to ICOLD (2012), Foster (2000) found that seepage failures account for 30% of total embankment dam failures. Piping, internal erosion, uplift, and sloughing are the main contributors to seepage failures.

Piping is the progressive erosion and subsequent removal of soil grains within the body or foundation of the dam. When permeable cavities or strata (such as sand and gravel) are available in the foundation of the dam, then water may seep at a huge rate through it. So this huge rate of flow may erode the soil and create a hollow below the foundation of the dam. As a result, this causes the dam to sink. Piping through the dam body is developed during the formation of concentrated flow channels in it. The development of flow channels is due to improper construction, insufficient compaction, shrinkage cracks, cracks in the embankment due to settlement, and so on. The soils are removed from these channels, and a hollow is formed in the dam body, causing the migration of materials. So this may cause the embankment and other structures of the dam to fail.

Sloughing, the other seepage failure category, occurs due to the progressive removal of soils from wet downstream faces. The repetition of wetting and sliding that takes place continuously causes dam failure. So this is termed sloughing. It usually occurs at the portion of the dam below the exit point and always remains in a wet condition. So this causes a reduction in the stability of the slope, and small-scale sliding may occur.

2.3.1.2 Hydraulic failure

Hydraulic failures are due to overtopping, erosion of upstream and downstream, and erosion of the toe of the dam. The failure of dams in this category is contribute 40% of total embankment dam failures (ICOLD, 2012). Some reasons for these category failures are:

Overtopping failures: occur if the design flood is underestimated, when spillway is insufficient capacity and the spillway gates are not properly operated.

Upstream face erosion: is caused by wind and wave action on the upstream face materials if the materials are not resisting the wave velocity. So this leads to the washing out of dam materials and causes the overturning of the protection slab on the upstream face.

Cracking: This is due to frost action upstream, which may cause the failure of a dam. This means that if frost presents in the upper parts of the dam, it causes heaving and cracking of the soils with dangerous seepage, which leads to failure.

Downstream ace erosion: is the other hydraulic failure of embankment dams, which mainly occurs during heavy rain on the downstream face. The rain forms gullies of erosion, which leads to dam failure.

Toe erosion: is one hydraulic failure of embankment dams that is caused by the downward movement of water from spillway buckets and tail water.

2.3.1.3 Structural failure

Structural failures are contributing about 25% of overall dam failures and caused by shear failures. Then this was leading to sliding of slope materials. Some of structural failures are described as follows.

I. Due to pore water pressure: excessive pore water pressure in confined seams of sand and silt, artesian pressure in abutments and similar factors are reducing the shear strength of the soils. Loose sand foundation may fail during liquefaction or flow sliding.

II. Upstream slope failure: In this case steep slope, weak embankment materials and sudden drawdown of reservoir are the causes of structural failure. In addition to this, seepage force acting along the direction of sliding causing the increase of driving force for slope direction and leads to catastrophic failures.

III, **Downstream slope failure:** This may occur when reservoir is at maximum rate; the downstream slope is more vulnerable to slide due to seepage force acting in the direction of driving force for sliding. Foundation of dam itself may slide if it is from silt or soft soil. This is because of the slow consolidation and expansion process of clay soils. This shows the reality that saturation will decrease the shear strength of the soil foundation.

IV Foundation slide: Dam body may slide if the foundation is from soft materials such as silt and soft soil. The slow consolidation and expansion of clay soils due to the saturation will decrease the shear strength of the soil foundation. Finally shear strength failure of foundation leads to sliding of both upstream and downstream slopes of dam body.

2.3.2 Embankment Dam Failure Treatment

The main purpose of site investigation for any project is identifying the condition of the site as per as its requirement. But usually what actually observed at site is not fit with the proposed project like dam which may because of the problem of geological condition present at that site.

In order to overcome these types of problems, geotechnical and engineering geological expert work together in identification of case of the problem by gathering information on: deep, shallow, soils, rocks, ground water, settlement and etc. Then these experts analysis the information they get from site visualization, insitu tests and laboratory test and finally reach the decision for improvement techniques selection by consideration of reasonable cost and other related factors.

2.3.2.1 Excavation of weak materials

Highly weathered materials excavated and replaced by sounded materials from other quarry site. This type of improvement techniques is effective if the problematic materials volume is not huge. In this case soil with low engineering property replaced by soil having better engineering properties. The process is done by combination of excavation and compaction of the materials. This means that, the replaced materials should be compacted to good performance.

2.3.2.2 Well Compaction

Compaction performed through external compactive effort in order to increase the density of the foundation soils and embankment fill. Types of compaction are selected based on the condition and material type at specified site. The parameters improved through compaction are increase in shear strength, durability, reduction of (permeability, compressibility, and liquefaction potential) and controls swelling and shrinking of soils.

2.3.2.3 Flattening slopes

Steep slopes of embankment dam are dangerous because it is more vulnerable to failure than gentle slope. Therefore, decreasing the steepness of the slope by different mechanisms improves the safety of the dam. In addition to the above, avoiding denser materials at top part of the slopes, grassing the surface of slopes and similar activities are used as improvement of slope failure of embankment dam.

2.3.2.4 Providing drainage filters

Drainage filters are used to control seepage through, under and around the embankment dam. Zoned embankment dams require horizontal drains, weighted graded filters, relief wells and chimney drains. The purpose these drains and filters are to reduce the seepage force which endangering the stability of downstream slopes.

2.3.2.5 Foundation Grouting

Grouting is methods used in ground improvement by injecting fluid like materials into subsurface soil or rock. Because of its cost, grouting is applied to limited to zones of relatively small volume that has special problem.

Grouting can be done in foundation during the following activities. The first one before construction to control (water problem, infill voids, reduce settlement and increase soil bearing capacity). During construction stage grouting controls (ground water flows, stabilize loose sand against liquefaction and provides proper lateral support). Grouting is very important also after construction by providing the following function. These are to reduce vibration of foundation by machine, to eliminate new seepage and etc.

2.4 Seepage potential Evaluation of dam Foundation

Seepage evaluation here is linked with permeability of foundation materials of the dam site. This is done by different techniques of investigations of surface and subsurface of the dam site. These are geological study, borehole logging and geophysical study of subsurface of the dam site. In addition to this insitu and laboratory tests are done for analysis the seepage case.

Seepage is common problem of dam especially embankment dams are susceptible to this kind of challenges.Rezagholilou (2006) is discussed about the potential problems of seepage regarding to the dam projects. He stated that the problem may occur in different parts of the dam as follows: The first one is internal erosion or piping of core materials along downstream or into foundation of the dam itself. Foundation material erosion along the permeable zone of the embankment downstream from core is another source of seepage failure as stated by Rezagholilou (2006).Sloughing also seepage problems usually occurs near to the toe portion of the dam below the exit point. Therefore, detail evaluation of seepage and deciding appropriate controlling method is very important for recommend safe and durable foundation of the dam. In order to choose the best method for controlling seepage problem at dam site the following activities should be performed.

2.4.1 Insitu and laboratory tests for Evaluation of Seepage at Dam site

Borehole logging is done at a site in order to perform direct gathering information about strata occur below the ground surface. This enables site engineers/engineering geologists to have a real data regarding rock composition, strata and soil layers (USBR, 1998). Besides to this, the characteristics of soils and rock in terms of porosity, permeability, depth of ground water, presence of seepage and leak condition in the area of under investigation. Structural weakness exploration is another work done through the borehole. Water tightness condition assessment of the dam foundation requires understanding the permeability of rocks and soils composing dam foundation and abutments. To evaluate permeability condition of the site insitu falling head and packer tests were carried out at proposed dam site. Degree of classification of permeability of the foundation and abutments were characterized by adopting classification of degree of permeability suggested by Bell (2007) for soil mass and adopting permeability degree classification suggested by Lashkaripour and Ghafoori (2002) for rock mass (table &) respectively.

Table 2.1 Permeability degree of soils from engineering geology Second edition Bell F.G (2007).

Coefficient of permeability(cm/sec)	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	

The borehole was kept fully cased above a test section length (L). The installed casing was flushed with clean water to remove the remaining drill cutting or mud. Falling head permeability testing was then performed by filling the borehole with clean water to the top or the near surface of the casing and measuring the fall the water level from the top of the casing to the dynamic water level in boreholes. The water head drawdown in boreholes was recorded for the 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 analysed using proper format and standards. The general borehole variable head permeability computation formula provided in (BS5930, 1999) was adopted for determination of tested section permeability values. The result was low to very low permeability on residual soil mass according to Bell F.G., 2007 and impervious to semi-pervious (USBR) classification.

$$K=\frac{A}{F(t2-t1)}\ \mathrm{Loge}\frac{H1}{H2} \quad \text{--------------------Eqn ------------------2.1}$$

Where K= permeability of soil.

H_1 = the variable head measured above the ground water table at time t_1 after commencement.

H_2 = the variable head measured above water table at time t_2 after commencement of test.

A= the cross-sectional area of borehole

F= Intake factor (Hvorsley, 1951) which is given by the following relation for the test condition being in borehole cased above a test section length (L).

$$F=\frac{2\pi L}{Loge^{(\frac{L}{D}}+(\frac{L}{D})^2)} \text{-----------------------------Eqn -----------------------2.2}$$

Table 2.2 permeability Classification of Rock Mass from Lashkaripour and Ghafoori (2002)

Lugeon value from packer test	Degree of permeability of rock mass
>60	Very High
30-60	High
10-30	Medium
3-10	Low
<3	Impervious

According to Lashkoripour and Ghafoori (2002) the result were ranged from impervious to very high degree of permeability.

The Tests procedure they performed was as follows.

- ✓ Borehole was drilled to the desired depth of test section
- ✓ Washing of test section was performed in order to remove drill cuttings until the returning water from borehole appeared clean.

- ✓ Lowering to proper size packer to the top of testing section and inflated with required sealing pressure magnitude.
- ✓ Water was applied into the test section with appropriate and regulated pressure.

Recording of flow rates at every two minutes' interval was conducted. The resulting lugeon values of tested sections for each run pressure were computed on based on relation given by Houlsby (1976).

Lugeon unit (Lu) $=\frac{Q}{PL}$*10 bars ------------Eqn 2.3 where,

Q= Average water intake in L/min

P= Total testing pressure in bar

L= Length of tested section

2.4.2 Geophysical subsurface study

Application of geophysical methods in assessment of seepage potential of embankment dam is the technics that widely used over the world. Appropriate geophysical tools are used to identify seepage zones and pathways through these dams (Nwokebuihe, 2017). Among the geophysical methods, Electrical Resistivity (ER)method is the best for dam investigation (Sjödahl et al., 2010 Panthulu et al., 2001; Karastathis et al., 2002; Kim et al., 2007;Lin et al., 2013) as cited in(Raji & Adedoyin, 2020) .The two dimensional electrical resistivity, 2D-ER is a non-destructive high resolution geophysical method that is the most suitable for dam investigation and monitoring (Raji & Adedoyin, 2020).This method is effective in terms of cost and usually applied to evaluate the competence of foundation rock before dam construction and also for safety test after dam construction (Raji & Adedoyin, 2020).According to (Raji& Adedoyin, 2020),The criteria of using electrical resistivity method for testing rock material is to test for ionic transmission through moisture present in cement voids or weathered section of the foundation rock. Highly permeable zones such as weak rock foundation, fractures, holes, voids and the likes are indicated by low resistivity (high conductivity). In contrast, the competent foundation rock with no void or defected structures are indicated by high resistivity value (low conductivity) because of lack of moisture content. Kayode et al. (2018), the resistivity of earth materials is known by conducting electrical current into the ground and measuring the resulting potential difference.

2.5 Seepage Controlling Methods through Dam foundation

Seepage is one of major problem that usually affects the embankment dam. This problem may be from embankment itself or from foundation of the dam. A site engineer puts decision to reduce this problem after deciding detail investigation. The decision is made as per as the site specifically. In order to obtain safe and stable dam during its service time, assessment of the problems within the site and controlling these problems are pre-requisite of the dam site. Geological structures such as cracks, joints and solution cavities are usually associated with seepage problems. Seepage control is used to prevent failure due to uplift pressure, piping through foundation sloughing and material erosion. Therefore, to choose the treatment techniques of seepage for different dam site there are important key point's inconsideration. These points are:

- ✓ Effectiveness of the selected techniques as per as the site condition.
- ✓ Cost issue consideration
- ✓ Availability of raw materials used for application of the chosen method.
- ✓ Trained man power to apply the techniques effectively
- ✓ Durability and etc.

Dam foundation sometimes highly susceptible to seepage. Highly weathered rocks, fractures, highly jointed rocks and other geological structures such as faults are cause to seepage of the foundation of the dam. To control these problems world researchers, propose different methods to reduce the seepage force path way based on the points listed above. The criteria to select the seepage control method for particular dam and foundation, compare and contrasting the merit and efficiency of different methods by means of flow nets or approximates is done (Gholilou, 2006). The primary purpose of controlling seepage controlling through a dam foundation is to prevent the foundation materials from piping and washing away which could result in structural failure due to loss of support (Rezagholilou & Khorasani, 2007). Also seepage controlling is used to reduce foundation uplift pressure.

Some seepage controlling methods used by researcher in worldwide in order to treat foundation seepage are curtain grouting, consolidation grouting, cut off trench, upstream impervious blanketing, diaphragm wall and etc.

2.5.1 Curtain grouting

Curtain grout is the barrier used to protect the foundation of a dam from seepage and it can be made either during initial construction or during repairing existing dam. The purpose is to strengthen the foundation of the dam. The curtain grouting is effective in condition where high permeability from single row to 3, 5 even more based on complexity of foundation materials within acceptable depth (Elgadir, 2019). The process is governed by lugeon test values before and after grout installation.

2.5.2 Consolidation grouting

Consolidation grouting is an engineering technique that done by injection of grout into discontinuity of rocks in order to improve the engineering properties of rock mass. The properties of rocks improved through this mechanism are permeability, deformation and strength of rock mass. This improvement technique is used to improve the properties of rock mass by enhancing durability and strength as well as for closing the fluid pass way in dam, tunnel and other projects.

2.5.3 Cut off Trench

Cut off trench is a type of seepage controlling method usually used when the dam foundation consist a thick deposit of pervious soil materials. This method is done whether to make complete cut off(compacted backfill trench, slurry trench or concrete wall) or allow certain amount of seepage in controlled way (partial cut off, upstream impervious blanket, downstream seepage berm, toe trench drainer relief wall).Factors which govern to use the seepage controlling methods listed above are: resistance of foundation with respect to piping, economic, water condition and amount of clay sized materials in suspension in the river(Sherard,1968).Partial cut offs are effective only when they extend down into in an intermediate stratum of lower permeability and this stratum must be in continuous across the valley of foundation to ensure three dimensional seepages around discontinuous stratum.

2.5.4 Upstream Impervious Blanketing

Upstream impervious blankets method should not be used when the reservoir head exceeds 200ft because the hydraulic gradient acting across the blanket may result in piping and serious leakage (Rezagholilou, 2006). Relief wells or toe trench drains are generally required for use with upstream blankets to control seepage / excessive uplift pressure and piping through the foundation. Even though the use of upstream blankets is as mentioned above, the effectiveness of this method is based on their length, thickness, vertical permeability of strata and permeability of soils on which they are placed.

2.5.5 Diaphragm wall

A diaphragm wall is a structural concrete wall constructed in a deep trench excavation either cast insitu or using precast concrete components. This method is used as structural retaining elements in civil engineering applications and it acts as permanent foundation walls in cut and covering the structures or deep basements (ASCE, 2012).

2.6. Soil and Rock Foundation of Embankment Dam

Foundation is a substructure element which transmits the structural loads to the earth safely. In case foundation materials should be strong enough to support the structures occurs above it. That mean the materials should not be undergone overstress and deformation which cause to settlement of the structures. So foundation of the dam must be within allowable bearing capacity, allowable settlement, considering shear failure and etc.

2.6.1 Soil Foundation of Embankment Dam

The consideration given to soil foundation at dam site is in situ strength, bearing capacity, permeability and soil types. Excavation of undesirable materials by stripping and removing the topsoil, boulders, organic materials and highly compressible soils are process required during preparation of soil foundation.

Standard penetration test (SPT) through the boreholes performed to measure its natural consistency of the soils based on different standards and corrections. According to Terzaghi and Peck (1967), SPT, Consistency and uniaxial compressive strength correlated as follows (table 2.3). Measurement of field penetration resistance per number of blows (N) was considered the number of blows required for the last 30 cm rod in to the soil stratum. It was refused when no observation of penetration in 10 successive blows. More than 50 blows for the any of the three penetration lengths (15cm) or sum of blow counts of 45cm penetration required more than 100 blows. The testing is discontinued when the N value is over 100 and it is called refusal point (Rahman, 2019).The blows recorded every 15cm penetration. However, the measured penetration resistance (N) values were not directly used for geotechnical evaluation because it was assumed that it could be affected by different correction factors. Hammer efficiency, diameter of borehole, drill rod length, type of sampler and overburden pressure were the main correction factors to be considered during using the results of SPT for geotechnical purpose.

Table 2.3 show correlation between SPT N with Consistency and unconfined compressive strength.

Consistency	SPT value N	Estimated Unconfined compressive strength(UCS) by Kpa
Very soft	<2	<25
Soft	2-4	25-50
Medium	4-8	50-100
Stiff	8-15	100-200
Very stiff	15-30	200-400
Hard	>30	>400

Table 2.4 Correction for Standard Penetration Test (SPT)

SPT Hammer Efficiency		
Hammer Type	Hammer Release Mechanism	Efficiency E_H
Automatic	Trip	0.700
Donut	Hand dropped	0.60
Donut	Cathead turns	0.50
Safety	Cathead + 2 turns	0.55-0.60
Drop/pin	Hand dropped	0.45
Boreholes, Sampler and rod length correction Factors		
Factors	Equipment variables	Correction factor
Borehole diameter C_B	65-115 mm	1.00
	150 mm	1.05
	200 mm	1.15
Sampler correction C_S	Standard Sampler	1
	Sampler with liner for dense sand	0.80
	Sampler with liner for loose sand	0.90
	Sampler without liner	1.0
Rod Length correction C_R	3-4m	0.75
	4-6m	0.85
	6-10m	0.95
	>10m	1
Average effective /overburden pressure(Po) Kpa	Depth of the test section (D)m average unit weight of the soil above test location(g)/kN/m^3	$C_N=[95.76/Po]^{1/2}$
	$N_{60}=E_H C_B C_S C_R C_N N$ $_{60}$averages= $(N1_{60}+N2_{60}+N2_{60}+N3_{60}+N4_{60}+N5_{60}+\ldots..)/n$	

Determination of shear parameters from SPT data value correlation attempted by different researcher (Meyerhof 1956; Peck et al. 1974; Wolff, 1989; Gibbs and Holtz, 1957; Kulhawy and Mayne, 1990; Hatanaka and Uchida, 1996 and Hara et al., 1974) were estimated shear strength of sand and clay soils by using different equation of correlation N_{60} of SPT with friction angle (ϕ') and undrained shear strength (C_u) parameters. For cohesive soils angle of friction is zero (Hettiarachchi, 2008)

$\phi' = \sqrt{20C_N N_{60}}$ +20 ------------------Eqn.2.4 where,

ϕ'= friction angle of soils, C_N overburden correction factor determined for sand soil and Eqn was estimated by Hatanaka and Uchida, 1996.

$C_U/P_a = 0.29(N_{60})^{0.72}$--------------------Eqn. 2.5 for clay soils

Proposed by Hara et al, 1997. Where Pa is atmospheric pressure=100kpa

C_u = Undrained shear strength.

$\phi' = 27.1 + 0.3N_{60} - 0.00054N_{60}^2$-------------------------Eqn. 2.6 Wolff, 1989 estimated from graphical presentation of the correlation.

$C_u = \frac{1}{\alpha}(\frac{Pa}{8.5}N_{60})$ ------------------------Eqn .2.7 Brown, 2007 estimated for cohesive soils. Where $1/\alpha$ is average = 0.3535 with 0.162 standard deviation.

After clearing undesirable materials by stripping, the foundation surface should be compacted. But if the soil foundation is a silt or clay with high water content and high degree of saturation compaction with heavy sheep foot or rubber- tire roller may disturb it. Instead light weight compaction equipment should be used. The filter gradation criteria of the soil must be done in order to control the seepage failure.

Engineering Soil classification is also important for the soil foundation. There are various classification systems of soils mass for different purpose. Among of these some of them are:

- ✓ Geological classification which is linked with its formation
- ✓ Soil classification by its structures (single-grained, honey-comb and flocculent)
- ✓ Grain size or textural classification
- ✓ AASHTO classification system
- ✓ USCS (Unified soil classification system)
- ✓ Indian standard soil classification(IS)

Unified soil classification is very important soil classification applied in many dam site. It was developed by Casagrande in 1948 and revised by American army corps in 1952 (Arora, 2003, Venkatramaiah, 2006). According to USCS soils categorized in to three groups. These are coarse grained, fine-grained and organic soils. Similarly soils are coarse grained if up to 50% passing Number 200 ASTM Sieve and fine-grained if greater than 50% passing Number 200 ASTM Sieve. The plasticity characteristics of soil are used to distinguish the first two categories while the third category easily identified based on its colour, odour and fibrous nature. The coarse grained soils are divided into well graded and poorly graded based on uniformity coefficient (C_u) and coefficient of curvature (C_c). That means well graded gravel C_u>4, well graded sand C_u>6 and well graded soil C_c=1 to 3. Fine grained soils are also subdivided into low plasticity (L) with liquid limit (LL) <50% and high plasticity (H) with Liquid limit (LL) > 50%.

Properties of soils which helps to assess the engineering behaviour of a soil and which assist in determining its classification accurately are termed as index properties. For coarse grained soils the main index properties are particle size and relative density while Atterberg limits and consistency for fine grained soils (Arora, 2003). Index properties of soils divided in to two which are analysed as individual grain particles and as collective soil mass aggregate properties. The properties of individual particles mainly based on its grain and independent of its formation.

The grain size of each soil has an important influence on the over all of its behaviour. The particle size distribution two separate systems are mainly used. These are Sieve analysis for coarse grained and sedimentation (hydrometer) method for fine grained soils. Sieve analysis test is performed for the coarse grained soils in grain size distribution while Hydrometer method test is conducted to determine the grain size distribution of fine soils. The sieve analysis is done for coarse grained soils with particle size greater than 75 microns which pass through set of sieves. The hydrometer method is conducted for soil's particle size between 75 microns to 0.2 micron fine soils. For particles smaller than 0.2-micron hydrometer method is not applicable instead x-ray diffraction is recommended (Arora, 2003).

Atterberg limits are basically defines the properties of soils relative to its consistency which explains the state soils based on the laboratory test (liquid, plastic, semi-solid and solid).

These categories are based on the presence of water in soils. Liquid limit and plastic limit are determined in the laboratory numerically mainly for fine-grain soils and the result is used in the classification of the soil of the site. Liquid limit (LL) is the boundary between liquid and plastic soils while plastic limit (PL) is the boundary between the plastic and semi-solid states. The shrinkage limit (SL) is another Atterberg limit which is the boundary between semi-solid and solid state of fine soils.

Compressibility of Foundation soils

Compressibility of soil is related with the volume change due to void filled with air or liquid and causes for the settlement when the load (Compressive force) is applied on it (Arora, 2003, Venkatramaiah, 2006). Occurrence of compression is due to:

- Compression of solid particles and water in the voids.
- Compression and expulsion of air in the voids.
- Expulsion of water in the voids.

However the influence of compressibility of solid and water is not much significant (Arora, 2003). The structural arrangement of particles for coarse grained soil and degree to which adjacent particles are bonded together for fine grained soils. Some structures of soils such as honey-combed structure is more compressible as it contain more porous and in case of a dense structure the compressibility is negligible because its void space less.

2.6.2 Rock Foundation of Embankment Dam

Sounded or strong rocks which resist differential settlement are good for embankment dam foundation. Top part of rock which is highly weathered and fractured should be removed in order to overcome the problems of differential settlement. In other case cracks, joints and opening beneath the core or elsewhere should be filled with mortar or lean concrete according to the width of the openings. If faults or wide joints on the foundation they should be dug out, cleaned and back filled with lean concrete or treated by another means at least to the depth three times their width. This creates structural bridge over the fault or joint-filling materials and will prevent the embankment fill from being lost into the fault or joint.

Classification of rock mass is very important at dam sit other mega projects. In other word the classification also used to recommend the improvement techniques or support required for the project. Geomechanics/ Rock mass rating (RMR) is one rock mass classification systems developed by Bieniawski, 1976 and revised in 1989. RMR of Bieniawski is based on six parameters to classify the rocks of the project site and it is widely used over the world. These are includes:

- Uniaxial compressive strength of the rock(UCS)
- Rock Quality Designation(RQD)
- Spacing of discontinuity
- Condition of discontinuity
- Ground water condition
- Orientation of discontinuity

Table 2.5 show relation of RMR with shear strength parameters.

Parameter	Assessment Values and rating						
UCS(Mpa)	>250	100-250	50-100	25-50	5-25	1-5	<1
Point Load(Mpa)	>10	4-10	2-4	1-2	For lower UCS preferred		
Rating	15	12	7	4	2	1	0
RQD (%)	90-100	75-90	50-75	25-50	<25		
Rating	20	17	13	8	3		
Discontinuity spacing	>2m	0.6-2m	20cm-60cm	6-20cm	<6cm		
Rating	20	15	10	8	5		
Discontinuity condition	Rough tight	Open<1mm	weathered	Filled gouge<5mm	Gouge >5mm		
Rating	30	25	20	10	0		
Gw condition	Dry	Damp	Wet	Dripping	Flowing		
Rating	15	10	7	4	0		
Orientation of discontinuity	Very favourable	Favourable	Fair	Unfavourable	V. unfavourable		
Rating for foundation	0	-2	-7	-15	-25		
RMR	100-81	80-61	60-41	40-21	<20		
Rock class	I	II	III	IV	V		
Rock Quality	Very good	Good	Fair	Poor	Very poor		
Cohesion(kpa)	>400	400-300	300-200	200-100	<100		
Ø in degrees	>45	45-35	35-25	25-15	<15		

The classification of rocks used to estimate strength and deformation properties of the rock mass. Among many classification of rock mass, Bieniawski rock mass rating (RMR) is widely used over the world in most mega project such as dam and tunnels. Classification could be in terms of strength, permeability and deformation. All rock masses contain discontinuities like joint, shear zones, bedding and contact zones. At Shallow depth, where stresses are low failure of the intact rock material is the minimum and the behaviour of the rock mass is controlled by sliding on the discontinuities (Hoek and Bray, 1981).

Rock quality designation is developed by Deere (1967) to provide a quantitative estimate of rock mass quality from core logs. RQD is the percentage sum of the all the core pieces equal or greater than 10 cm long by core interval length (Jonson, 1988). It is very important parameter to describe the engineering properties of rock mass at the site.

$$RQD = \sum \frac{Length\ of\ core\ pieces > 10cm\ x100}{Total\ length\ of\ core\ run} \text{------------------------------ Eqn 2.8}$$

Weakness zones developed on rock mass are called discontinuities. It influences greatly the properties of rocks such as strength, permeability and deformation. Joints, fractures, beddings, fissures and etc. are some kind of discontinuity. The main factors related to discontinuities are orientation, infilling materials, continuity, spacing, and surface and ground water characteristics. Therefore, presences of these discontinuities control the permeability, weathering, stability, strength and quality of rock masses.

Weathering means decay and changes of earth materials (rocks & soils) from its original to slightly-completely to new form as result of external and internal factors. It occurs in every environment however it is dominant in hot climate. Discontinuities and its infilled materials control the degree of weathering. Degree of weathering is increased with increasing number of discontinuities (Price etal. 2009). The rock type, weathering process and time are other parameters control the degree of weathering. Depth and degree of weathering is essential parameters to limit the stripping of foundation of dam. Presence of water with discontinuity triggers the degree of chemical weathering. Deformation of rock mass means the capacity of the rock to strain under applied loads or in response to unloads on excavations. It is very important for dam site as it is responsible for the failure of the project. Deformation of rock mass is mainly resulted from the closure of discontinuities, elastic and plastic behavior of intact rocks which comprises rock mass. In order to measure the deformation of rock mass, modulus deformation is used (Ed).

Modulus deformation is the sum of deformation occurs with closure of joints in the rock mass under compression(plastic) and occurs with continued stress application after crack closure (elastic) as stated by (Jonson 1988). Similarly, Agrawell etal. 1991 was produced empirical formula relation to determine the modulus deformation by using RMR of Beiniawsk.

2.7. Embankment Dam Construction Materials

Embankment dams are constructed of all geomaterials with the exception of organic soils and peats (USSD, 2011).This is done by consideration of different parameters and condition that should be affects construction materials in different parts of a dam. Selecting appropriate construction material reduces the cost for remedial measures (Rostislav and Magdalena, 2017).USSD committee on embankment dam materials are prepared criteria how to select materials for different parts of dam filling and under what condition the material meets the requirements. Materials required for different parts of embankment dams are evaluated depend on purpose of each part of the dam (USBR, 2012). The properties of core, filter, shell and riprap material should be individually analysed (Sherard, 1967).

When soils are used as construction materials, the investigation must be carried out on the general suitability of both quality and quantity as per as the project requirement. According to (USSD, 2011; USBR, 2012; USACE, 2004) for soil materials the main considerations are index properties, shear strength, compressibility, permeability, relative density, erosion, gradations and liquefaction potentials.

Main parameters analysed in rock fill materials are performing engineering properties of rocks such as gradation, compacted unit weight, compressibility, permeability, shear strength, and deformability and durability (USSD, 2011). Rocks are widely used construction material for embankment dams over the world. According to USSD (2011) rocks are used primarily as dumped rock fill until 1940 in high lifts initially without sluicing with water and later adding water sluicing to aid compaction. Similarly the compaction of rock fill started about 1960 and by 1980 most rock fill materials for embankment were well compacted in lifts not exceeding about 1.5m.

2.7.1 Selection of fill Materials

Construction materials for embankment dam are different for each zoned parts of the dam.

According to USBR, 2012(design standard no.13) the distance of construction materials should be close to the project site as much as possible to minimize the cost for transportation. Generally selection of fill materials for each part of zoned dam based on the requirement of each part as per as its functions.

2.7.1.1 Core Materials of embankment dam

Impervious material is required for core area of embankment dam. The requirement of core zone is availability of those materials within acceptable quality and quantity. Plus economic distance from the dam site and important properties of soils such as permeability, compacted density, shear strength, compressibility, flexibility and erosion resistance are must be considered during its selection. According to Bhrat Singh (1995) material with permeability of 10^{-5}cm/s and lower can be used in core of the embankment dams. Soils with high compressibility must be avoided due to its excessive settlement and cracking high pore pressure can be developed.

Flexibility and erosion resistance properties are very critical for core materials. Flexibility is the ability to deform without cracking while erosion resistance is the ability of soils withstands the erosion action of water leaking through possible cracks (Singh, 1995). Flexibility increases with increase in plasticity index (PI) but for very high values of plasticity index it may be associated with high compressibility.

The two main sources of erosion resistance are cohesion of fines and the resistive action of coarse particles to the flowing water and their tendency to wedge up in the leakage channel. According to Sherard (1963), erosion resistance of materials increases with plasticity index and compacted density. Well graded sand and gravel mixture with enough finer particles to provide imperviousness as described in the following table.

Thickness of core is determined based on available materials, the design transition filter sand seismic condition of the site (USACE, 1994). For materials having a high erosion resistance and good flexibility, small thickness of the core is required. Larger thickness of core is mandatory in areas high seismic activity and in areas of greater probability of cracking (Sherard et al., 1963).

Table 2.6 Suitability of soil for construction of dam (Bharat Singh, 1995).

Relative suitability	Homogeneous Dams	Impervious core	Pervious Shell	Impervious Blanket
Very suitable	GC	GC	SW,GW	GC
Suitable	CL-CI	CL,CI	GM	CL,CI
Fairly suitable	SP,SM,CH	GM,GC,SM,SC,CH	SP,GP	CH,SM,SC,GC
Poor	-	ML,MI,MH	-	-
Not suitable	-	OL,OI,OH,Pt	-	-

2.7.1.2 Shell Materials

Shell is material required to stabilize the dam and to withstand the horizontal thrust exerted by the water impounded behind the dam. The basic requirement of the materials used for this purpose is soundness, sufficient permeability and its shear strength. Shell materials are embankment dam part which is being on either side of core zone to provide the stability to the main by accommodating its weight and to withstand the thrust of the impounded water. These two sides are upstream shell and downstream shell. The upstream shell used to stabilize during end of construction, rapid drawdown, earthquake and loading condition while the downstream shell used as drain controller line of seepage and provides stability under high reservoir levels and during earthquakes.

2.7.1.3 Filter Materials

Pervious materials are used as filters and drains. The basic requirement of use materials as filters drains or drainage blankets are processing such as crushing, washing and blending. The main objective of filter is that, it protects particle migration and allows free drainage movement of water within and above embankment. In filter criteria, filters must retain the protected soil but have permeability greater than the protected soil. It has no particular flow or drainage capacity since the flow will be perpendicular to the interface between the protected soil and filter. Correct gradation, appropriate handling and placing with care to avoid contamination segregation to meet its objective (Cendergren, 1977). Sometimes naturally occurring sand and gravel can be used for filters and drains with little or no processing. For drainage aggregate usually need to be treated. However most natural sand and gravel are highly variable in grading from point to point in a borrow area and they may interbedded with silt and clay which is difficult to remove. In that case more economical is crushing the required sizes from suitable locally available rocks at the site.

Filter selection criteria are proposed by different authors and organizations. Well known filter section criteria proposer are Terzaghi, United State Bureau of Reclamation (USBR), Sherard and Indian filter criteria. Filter criteria stated by USBR, 2011; USSD, 2021 were used to find the limits of base soil and filter. The base soil categorized in to four categories based on percent finer that passed sieve number 200(0.075mm) as follows. In Design fraction finer than the #100 sieve, 0.15mm was recommended (Humphries and Connors, 1989). This resulted in a fine filter with a maximum $D_{15}F$ = 0.5mm and minimum was 0.1mm.

Permeability of filter should be at least 25 times the permeability base soil and this is obtained when D_{15} F larger than 5 times $D_{15}B$(USBR, 2011).So for uniformly to moderately graded sand and gravel size filter coefficient of uniformity(1.5 to 8) and permeability estimated by using next empirical equation (USBR,2011).

$k = 0.35\ (D15F)^2$--Eqn 2.9

Where k= cm/s and D15F is in mm

Table 2.7 Category of base soil for filter criteria as per USBR, 2011; USSD, 2021

% finer than (0.075)	Category	Filter criteria	Description
Greater than 85	I	$D_{15}F \leq 9*D_{85}B$	Fine silts and clays
40-85	II	$D_{15}F \leq 0.7mm$	Silts, clays, silty sand and clay sand
15-39	III	$D_{15}F \leq 0.7mm + (\frac{40-A)(4xD85B-0.7mm)}{25}$ A=% passing No. 200 sieve for A<4xD85B ,use 0.7mm For dispersive use 0.5mm	Silty and clayey sands and gravel
Less than 15	IV		Sands and gravel

Table 2.8 Limits of $D_{10}F$ and $D_{90}F$ for preventing segregation (USSD, 2021)

Base Soil Category	If D_{10} is(mm)	Then Maximum D_{90} is,(mm)
FOR ALL CATEGORIES	<0.5	20
	0.5-1.0	25
	1.0-2.0	30
	2.0-5.0	40
	5.0-10	50
	>10	60

Terzaghi's Filter Selection Criteria (1930)

Filter selection criteria proposed by Terzaghi shortly described as follows.

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

$$\frac{D15\ of\ filter}{D85\ \ of\ protected\ layer} < 4 \text{ to } 5$$

2) The 15% size of filter material ($D_{15}F$) must be at least 4 or 5 times the 15% ($D_{15}B$) of the protected soil in order to adequate permeability.

$$\frac{D15\ of\ filter}{D15\ of\ protected\ layer} > 4 \text{ to } 5$$

Other Requirement of good filter He listed as follows:

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

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

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

iv) If the base materials ranges from gravel over 10% > 4.5mm to silt over 10% passing 75m, the base material should be analysed on the basis of the gradation fraction smaller than 4.75m

USBR Filter Selection criteria (1955)

The filter investigation was done by Karpoff, 1955 in the USBR central soil laboratory using sandy silts adjacent to uniform fine to medium sands as base materials. The USBR filter selection criteria was based on experiments and it improved many dams in U.S.A. The other requirement is the same as in the Terzaghi's criteria as shown in next table.

Table 2.9 USBR Filter Design Criteria

S.No	Filter material characteristics	Ratio $R_{50} = \frac{D50\ of\ filter}{D50\ of\ base}$	Ratio $R_{15} = \frac{D15\ of\ filter}{D15\ 0f\ base}$
1	Uniform grain size distribution uniformity coefficient Cu $=\frac{D60}{D10}$ = 3 to 4	5 to 10	-
2	Well graded to poorly graded(non -uniform) sub-rounded grains	12 to 58	12 to 40
3	Well graded to poorly graded(non-uniform) angular particle	4 to 30	6 to 18

Sherard's Recommendation for Filter Selection Criteria

Sherard recommended on filter selection criteria as follows:

i) The filter is successful in its function of arresting particles migration if,

$\frac{D15\ of\ filter}{D85\ of\ base} < 9$

ii) The size of the pore channel which governs permeability is determined by the size of finer filter particles and it is represented by $D_{15}F$ size.

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

Indian Standard Code for Filter selection

Indian standard code on filter selection criteria's are recommended as follows:

i) $\frac{\text{D15 of filter}}{\text{D85 of base}} < 5$

ii) $\frac{D15\ of\ filter}{D15\ of\ base} > 4$ and < 20

iii) $\frac{D50\ of\ filter}{D50\ of\ base} < 25$

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

Requirements for a good filter are;

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

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

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

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

By considering these all factors Best filter is decided.

2.7.1.4 Materials for Riprap

Materials required for slope protection which is known as riprap rocks fragment must fulfil two main requirement (USBR, 2012).The first requirement is that, the rock fragment should be produced as suitable sizes, shapes, gradation and weights as possible while the second requirement is the fragment should be tough and durable enough to with stand transportation and placement without breaking. Also it should be withstand weathering, wave action and other destructive forces.

Riprap is used in both upstream and downstream of embankment to improve the erosion action of wind, rain water and wave action of reservoir. Different types of ripraps are used based on site specific condition. These are dumped stone, hand placed and soil cement riprap.

2.8 Previous Work

In previous study WWDSE (2016) covers all parts of Fato Dam including reservoir, foundation, spillway, construction materials and etc. On foundation part the study showed that rock foundation had high lugeon values (permeability) while soil foundation indicated from very low to low permeability. Geophysics techniques done on the area indicated that the area was affected by weathering and fracturing of basaltic rocks available at the site. Therefore, the rock foundation especially in left abutment seepage problems .They recommended further investigation and analysis on both construction material and foundation of the the dam.

So current work was done on foundation and construction materials for further analysis. Seepage analysis of foundation and dam body as whole was done by using plaxis software. Simulation of construction materials by using standards of gradation was performed.

2.9 Background of Plaxis 2D Software

Plaxis 2D software is engineering software programs applied for the purpose of geotechnical analysis of site. Plaxis 2D is a types finite element (FE) programs performed for different engineering activities such as deformation, stability and flow analysis. Geotechnical modeling by plaxis 2D software is done by plain strain or an axisymmetric. This programs include two user interface programs (INPUT and OUTPUT) to enable the user to quickly generate geometry model and finite element (FE) mesh. The geometry is based on the representative vertical cross-section. Input program is a prep-processor in which define of problem geometry, create mesh and define calculation phase while Output is post-processor that used to perform calculation in 2D view dimension and plot cross section or graphs.

So for present study I used this software to analysis the seepage of Fato dam foundation and its body with and without remedial measures.

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CHAPTER III General Overview of Study Area

3.1 General

This part mainly focuses on the overall conditions of the project area. The parameters that will be discussed in this chapter are geology, drainage, climate, morphology, social settlement, and the land use- Land Cover of the study area. These mainly affect the site's suitability for the proposed project. A geological investigation is used to analyze the area concerning geological hazards, tectonic activities, lithological composition, the suitability of the site, and the suitability of the construction materials. This activity is done to mitigate the risks that come from geo-activities and to enhance the sustainable use of resources for different purposes.

3.2 Regional geological setting

According to Ethiopian geology, the Fato dam site comprises rocks from Jurassic to Quaternary age (Tefera et al. 1996). The report of the Akaki Beseka map sheet by Beshawered (2010) also described the geological formation of the area, which comprises pre-rift (Adigrat sand stone), main-rift units (Babich-Guder basalt, Wechecha trachyte), and quaternary surficial deposits.

Adigrat Formation

The Adigrat formation includes the whole succession of clastic rocks resting on the Precambrian basement and overlain by Antalo formation

Alajae Formation

Alajae formations are other units that consists aphyric flood basalts associated with rhyolite (ignimbrite) and subordinate trachyte. This formation makes the bulk of the volcanic succession on the North western and South Eastern Plateau. On the north-western plateau, the Alajae rests conformably on the Aiba Basalts but in some places (e.g. Kesem gorge, Mugher canyon and in most outcrops on the southeastern plateau) it directly overlies on the Mesozoic sediments.

Tarmaber-Megezez

Tarmaber-Megezez formations represent Oligocene to Miocene basaltic shield volcanism on the North-western and south-eastern plateaus. This unit is the younger shield volcanoes. The central type Tarmabar formation basaltic formation was followed by fissural eruption particularly along the escarpments of north-western and south-eastern plateaus and they are alkaline in nature.

Quaternary plateau

According to Beshawered (2010), the geology of the area comprises pre-rift (Adigrat sand stone), main-rift units (Babich-Guder Basalt, Wechecha Trachyte), and quaternary surficial deposits. Babich-Guder basalt is a dark-colored, fine grained rock mainly composed of plagioclase, pyroxene, and opaque minerals. It has fresh, fine grained outcrops but in some places it also has a vesicular variety, which in some places is filled with secondary minerals like zeolite or calcite.

Wechecha trachyte is composed of trachyte and pyroclastic materials. The trachyte is coarse-grained, light gray, brown-grey to dark-grey in fresh samples, to pinkish yellow to reddish brown in weathered ones. The pyroclastic materials are exposed near Wenchi and Dendi Lakes, and this unit is result of pre-caldera formation. The formation contains ignimbrites associated with ash flows, pumice falls, and surge deposits. This formation generally lies over flat topography and marshy areas.

The older basaltic lava flows are separated from the younger Quaternary volcanic rocks of the Ambo area by the Addis Ababa-Ambo fault escarpment. The southern side of the major fault escarpment, including Ambo town is underlain by quaternary volcanic rocks. The most structural features of the area are the Addis Ababa-Ambo-Ghedo fault zone, which is east-west oriented, and the NNE-SSW trending Guder lineaments, which cross one another almost at a right angle near Ambo town (Tsegaye Abebe, 1993).

The Quaternary volcanic rocks in the area have resulted in the central type of eruption, mainly forming Wenchi and Dendi volcanoes at the southern part of the study area (Mohr, 1970). The E-W running fault has given rise to the exposure of the Mesozoic sediments and the juxtaposition of the sediments with the volcanic cover.

3.3 Local Geology of the Fato Dam site

The Fato Dam site is situated in the central Ethiopian plateau comprising of tertiary volcanic and quaternary superficial sediment deposits. The volcanic rocks of the site are tertiary basalt and pyroclastic deposits. The dam site area mainly comprises units of basaltic lava flows overlain by thin to thick residual soil masses formed from prolonged weathering of parent basalt rocks.

Three types of basaltic rocks compose the project site. These are vesicular basalt, amygdaloidal basalt and aphanitic basalt from top to bottom. Most of the basalt available in the study area experienced joint and fractures intensively. A vesicular basalt unit was outcropped along the river channel at the Fato dam site, and it has extensions into the abutments. It was also covered both upstream and downstream of the dam site. It was reddish to dark gray with dislocation and with filling materials. Its top part, as indicated by the borehole log and geophysical survey results, was completely weathered, fractured, and weak in strength, especially at the left abutment. The amygdaloidal basalt vesicles are filled with calcite, silt, and clay materials. It is dark to gray in color, weathered, fractured, and medium to strong in strength. Aphanitic basalt was also dark gray in color, slightly weathered, fractured, and had strong healed joints filled with calcite materials.

Pyroclastic rocks found in the study area comprise both pyroclastic flow (ignimbrite) units and pyroclastic fall deposits (pumice, ash, and scoria). These pyroclastic are found in the north-east, south-east, and southeast of the Fato dam site.

The quaternary superficial deposits in the present study area are residual soil masses formed from weathered basalt rocks underlying it and recent alluvial deposits along the Fato River (FDBH4 at the intake area). The residual soil covers thickly and dominantly the south and southwest of the project area. The upper portion of the residual soil (0.5–1 m) is rich in organic matter and plant roots, forming a dark brown top soil layer. The soil mass of the area was dominated by silt and clay with sand (silty clay, clayey silt, and sandy silt) and an increased inclusion of angular basalt gravel grains near contact with the parent rocks (borehole logging data). The thickness of the soil at the site varies from 8m (left abutment) to 33m (right abutment) at the dam site.

Figure 3.1 Basalt rock exposed in river bed.

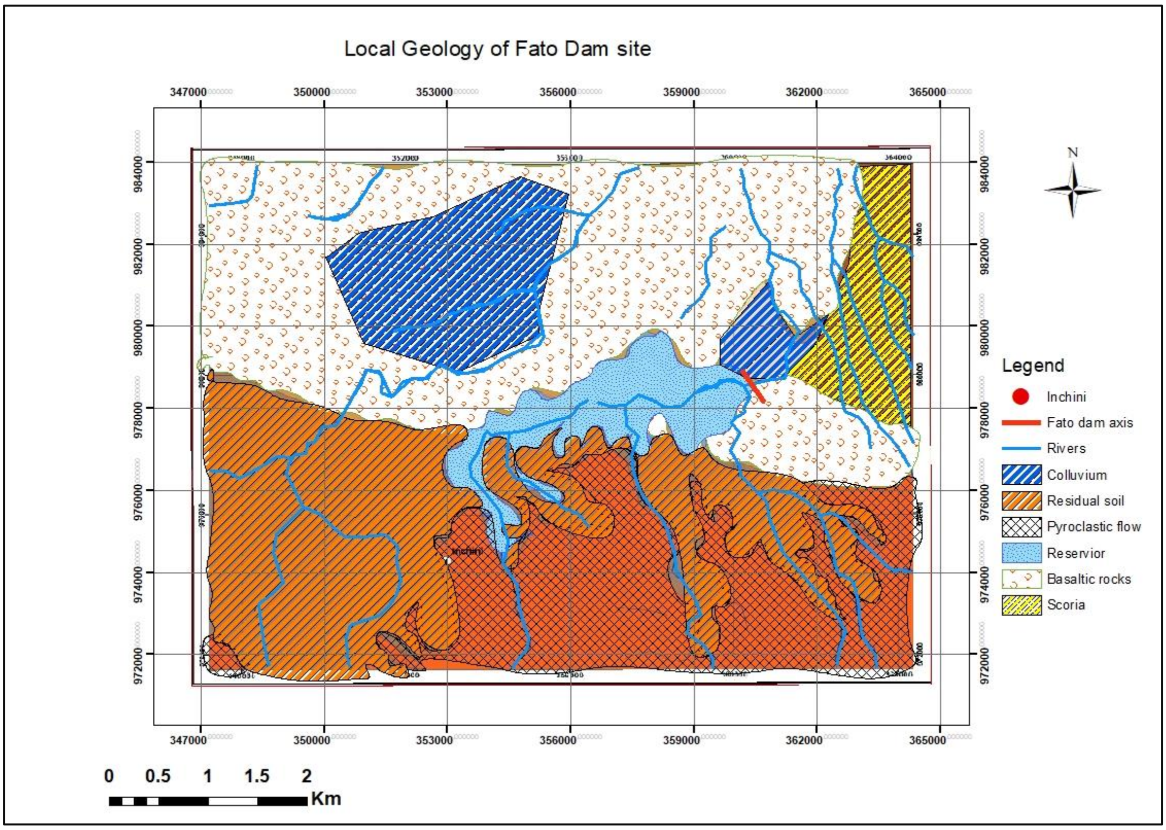

Figure 3.2 Geological map of Fato dam site

3.4 Geohazard Assessment of the project Area

Geohazard is the phenomena which may cause damage to the shape of land or to the lives of human and other living things. It may be happen deep inside the earth crust and also on earth's surface. The outer land shape is affected by different natural and manmade activity. Some natural hazards that may affect shape of the earth are earthquake, volcanic, Landslides, flooding, avalanche and etc. These hazards are occurs naturally however, sometimes human activities such as overgrazing, agricultural activity on elevated area, firing forest and etc. are triggering factors for these hazards.

3.4.1 Seismic Activity Assessment

The primary factors considered in seismic hazard assessment of dam site are regional factors (seismic history, seismology and seismo- tectonic interpretation), local factors (local geology, hydrogeology, geophysical study, geotechnical data and extraction of natural resources) Wieland and H. Fan (2004), ICOLD Bullet 72 Revised (2010) and Wieland (2008). Regional geologic history, physiographic, structures, fault displacement and rate of subsidence are some regional factors considered during site investigation for dam project.

The Main Ethiopian Rift (MER) is part of the East African Rift and Afar Depression. These places are considered as locus of volcanic and seismic activities because they represent an extensional tectonics in action. The MER meets the two oceanic rifts such as Red sea and Gulf of Aden in the Afar depression/triangle forming three rift Triple junctions. A comparison of rift structures and topographic relief in MER reveals that tertiary volcanoes and flood basalts also contribute to the high amplitude and short wavelength aspects of topography, particularly in the vicinity of the rift system.

The Fato dam site is located in the western plateau, which is far from the active rift however, we can't rule out the occurrence of the strong earthquakes close to the lift margin that can induce damage elsewhere. Hypocenter depth of well constrained events is 5-7km from modeled earthquake in the main Ethiopian rift (from project design report).The approximate elastic plate thickness in Afar and main Ethiopian rift possibly indicating the depth to brittle –ductile transition zone in this part of the rift. The shallowness of the depth estimates agree with the macro-seismic report available from a wide area reported for Hosanna and Yirgalem earthquakes in the southern Ethiopia. Further the potential future shallow crustal deformation may cause significant loss of life and properties unless measures are taken in improving building standards of houses, roads, bridges and dam. Accordingly, project owner for Fato dam site .The results of assessment are summarized in the following table (3.1) conducted site specific seismic hazard assessment.

Table 3.1 Summaries of GA Fato dam site

Return period in yrs	Ground motion Amplitude in % of g for Boore-Joyner-Fumal(1993,1997)					
	Period=0.2sec		Period =1.0sec		Period=2sec	
	Rock	soil	Rock	Soil	Rock	Soil
50	6.31	6.56	3.78	4.14	2.65	2.89
100	7.37	7.70	4.29	4.72	2.98	3.27
200	8.60	9.05	4.86	5.37	3.35	3.70
500	10.56	11.19	5.74	6.38	3.91	4.35
1000	12.32	13.14	6.51	7.26	4.40	4.92
10,000	20.48	21.86	9.87	11.19	6.52	7.39

For the soil of site, an average thickness of 30 meters with the shear wave velocity of 310m/s was adopted for all periods considered according to Boore et al, 1997. The PGA values in table (3-1.) were relatively low due to the far location of the planned dam at Fato site from the active rift margin, which still needs design consideration. The main and south-western Ethiopia rift is the one contributing most to the hazard at that site. Dams are normally allocated a hazard category using the method of ICOLD bulletin 72, which takes into account reservoir capacity, dam height, number of persons at risk and potential downstream damage. U.S Army Corps of Engineer the manual number EM-1110-2-6053 recommends Pseudostatic coefficient shall be two third of the horizontal peak ground acceleration.

The important implication here is that the location of Fato dam site is far from the currently active rift margins but it can be affected by distant and damaging earthquakes that may source from the rift margin border faults. Accordingly, for the 0.2 second period and soil site the 67% PGA values of 1,000 and 10,000 years considered as Operation Basis Earthquake(OBE) and Safety Evaluation Earthquake(SEE) values, there for slope stability analysis of Fato dam the PGA values of 0.09g and 0.14g were considered as the horizontal ground acceleration(α_h) for OBE and SEE, respectively. In case of the vertical ground acceleration (α_v) for both for OBE and SEE are considering 50% of the horizontal acceleration.

3.4.2 Mass movement

Mass movement is Movement of materials downhill under the influence of gravity. This movement may cause either by natural process or human activity. Landslides, rock fall, creep, erosion and the likes are examples of such type of movement. Gravity, slope, water condition, Rain fall, vegetation, human activity and type of regolith/parent rocks are factors that mainly affect the movement of material down ward of the slopes.

However in Fato dam site mass movement is not more endanger but small scale slide of weathered materials are occurred in some hilly area. The elevated area covered with vegetation but when vegetation removed the erosion may happen in the down of the elevated land. This may trigger the landslides if appropriate measures do not be taken.

3.5 Physiographic of the Fato Dam site

Topographically the area was characterized by high mountainous relief hills-central highlands and steep severely dissected side slopes and plateau. The Northern part of the area was marked by an East-West oriented regional fault. The highest peak of the site was the village of Hole (2498ma.s.l) located at the Northern sector. The area contains rivers such as Bello (Upper Guder), Fato and Melke were perennial and drain flow to the Northeast.

3.6 Climate Condition of Fato Dam site

The four nearest Rain gauge stations to the project area were Inchini,Ambo,Guder and Gedo gauging stations while Inchini station was much more representative of the reservoir. Those stations had a relatively long record covering from 1954 to present with missing values here and there. The stations location in association with the length of all climate data records presented on next table. In those station climates data such as precipitation (rain fall), Humidity, wind speed, sunshine and temperature (max. and Min.)Were explained. Mean annual rain fall of the site was about 680mm to 1200mm. (appendix 1)

Table 3.2 Location of rain fall observation station near to the project area.

S.N	Station Name	Long. E	Lat. N	Elevation(m)	Periods	No. missed data by no of month (%)
1	Inchini	$38^0 22'$	$08^0 19'$	2690	1976-2019(43yrs)	25(6%)
2	Gedo	$37^0 26'$	$09^0 03'$	2500	1954-2019(65yrs)	57(8%)
3	Guder	$39^0 47'$	$08^0 57'$	2002	1964-2019(55yrs)	10(1.4%)
4	Ambo	$37^0 52'$	$08^0 58'$	2050	1954-2018(64yrs)	64(9%)

3.6 Vegetation and Land use practice

The economical use of the land in the Fato dam area was mainly agricultural activity. The study area contributes the highest proportion of agricultural production and there was a high potential for rain fed and irrigated agriculture. Vegetation cover was also better compared to the other highlands of parts of the country. Some farmers in the area practiced of retaining the scattered trees on the crop land in addition to patches of forest, wood and shrub vegetation. However, at the present most of the area were under threat of soil erosion that had constrained the agricultural potential of the area coupled with other limiting factors. Hence there was the evidence of sheet and rill erosion of soils, rock outcrops and gully erosion on steep cultivated land, open shrub land and grazing land and the like existed in the project area.

3.7 Ground water and surface water condition

Ground water is one of water resource found below earth surface in aquifer area. However, in engineering project its occurrence in shallow depth is not mandatory. As it observed from piezometer installations of the foundation site of Fato dam the ground water table occurred greater than 7m from the surface. The installations were done during core drilling in two boreholes of foundation area Left abutment (FDBH3) and Centre of dam axis left bank side of river (FDBH2). According to the final geotechnical feasibility report record of ground water level was done twice a day (morning and afternoon) for first two weeks since installation and once afterwards.

Surface hydrological activity of the Fato dam site linked to the mean annual rain fall of the site and rivers found in this area. Activity of this hydrological system the influence of metrological data was major parameters. Ambo, Guder, Inchini and Gedo stations were record the rain fall data and temperature (max, min) however, relative humidity, sunshine and wind speed not fully recorded data. The Fato gauging station was occurred at about 3km upstream from the Fato dam axis. Based on flow, site condition and design parameters Fato dam was designed with dam axis with measurement listed in the following table.

Table 3.3 General design layout of Fato dam site

Parameters	Design specified measurement(m)	Elevation(m)
Elevation of dam crest/Top of dam	40.1	2432
Crest width	10	
Crest length	283	
Maximum water level (MWL)	38.8	2430.2
Reservoir full level (RFL)	37.6	2429
Normal Free board	3	2429-2432
Minimum free board	1.8	2430.2-2432
Lower foundation level		2389.9

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CHAPTER IV METHODOLOGY AND MATERIALS USED

4.1 Methodology and Data Collection Procedures

The methodologies used throughout this paper were integrated geophysical and engineering geological approach would be applied to achieve the objective this research. This research work was done in three phases: pre-field work, Field work and post field work. In this chapter the flows of ideas from initial to final sequence was described.

4.2 Pre-field/desk work

Desk study includes that identification of research problem and assessment of literature review on the problem was identified. Collection of secondary data from different sources was done in this stage. Journals, published and unpublished MSc thesis, geological map and many reports related to the problem were reviewed. Fato dam site geotechnical report and final detail design report (WWDSE, 2016/17) was collected to understand the gaps of the project under construction.

4.3 Field work

The field work was done at Fato dam site in order to collect information concerning on geological, and engineering geological of surface and subsurface of the site. The site specific analysis was also done on the construction materials of the Fato dam site. The field activities done were:

- ✓ Surface outcrop description and measurement
- ✓ Collection of engineering rock classification data to analysis the strength and deformability characteristic of the rock mass found at the foundation area.
- ✓ Schmidt hammer test of rock mass
- ✓ Collection of photographs of the site and exposures.

4.4 Post field work

After the completion of the field work, organization of the data collected from field and also secondary data from different sources were collected. Engineering classification of rocks and soils of foundation and borrow area were done based on site investigation and secondary data (core log) of the project.

Table 4.1 Detail of soil and rocks tests done on each part of the foundation

Location		Tests	No. Sample	Type of sample
Left Abutment(FDBH1)	E 360369 N 979608	SPT	4	Soil
		Falling head permeability	3	Soil
		Packer Test	4	Rock
		Grain Size analysis	4	Soil
		Atterberg Limits	4	Soil
		Gradation	4	Soil
		Bulk Unit weight	4	Soil and rock
		Free Swell %	3	Soil
		UCS	2	Rock
Centre of dam axis (FDBH2)	E 360325 N 979541	SPT	1	Soil
		Falling Head permeability	4	Rock
		Packer test	5	Rock
		Specific Gravity	4	Rock
		Bulk Unit Weight	2	Rock
		Water Absorption	2	Rock
Right Abutment(FDBH3)	E 360255 N 979409	SPT	14	Soil
		Falling Head permeability	5	Soil
		Packer test	1	Rock
		Grain size analysis	15	Soil
		Atterberg limits	15	Soil
		Gradation	15	Soil
		Specific Gravity	2	Rock
		Bulk Unit weight	19	Soil and rock
		Water absorption	2	Rock
		Free swell %	4	Soil
Intake Area(FDBH4)	E 360265 N 979585	Falling Head permeability	3	Rock
		Packer test	4	Rock
		UCS	1	Rock
		Bulk Unit Weight	1	Rock
		Water Absorption	1	Rock

Then data processing, analysis and interpretation were done in order to produce detail engineering geological map of dam foundation and construction materials. Construction materials proposed for different parts of the dam structures were identified from the area nearest to the site as much as possible (table 4.2). Finally, conclusion and recommendation were discussed based on the present study findings.

Table 4.2 Detail of tests done on soils and rocks for construction materials.

Test pit (TP)	Sample ID	Coordinate	Tests	No. Tests	Used for	Distance from dam site(km)
Clay borrow	CBTP-01	E360857N980031	Grain size	3	Core zone	0.6
	CBTP-02	E360994N980040	Specific Gravity	6		0.7
	CBTP-03	E360651N980145	Atterberg limits	3		0.6
	CBTP-04	E360399N979376	Free swell	2		0.6
	CBTP-05	E360249N980382	Permeability	5		0.1
			OMC (%)	5		
			MDD(g/cc)	5		
	FCBTP-01	E360537N978765	Organic content	5		0.8
			Shear strength	6		
			Gradation	3		
Sand borrow	UGSS-01		Grain size	7	Filter and Transition zone	30
				7		
	UGSS-02		Specific Gravity	4		
	UGSS-03					
	UGSS-04		Bulk unit weight	7		
			Gradation			
Rock and crushed aggregate	FSRQ1 BDS1	E 355223 N 982250	Specific gravity	3	Rock fill and Riprap	Near to the site
	FSRQ2 RS2	E 357121 N 981469	Bulk density	3		
	FSRQ3 RS3	Fato river from Surface	Water absorption	3		
			Point load	3		

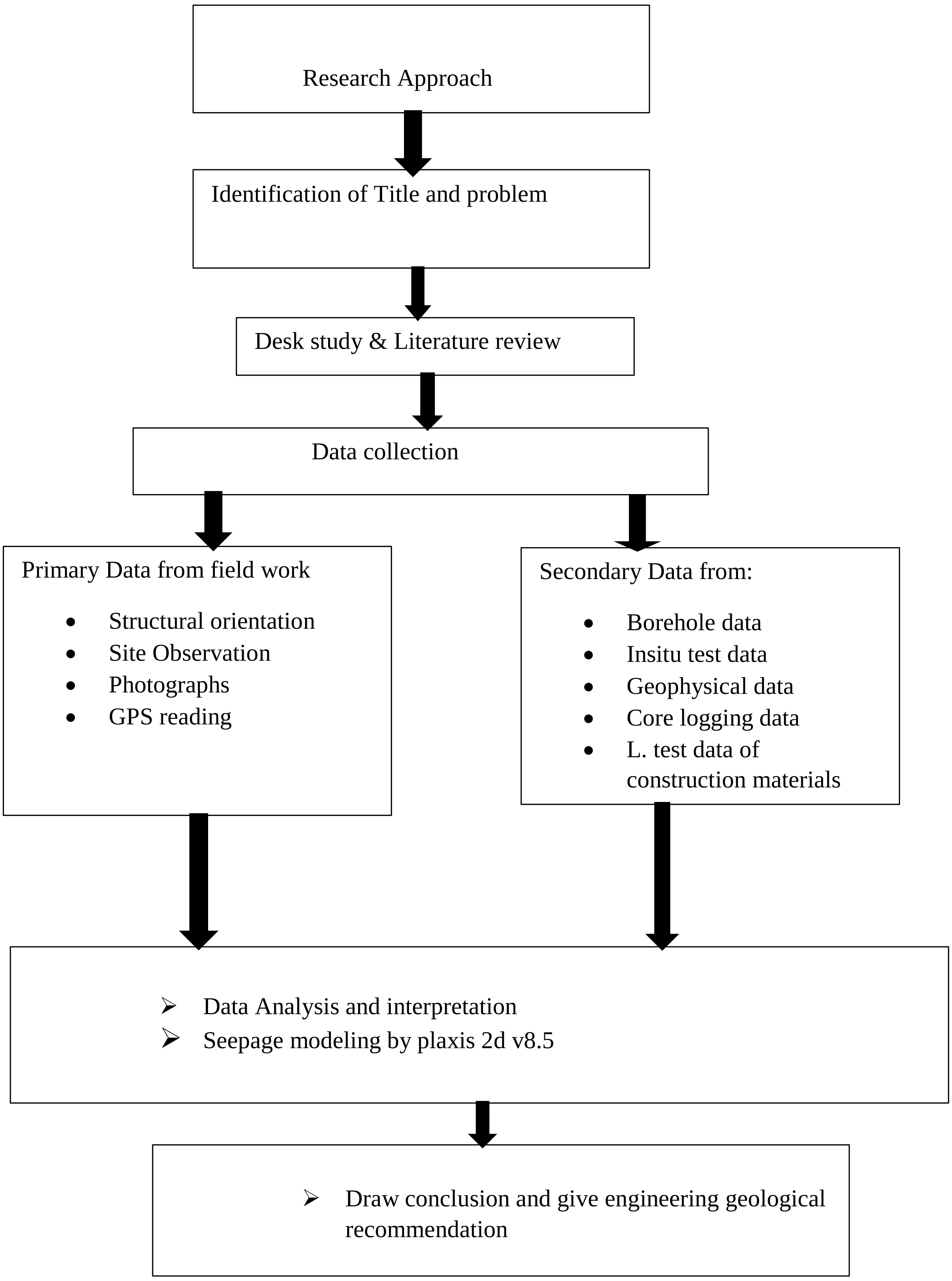

Figure 4.1 conceptual Frame works.

4.5 Materials used during present study

Different materials were used during the present study. Among these materials, a camera, a geological hammer, and a Schmidt hammer were used at field work; however, the availability of exposure at the riverside was low. Only on the top of the ridge of the left abutment were jointed and fractured basaltic rocks available to measure its orientation at that time.

Materials used for result analysis and interpretation (Excel Spreadsheet, Microsoft Word, Arch GIS, and Plaxis 2D software) were utilized in this work.

Plaxis 2D Software

As described in the above 2.9 section, plaxis 2D software is used to model stability, deformation, and flow analysis in geotechnical engineering. So here, plaxis 2D v8.5 was used to analyze the seepage potential of the Fato embankment dam project. The Fato dam site had seepage problems as described by seepage modelling with plaxis software. That mean the quantity of seepage was higher when remedial measures were not used. However, after remedial measures used in modeling the seepage flow and seepage quantity of the foundation and dam body was reduced.

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CHAPTER V ENGINEERING GEOLOGICAL EVALUATION OF FATO DAM FOUNDATION

General

The dam project is an area of work where engineering geology has a high contribution to its success. Accordingly, a geologist or engineering geologist is involved to apply the knowledge of earth science to the dam site. Woodward (2005) summarized the four main tasks of the engineering geologist at the dam site. The first one was the identification of dam foundations, spillways, diversion tunnels, and outlet works with their strength, durability, and water tightness properties. The second was a description of the geology of the reservoir area and its water tightness condition. The third issue was that the engineering geologist decided to solve if the dam site had phase stability problems at slopes and identify whether the reservoir area was susceptible to landslides. The fourth was the suitability and types of construction materials required for the proposed dam site.

In this chapter, the engineering and geological evaluation of the Fato dam foundation will be discussed. "Dam foundation" includes both the valley floor and the abutment upon which the embankment will be built (USBR, 2012). Results, interpretation, and discussion of the present study presented in this chapter regarding foundation-related issues and construction material-related issues are discussed in the next chapter.

5.1 Engineering Properties of Soil foundation

The soil foundation of Fato Dam was mainly formed from underlying basalt rock through the process of weathering and decomposition; however, a few top parts of the riverside were formed from recent alluvial deposits. As indicated from geotechnical core drilling boreholes, the thickness of the soil mass varies from 3m to 8m on the left abutment and from 5m to 30m on the right abutment. Types of laboratory tests analyzed for the foundation of the dam were grain size analysis, SPT data interpretation, Atterberg limits, bulk unit weight, permeability, and free swell. The uppermost portion of the residual soil (0.5–1 m) is rich in organic matter and plant roots, forming a dark brown top layer. The underlying top soil horizon is the residual soil mass, characterized by variegated colors of brownish, red, whitish, and yellowish. The evidence to say residual soil was that the data from core logging and core boxes shows the process of weathering around contact with bedrock. So it was the result of parent weathering that it was overlie. Except for a few parts around the intake area, which were recent alluvial (FDBH4), the other areas were covered by thick residual soils.

Grading of the foundation soil is generally dominated by cohesive soil masses of sand, silty clay, clayey silt, and sandy silt with properties of plasticity ranging from low to high. The inclusion of angular gravel grains increased near the contact with the parent basalt rock.

Figure Soil mass from core log foundation area.

Standard penetration Test (SPT)

Index properties can be determined from a remolded or disturbed sample. In the case of soil mass, the properties were determined from its formation, soil history, and soil structure from the undisturbed samples by in situ tests.

So in-situ strength and permeability tests were done on the Fato dam site. Standard penetration tests (SPT), falling head permeability tests, and packer tests were conducted in the boreholes where the actual condition was found to be in accordance with the particular test type requirements. The standard penetration test results on residual soils of the Fato dam foundation were classified as medium to stiff consistency (N60 ranged from 4 to 15) in its natural state of occurrence at most depths of the drilled borehole, especially the right and left abutments of the dam site.

FDBH1 was drilled at the left abutment of the dam site. In this location, the thickness of the soil was less than 10m. So SPT was done for 8m only in the left abutment. The overly weathered and fractured vesicular basalt rock in the left abutment. At the center (FDBH2), which was the river side, almost the entire rock of the Fato River was covered by less than 2m of soil overlying the bedrock. Therefore, SPT was done for only the soil part. In the case of the right abutment (FDBH3), the thickness of the soils is greater than 30 m, as described below in table (5.1). Because of this, about 14 SPT tests were done in FDBH3 of the right abutment. Finally, the FDBH4 indicated from the logging only a few meter-thick alluvial soil, which may have been removed by excavation. As a result, around the intake area, in-situ SPT was not done there.

Table 5.1 Summary of SPT insitu test result data of Fato dam site.

BH No.	SPT Test No.	Testing Depth (m)	Soil Type Description	Penetration Resistance					Easting	Northing	location
				N_1	N_2	N_3	N	N_{60}			
FD-BH-1	SPT-1	1.8	Silty Clay	3	5	6	11	4			
	SPT-2	3.5	Silty Clay	3	4	5	9	5	360369	979608	Left Abutment
	SPT-3	6	Sandy Silty Clay	6	11	11	22	5			
	SPT-4	8	Gravely Sandy Silt	16	30	38	68	39			
FD-BH-2	SPT-1	1.5	Sandy Silty Clay	3	3	4	7	3	360325	979541	Centre
	SPT-1	1.5	Silty sand with gravel	3	6	7	13	6			
	SPT-2	3.5	Silty Clay	2	3	3	6	3			
	SPT-3	6.6	Sandy Silty Clay	6	8	9	17	10			
	SPT-4	8.05	Clayey sand and silt	6	10	10	20	11			
	SPT-5	11.3	Sandy silt with clay	1	2	4	6	4			
	SPT-6	13.35	Sandy Silty Clay	6	6	5	11	7			
	SPT-7	15	Sandy Silty Clay	5	7	8	15	10			
FD-BH-3	SPT-8	17.1	Silty clayey sand	2	5	5	10	6	360225	979409	Right Abutment
	SPT-9	19	silty clay with sand	3	7	7	14	9			
	SPT-10	20.45	Clayey Silt with Sand	3	5	6	11	7			
	SPT-11	22.4	Clayey sandy Silt	2	3	3	6	4			
	SPT-12	22.55	Clayey Silty Sand	4	9	0	19	12			
	SPT-13	26.5	Clayey Silty and Sand	4	6	7	13	8			
	SPT-14	29	Clayey Silty and Sand	4	11	14	25	16			

Shear Strength of foundation Soil

Shear strength is the maximum resistance of soil before its failure. The shear strength of soil is a main engineering property that controls the stability of soil mass when loads are applied to the soil foundation. The shear strength of the Fato Dam foundation was estimated from the SPT value by using correlation standards. The type of soil in the foundation area of the Fato Dam site is composed of different classes of soil, in which cohesive soil is dominant. The estimated shear strength (Cu) for clay soil minimum was 64 kpa, calculated by using Eqn (2.5) in FDBH3 at a depth of 3.5m with a N60 value of 3. In FDBH1, at a depth of 8 m, its friction angle was estimated at around 38.58 degrees, with a correlation with N60 =39.

The top part of the soil, especially from 0.5 to 1 m, had low strength due to organic materials from plant roots. According to Stroud (1975), Terzaghi, and Peck (1967), the correlation SPT N60 and consistence of the Fato dam foundation area fall between medium to stiff, while strength falls between 50 to 400 Kpa in a range of 4 to 15 dominant in different depths (table 2.3). Similarly, in the left abutment of the dam, from a depth of 1.8–8 m, it is medium stiff and its uniaxial compressive strength is between 50–100 kpa, and at 8 m, it falls into hard with a strength greater than 400 kpa. In the case of the right abutment except SPT2, the others fall in medium to stiff soils with strength 50–200 kpa.

Grain Size Analysis

For the Fato Dam site, both sieve analysis and hydrometer tests were conducted on the soil mass of the site. The grading of foundation soils was dominated by cohesive soil masses (sand, silty clay, clay silty, and sandy silt). The angular gravel was increased near the parent rock contact. A grain size analysis test was done on both sides of the abutments. The test was done on samples taken from different depths of FDBH1 and FDBH3 of the left and right abutments, respectively.

Table 5.2 shows combined tests of sieve and hydrometer test result of FDBH1 different depth

Grain size(mm) Depth(1.80-2.25m)	Percent of finer (%)(1.80-2.25m)	Grain size(mm) depth (3.50-3.95m)	Percent of finer (%)(3.5-3.95m)	Grain size (mm) depth (8.45-8.80m)	Percent of finer (%) depth (8.45-8.8m)	Grain size(mm) depth (6.0-6.45m)	Percent of finer (%)(6.0-6.45m)
2	100	2	100	2	100	2	100
1.18	99.4	1.18	98.9	1.18	99.8	1.18	85.8
0.6	97.6	0.6	95.2	0.6	98.3	0.6	75.9
0.3	95.5	0.3	91.3	0.3	95.9	0.3	68.7
0.15	94.2	0.15	88.7	0.15	94.2	0.15	63.4
0.075	92	0.075	86.5	0.075	92.1	0.075	53.5
0.0311	75.24	0.0302	67.51	0.029	76.16	0.0351	12.86
0.0201	69.14	0.0198	58.71	0.0185	74.21	0.0223	12.06
0.0121	58.97	0.0118	48.92	0.0109	70.3	0.013	9.65
0.0087	52.87	0.0086	41.09	0.008	60.54	0.0092	8.04
0.0063	46.77	0.0061	37.18	0.0057	56.63	0.0065	6.43
0.0032	32.53	0.0031	29.35	0.0029	48.82	0.0032	3.22
0.0014	26.43	0.0013	25.44	0.0012	48.82	0.0014	0

Table 5.3 FDBH3 Soil grain size laboratory test result

Borehole	Depth	Grain size			Location	
		Sand %	Silt%	Clay%		
FDBH3	4.50-5.10m	22.28	42.98	34.74	Right Abutment	E 360225 N 979409
	6.60-7.05m	23.59	34.36	42.05		
	8.05-8.50m	36.61	39.05	24.34		
	10.80-11.30m	26.13	54.82	19.04		
	13.35-13.80m	30	32.93	37.07		
	15.00-15.45m	23.33	33.54	43.12		
	17.10-17.55m	50.33	20.23	29.43		
	19.00-19.45m	19.15	34.46	46.39		
	20.45-20.90m	14.93	50.49	34.58		
	22.40-22.85m	24.14	54.26	21.59		
	24.15-24.35m	17.48	41.73	40.8		
	24.55-25.00m	36.22	33.86	29.91		
	26.50-26.95m	38.17	36.11	25.72		
	29.00-29.45m	45.09	39.26	15.65		
	29.60-29.95	13.25	61.39	25.36		

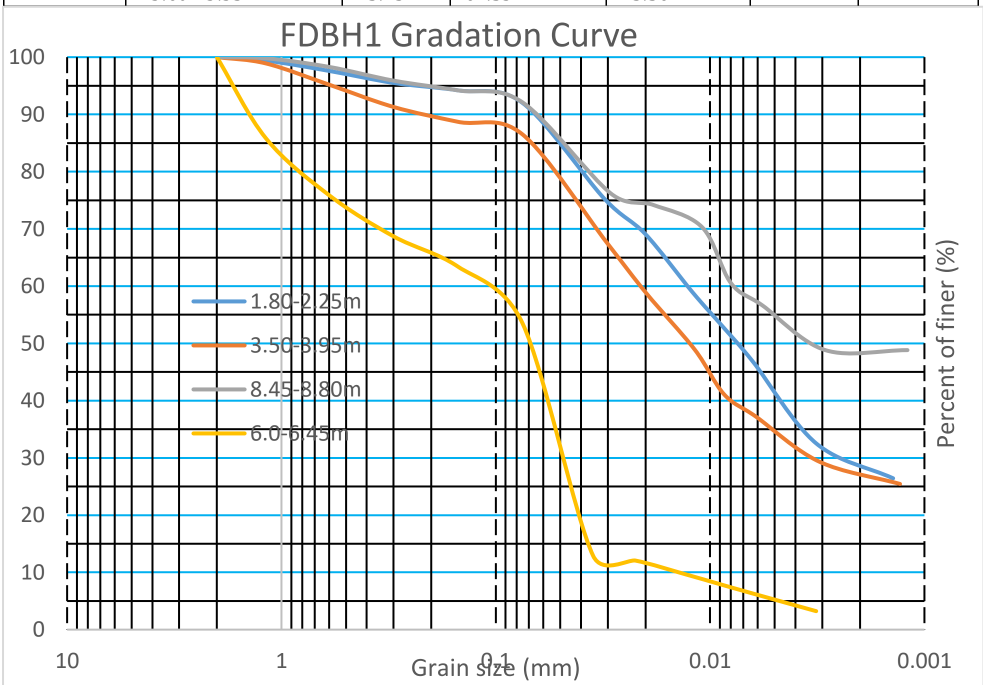

Figure 5.1 Gradation curve of FDBH1

As observed from Figure 5.2, silt-sized soils were dominant in average through the different depths of the right abutment. Sand-sized soils occurred at different depths, from 13.25% to 50.3%, and clay-sized soils were from 15.65% to 46.39%. In the depths of 1.80 to 3.95m and 8.45 to 8.80 m, the percent of fines was greater than 25%. The soil from 6.0 to 6.45m was coarser than the other three test sections. D10, D30, and D60 of the depths of 6.0 to 6.45 m, respectively, are 0.015mm, 0.05 mm, and 0.12mm. Its coefficient of uniformity, Cu =2.4<4

Coefficient of curvature (Cc) = (0.05)2/ (0.015*0.12) =1.4. So it was poorly graded soil.

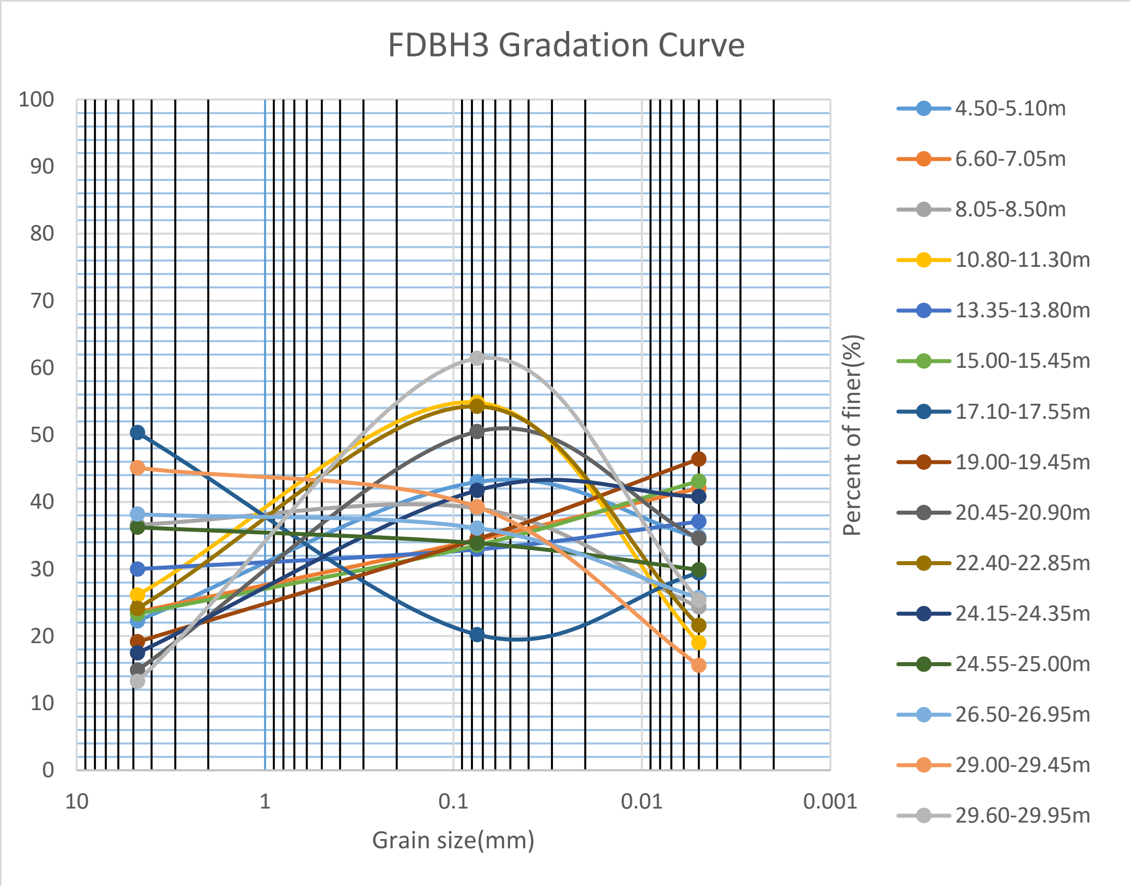

Figure 5.2 Gradation Curve of FDBH3

Atterberg Limits Foundation Soils

From the laboratory test result shown in the table below, the value of the liquid limit of the Fato Dam soil foundation ranged from 39.33% to 61.86%, which falls in between 35–50% and 50–70%. This result is interpreted as having medium to high plasticity. The plastic limit was 24.78% to 49.23%, which falls in the plastic index, which was derived from the two parameters and ranged from 9.17% to 24.21%. The result of the Atterberg limit shows medium to high plasticity interpretation as observed from the plasticity index.

Table 5.4 Atterberg Limit test results of FDBH1 and FDBH3

FDBH1	Depth	Liquid limit %	Plastic limit %	Plastic Index %	Location		Unified soil classification
	1.80-2.25m	59.17	36.64	22.53	Left Abutment	E 360369 N 979608	MH
	3.50- 3.95m	54.50	29.24	25.26			MH
	6.00-6.45m	42.94	33.77	9.17			ML
	8.00-8.45m	53.40	35.04	18.36			MH
FDBH3	4.50-5.10m	39.33	24.78	14.55	Right Abutment	E 360225 N 979409	ML
	6.60-7.05m	53.70	26.42	27.28			CH
	8.05-8.50m	45.60	26.61	18.99			ML
	10.80-11.30m	52.50	38.83	13.67			MH
	13.35-13.80m	53.00	37.34	15.66			CH
	15.00-15.45m	55.30	29.23	26.07			CH
	17.10-17.55m	53.90	38.52	15.38			SC
	19.00-19.45m	57.90	41.04	16.86			CH
	20.45-20.90m	61.86	40.09	21.77			MH
	22.40-22.85m	61.00	49.23	11.77			MH
	24.15-24.35	55.75	32.69	23.06			MH
	24.55-25.00m	41.70	27.70	14.00			ML
	26.50-26.95m	49.10	34.55	14.55			ML
	29.00-29.45m	45.00	31.27	13.73			ML
	29.60-29.85m	53.50	39.47	14.03			MH

Engineering Classification of foundation soils

The engineering properties and behavior of soil vary with different deposits and the nature of its occurrence. According to the Unified Soil Classification System, the Fato Dam foundation soils were analysed based on grain size analysis (tables 5.2 and 5.3) and Atterberg limits (table 5.4). Soils from boreholes FDBH1 and FDBH3 are classified as clayey sands (SC), high plasticity silt (MH), low plasticity silts (ML), high plasticity clay (CH), and low to medium plasticity clayey (CL). The expansiveness of the soils was determined from the free swell laboratory results. Accordingly, Fato Dam foundation soils degrees of expansion ranged from low to medium, as laboratory results for the site ranged from 14.1% to 67.5%, which shows from non-critical to marginal (table 5.5).

Table 5.5 Free swell test results of FDBH1 and FDBH3

	Depth	Free swell %	Location	
FDBH1	1.80-2.25m	62.5	Left Abutment	E 360369 N 979608
	3.50-3.95m	67.5		
	6.00-6.45m	62.5		
FDBH3	1.50-1.95	30	Right Abutment	E 360225 N 979409
	3.50-3.95	40		
	15.00-15.45m	14.01		
	17.10-17.55m	37.5		

The result of free swell in FDBH3 shows less than 50 %(non-critical range), in FDBH1 62.5 to 67.5% (marginal) which mean that the site has no major problem of swelling (Gibbs& Holtz, 1957).

Table 5.6 Bulk Unit weight test result of FDBH1 and FDBH3

Borehole	Depth	Bulk unit weight(KN/m^3)	Location	
FDBH1	1.80-2.25m	22.34	Left Abutment	E 360369 N 979608
	3.50-3.95m	22.44		
	6.00-6.45m	24.21		
FDBH3	1.50-1.95m	14.01	Right Abutment	E 360225 N 979409
	3.5-3.95m	17.25		
	4.50-5.10m	13.72		
	6.60-7.05m	16.46		
	8.05-8.50m	14.5		
	10.80-11.30m	14.9		
	13.35-13.80m	17.35		
	15.00-15.45m	12.25		
	17.10-17.55m	15.19		
	19.00-19.45m	16.76		
	20.45-20.90m	15.48		
	22.40-22.85m	12.94		
	24.15-24.35m	17.05		
	24.55-25.00m	14.41		
	26.50-26.95m	14.99		
	29.00-29.45m	16.27		
	29.60-29.95	15.78		

Permeability of Foundation soils

Permeability is a property of soils that permits liquid flow, and it is a very important engineering property of soils. An in situ permeability test of the soil foundation was conducted through the boreholes, specifically the falling head. A total of 10 tests were done on the Fato dam site. The degree of permeability of the Fato Dam site foundation area was low (0–15 m) in the left abutment. In the case of the right abutment, it was very low (5.2-10.8m) and low (15-34.5 m), according to Bell (2000). However, according to the USBR design standard, it falls within impervious in the depth of 5.2-10.8m in the right abutment, and the other tests performed were semi-pervious.

Table 5.7 Falling permeability test results of FDBH1 and FDBH3

Location		Falling head permeability test result(cm/s)	Depth(m)	Degree Permeability of Left and Right Abutment	
				According to Bell(2007)	As per as USBR
FDBH1(Left Abutment)	E 360369 N 979608	5.5×10^{-6}	0-5	Low	semi pervious
		3.3×10^{-6}	5-10		
		1.4×10^{-6}	10-15		
FDBH3(Right Abutment)	E 360225 N 979409	8.63×10^{-7}	5.2-10.8	Very Low	Impervious
		3.38×10^{-6}	10.8-15	Low	Semi pervious
		3.46×10^{-6}	15-20		
		3.54×10^{-6}	20.42-25		
		1.16×10^{-5}	25-29.45		
		1.23×10^{-5}	29.5-34.55		

5.2 Engineering properties of Rock Foundation

Engineering characterization of rock foundations is used to evaluate the rocks suitability for the dam foundation based on certain engineering properties. Strength, permeability, and deformation characteristics of rock masses are the main engineering properties used to evaluate the rock foundation of dams. For the present study, borehole data and in-situ tests were used to analyze the foundation rock mass.

Table 5.8 Summary of laboratory Results of rock Foundation

Borehole	Depth(m)	Specific Gravity	Bulk Unit weight(KN/m^3)	Water absorption (%)	UCS(Mpa)	Location
FDBH1	14-14.23m	2.97	25.31	6.25	137.32	LA
	28.05-28.23m	2.94	25.31	6	131	
FDBH2	7.55-7.88m	2.46	28.91	3	130	Centre
	14.78-15.05m	2.96	24.7	1	169	
	25.85-26.15m	2.59	27.73	1	217	
	26.15-26.30m	2.72	26.20	1	217	
FDBH3	33.20-33.45m	2.77	22.93	1.6	108	RA
	41.70-42.00	2.94	25.68	2	105	
FDBH4	3.10-3.36	2.78	28.94	1.52	479	Intake

Discontinuities and Strength Condition of Foundation rock

The rock mass at the Fato dam site was slightly weathered and fractured in many parts of the dam site. However, completely weathered and partially decomposed vesicular basalt rocks were also available at the top part of the foundation area (left abutment).

According to Bieniawski's (1989) rock mass classification, the rock mass of the Fato dam site ranged from fair to good quality. That means the rock mass rating (RMR) values ranged from 47 to 79. The result was based on the 6 parameters listed in Chapter 2 (table 2.5) above and those parameters collected from borehole and core logging and also from surface information.

The estimation of deformation modulus was performed by using the empirical relation Fato dam site (Ed), which ranged from 8 to 58 Gpa. The Serafim and Pereira (1983) relation for RMR values 10 to 50 (UCS > 100 MPa) and the Bieniawski (1973) relation for RMR > 55 with UCS > 100 MPa were used in the determination of the deformation modulus of the site.

$E_d = 10^{(RMR-10)/40}$(GPa)…………………………………Serafim & Pereira (1983) -----Eqn (5.1) was used for RMR between 10 to 50 and UCS greater than 100 Mpa.

E_d = (2RMR-100) GPa…………………………. Eqn. (5.2).Bieniawski relation (1978) for RMR greater than 50 while UCS greater than 100 Mpa.

The rock mass of Fato dam site were moderate to strong in its strength. The uniaxial compressive strength (UCS) was ranged from 42Mpa to 478Mpa. According to empirical approach developed by Hoek et al. (1992) for direct determination of shear strength parameters from RMR values. Similarly;

Cohesion'C'= 0.05RMR…………Eqn (5.3)

Angle of internal friction Ø= 0.5RMR+5………. Eqn (5.4)

Estimated cohesion was between 231 to 400kpa and the angle of internal friction was ranged from 28.5 to 43.9 degrees from Bieniawski 1989 RMR Values.

Rock Quality Designation (RQD)

From the borehole data, the Fato Dam foundation showed several ranges of rock quality designations in the area. In borehole FDBH1, which was drilled on the left abutment of the dam, RQD varies from <25 % average to depth (8.6 to 12m). Similarly, in the depths of 12–23.55 m and 23.5–25.55 m, the average value of RQD was 44.6 and 77%, respectively. FDBH2 was drilled at the center of the dam axis, and the average RQD varied from 36–90% in different depths (3.40 to 31.15m). The right abutment borehole (FDBH3) ranged from 30-42m (37.5 to 88.75%), and the intake borehole (FDBH4) ranged from 63 to 88%. Overall, based on average RQD, the boreholes contain dominantly rocks ranging from fair to good in quality.

Figure 5.3 Jointed and weathered basaltic unit at top ridge of left abutment

Table 5.9 RQD variation in geological borehole logging of Fato dam site

Boreholes	Easting	Northing	Elevation(m)	Depth(m)	Average RQD (%)	Location	Rock quality
FDBH-1	0360369	979608	2415	8.6-12	0	Left Abutment	Very poor
				12-23.55	44.6		Poor
				23.55-36.55	77		Good
				36.55-40.2	53.5		Fair
FDBH-2	360325	979541	2401	3.40-7.55	87	Centre of dam axis	Good
				7.55-16.75	52		Fair
				16.75-22.85	36		Poor
				22.85-31.35	90		Good
FDBH-3	360255	979409	2430	30-31.65	88	Right Abutment	Good
				31.65-34.55	37.5		Poor
				34.55-42	88.75		Good
FDBH-4	360265	979585	2399	0.6-5.9	88.45	Intake of foundation	Good
				5.9-11.8	63.38		Fair
				11.8-16.45	84.75		Good
				16.45-20.1	22.1		Very poor
				20.1-22.1	70		Fair

Table 5.10 Rock mass class and Properties Determined by WWDSE (2016)

BH-No.	Depth(M)	Intact Rock strength	Rock mass Class and properties					
FDBH1		UCS(Mpa)/ From point load(Mpa)	RMR	Class	Quality	C(kpa)	Ø(degree)	Em(Gpa)
	8.6-12	42	47	III	Fair	231.6	28.8	8
	12-23.55	138	67	II	Good	336.8	38.7	34
	23.55-36.55		76	II	Good	384.2	43.4	52
	36.55-40.2	131*	60	II	Good	300	35	20
FDBH2	3.4-7.55	130*	77	II	Good	389.5	43.9	54
	7.55-16.75	169*	62	II	Good	310.5	36.1	24
	16.75-22.85	205*	58	III	Fair	289.5	33.9	16
	22.85-31.35	217*	74	II	Good	373.7	42.4	48
FDBH3	30-31.65	108	72	II	Good	363.2	41.3	44
	31.65-34.55		63	II	Good	315.8	36.6	26
	34.55-42	105	72	II	Good	363.2	41.3	44
FDBH4	0.6-5.9	478	79	II	Good	400	45	58
	5.9-11.8	105	62	II	Good	310.5	36.1	24
	11.8-16.45		74	II	Good	373.7	42.4	48
	20.1-22.1		70	II	Good	352.6	40.3	40

‘’*’’means that UCS derived from point load.

Permeability of Foundation Rock

For the Fato dam site, a packer test was conducted in the borehole in the foundation area, including the left abutment, the right abutment, the intake of the foundation, and the center of the dam axis. As described in table (5.10) below, the result of the packer test of the area shows a lugeon value from 1 Lu to greater than 100 Lu. The type of packer test applied to the site was single-method where required. Because in weak rocks, a single packer is more applicable than a double method. The type of machine performing the packer test was pneumatic. Ordering a single-method packer includes:

- ✓ Holes drilled to a particular depth
- ✓ The core barrel was removed, and then
- ✓ The hole was cleaned with water.

The method of testing consisted of five tests or runs, each of ten or five minutes duration, with a particular corresponding pressure magnitude. The testing pressure was applied in the order A-B-C-B-A, with increased and decreased sequences. C = peak testing gauge pressure While A is a low testing pressure, its magnitude is equal to one third of the peak pressure, as shown in the table below. The peak pressure was obtained after deducting the borehole mid-height water column pressure (ground water table) from the total testing pressure (0.1 to 0.2 bar/m), which was measured and computed from the bottom of the test section. The length of the test section was five meters.

The calculated five run values for laminar flow were nearly the same. Accepted permeability was the average of the five run values. The average lugeon values corresponding to the 2nd and 4th testing pressures were for dilation. Similarly, the lugeon value corresponding to peak pressure was for turbulent flow patterns. For washout of joint filling materials, there was a progressive increase in all the Lu without any return to peak, and this was an indication of the permanent washout of material (rock movement caused by testing). The accepted permeability of washout was calculated from the final lower pressure. In the case of void filling, it was calculated from the final lower pressure (which shows a progressive decrease in the five-run Lu).

The corresponding test section permeability (k) values were obtained from the relation Houlsby (1976) that 1 Lu units are approximately equal to 1.85 *10-7 m/sec or 1.85 *10-5cm/sec. Therefore, 14 total packer tests were performed to determine the rock mass permeability at different depths of boreholes. The sequence of pressures applied during the test is described in the following table.

Table shows sequence of Pressure applied in packer test

Test Stage	Description	Pressure Step
1st	Low	0.50*Pmax
2nd	Medium	0.75Pmax
3rd	Peak	Pmax
4th	Medium	0.75Pmax
5th	Low	0.50Pmax

Table 5.11 Packer Permeability test result of rock foundation

Location		Test Type	Test Section (m)	Rock permeability (Lugeon value) LU	Permeability condition
FDBH1(Left Abutment)	E 360369 N 979608	Packer test	16-20.35	1	Impervious
			20.4-25	10	Low
			25-30	<1	Impervious
			30-34.7	23	Medium
FDBH2(Centre of Dam axis)	E 360325 N 979541		5-10	16	Medium
			10-15	11	Medium
			15-20	24	Medium
			19-24.35	>100	Very high
			25.3-30.35	21	Medium
FDBH3(Left Abutment)	E 360255 N 979409		37-42	6	Low
FDBH4(Intake area)	E 360265 N 979585		2-7.4	10	Low
			7.40-11.8	75	Very high
			12-17	12	Medium
			17-22.1	90	Very high

As tabulated in the above table, the result of the lugeon value (Lu) of the dam foundation area was greater than 3 Lu for most of the tests done at different depths. This increasing permeability condition was due to the fractured and weathered nature of the bedrock. From the geophysical investigation and engineering geological cross-section (Figure 5.4), a deep localized fractured formation was analyzed. In addition, the availability of vesicular basalt in the area itself causes increased lugeon value or permeability. Therefore, the degree of permeability should be improved in order to decrease the seepage problem that may happen during and after construction.

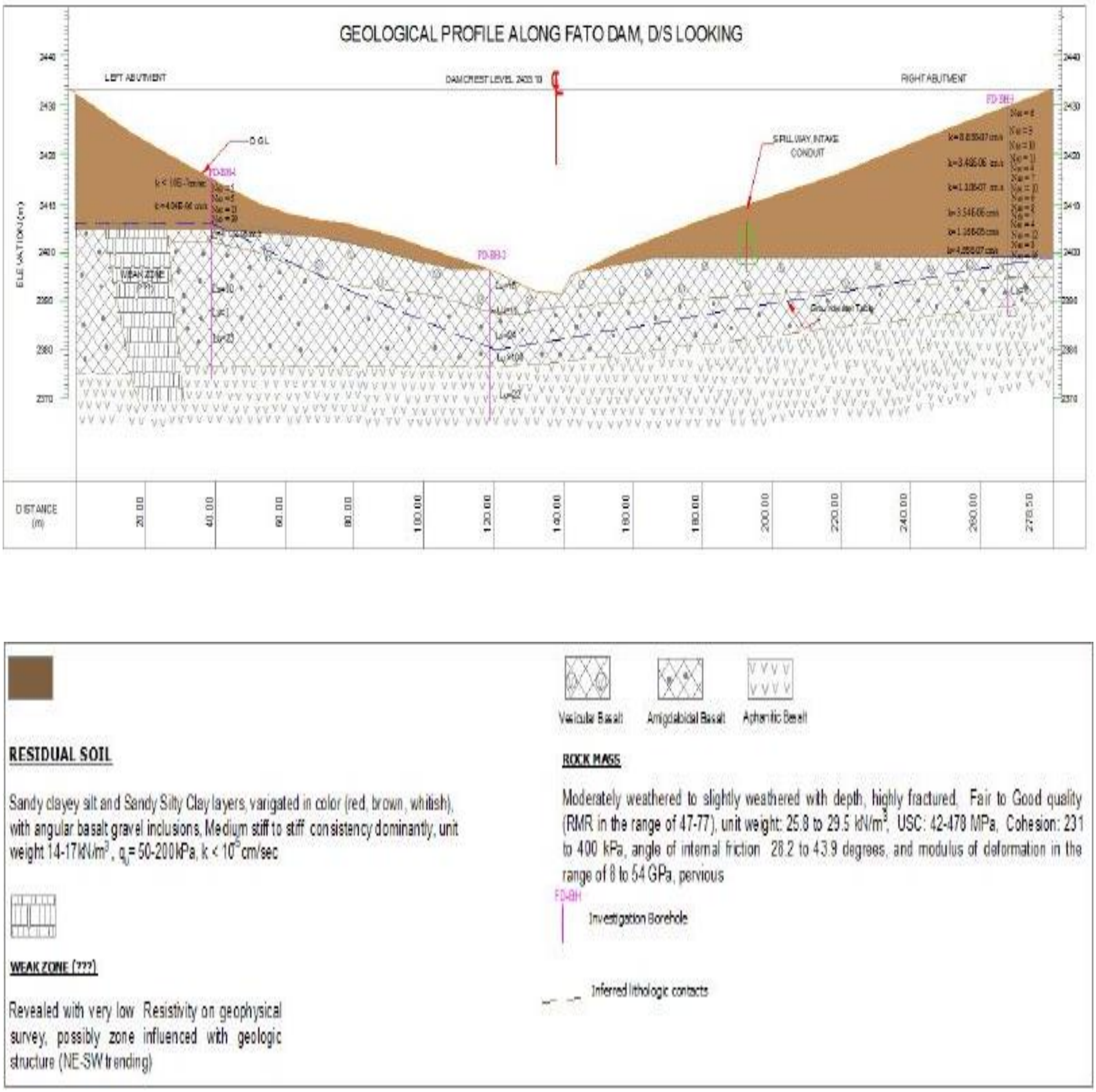

Figure 5.4 Engineering Geological Cross-section Fato dam axis taken from WWDSE, 2016

5.3 Geophysical Survey of Foundation

For the Fato Dam site, the project consultancy conducted resistivity surveying by using vertical electrical sounding and electrical imaging methods. The purpose of the survey was to evaluate the site to meet the objectives of the project. The process was used for mapping the subsurface in addition to borehole information about the site.

The information was about geologic conditions based on the interpretation of resistivity measured to locate the extent and nature of geologic structure or weak zone in the foundation and the location of the ground water table. In addition, the survey was used to interpolate the geomaterials between the geotechnical boreholes.

The electrical resistivity measured from the field work was analyzed and interpreted using appropriate software. The technical standards and calibration to the actual condition of geologic formations results were presented in tabulated summaries in the form of imagine sections and graphs for corresponding imaging lines and VES points tested.

Table 5.12 shows survey lines of electrical imaging.

Survey	Survey Line	Coordination (UTM)				Length of survey line	Depth of Investigation	Remark
		Starting		Ending		(m)	(m)	
		Easting	Northing	Easting	Northing			
Imaging	GPA1	360184	979379	360365	979705	375	55	Upstream of dam
	GPA2	360229	979363	360411	979705	375	55	Axis of Dam
	GPA3	360276	979345	360449	979677	375	55	Downstream of dam
	GPA4	360191	979582	360403	979442	255	55	Right abutment
	GPA5	360251	979669	360450	979517	255	55	Left Abutment
	GPA6	360014	979602	360475	979143	685	55	Spillway
	Total					2320		

Table 5.13 Location of vertical electrical sounding (VES).

No.	VES point	Coordinate		Length of AB/2	Remark
		Easting	Northing		
1	VESA1	360288	979498	150	Right abutment
2	VESA2	360348	979560	100	Left abutment
3	VESA3	360263	979430	100	Right abutment
4	VESA4	360103	979445	100	Spillway channel
5	VESA5	360187	979278	100	Spillway channel

The information concerning soil and rock was collected from resistivity imaging and a vertical electrical sounding survey. That information was analyzed and interpreted by using computer tomography and Win resistance software,

respectively, and corresponding resistivity sections. 2D (for imaging) and 1D (for VES) were produced for each survey line or point.
From the geophysical survey conducted at the Fato dam site, the findings were:

The middle and central portions of the site are comprised of competent bedrock with little or no soil overburden at the surface.
The soil mass overlying the bedrock was significant on abutments, with a thickness estimated at up to 30 meters.
The investigation had interpreted the condition of the bedrock as weathered and fractured.
At the left abutment of the proposed dam site, a relatively weaker and deeper localized zone was interpreted.

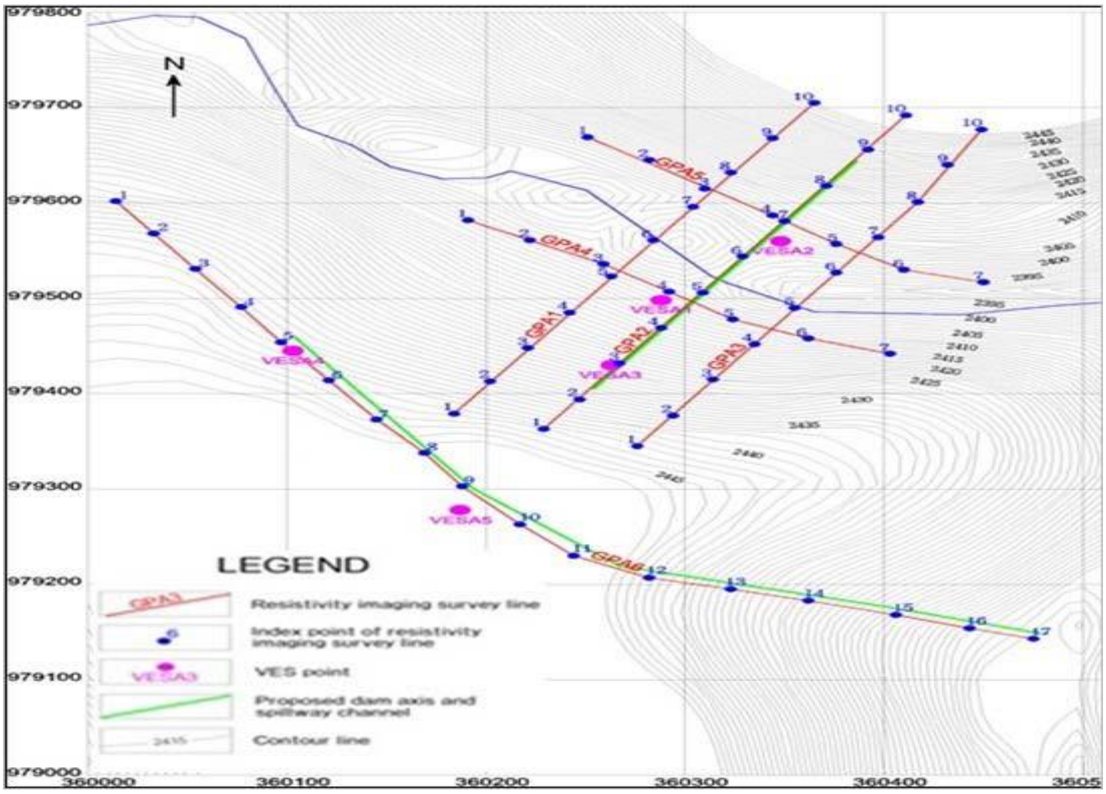

Figure 5.5 Geophysical investigation points of Fato dam site.

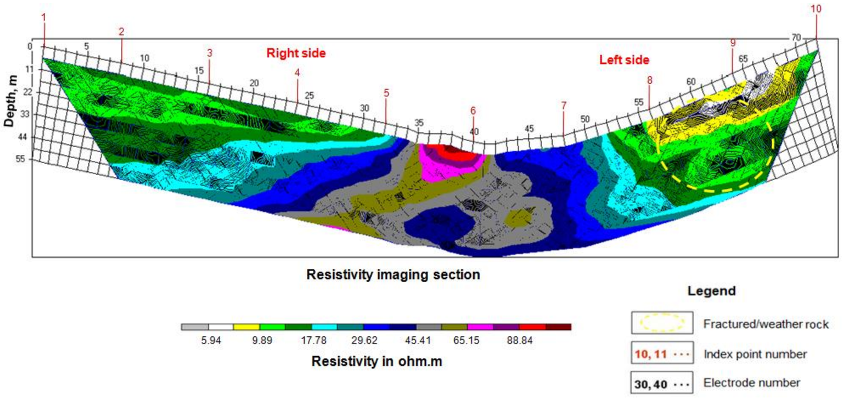

Figure 5.6 Imaging section for GPA-1

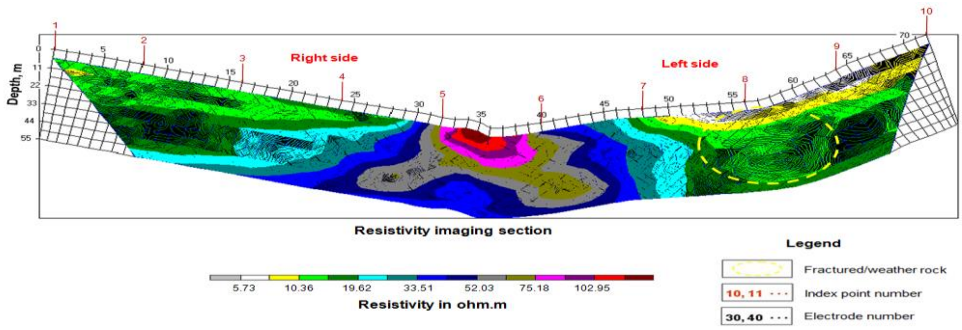

Figure 5.7 Imaging section for GPA-2

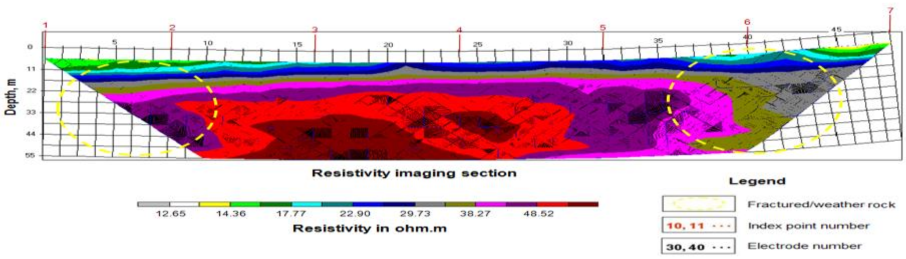

Figure 5.8 Imaging section for GPA

The yellow colour indicated by the image's index shows the area of weak and fractured formation at the site. By correlating the index expressed in yellow colour resistivity imaging with the weak formation boreholes (FDBH1) left abutment, the area was affected by geological structures (discontinuity). The fragment of vesicular from the core box (Figure 5.9) showed that the weathered and fractured rock formation indicated yellow index symbols of geophysical interpretation (Figures 5.6 and 5.7). This made the foundation of the dam problematic if not improved in terms of strength and seepage.

Table 5.14 Summary of VES layers and lithology description

VES	Layers	Resistivity(Ω m)	Thickness (M)	Depth(m)	Expected lithology
VESA-1	1	60	1.4	1.4	Top soil
	2	7.3	3.6	5	Top soil
	3	53	….	…..	Top soil
VESA-2	1	25	1	1	Soils & weathered basalt
	2	21.1	1.1	2.1	Fractured &Weathered basalt
	3	24.1	11.4	13.5	Weak basalt formation
	4	82.9	…..	….	Localized weak basalt formation
VESA-3	1	147	1.4	1.4	Top soils
	2	13.9	2.4	3.8	Top soils
	3	8.8	21.8	25.6	Soil mass
	4	14.8	3	28.6	Soil mass
	5	203.7	…	….	Vesicular Basalt rocks

Figure 5.9 of core box shows that fractured vesicular basalt from left abutment (FDBH1)

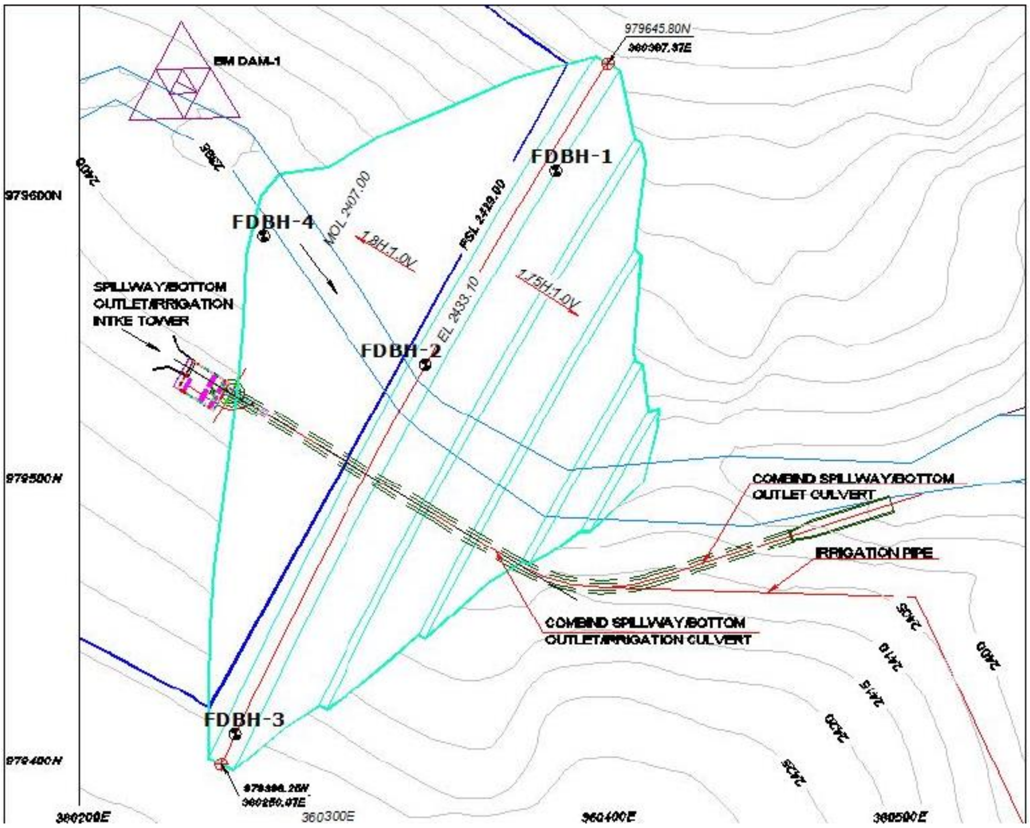

Figure 5.10 shows Location map of Boreholes drilled at dam axis

5.4 Seepage Potential of Fato Dam Foundation

The dam foundation includes both the valley floor and the abutment upon which the embankment will be built. The in-situ permeability test was done to analyze the permeability of the soil mass and rock mass of the foundation. The presence of fractured bedrock along the river channel is anticipated to form potential water-pervious foundation and abutment zones under the dam.

From the summary of the in-situ tests at the Fato dam site through the boreholes, the results generally characterize the residual soil foundation and abutment area as having low to very low permeability, compared to low to very high permeability in the case of basaltic rock masses. From the packer test results in (table 5.11) above, lugeon values greater than 100 Lu in FD-BH2, which was drilled in river bed, and in FD-BH4 of the intake area, lugeon values of 75 Lu (7.40–11.80 m depth) and 90 Lu (17–22.1 m depth) were categorized as having a very high degree of permeability. The information from borehole permeability and strength tests is used to analyze the leakage problems of the foundation and abutment (Kalkani, 1997). In the case of the Fato dam site, the seepage condition of the abutment and foundation area must have been considered during construction and monitoring. Because the data from the geophysical study and bore hole indicate pervious material is available, particularly in the left abutment, localized fractured porphyritic basalt is dominant in the area. The result of seepage analysis by plaxis 2D v8.5 in table(5.16) indicates that there was a seepage problem at the site (table 5.16). Therefore, appropriate measures should be taken to improve the site's condition.

Left Abutment Seepage Condition

The lithology of the left abutment of Fato Dam comprises tertiary basalt rocks such as vesicular basalt, amygdaloidal basalt, and aphanitic basalt. These are associated with fracturing due to tectonic activity and cooling joints. These units extend into both abutments, upstream and downstream of the site. In addition to this, the left abutment residual soils and quaternary sediments are overlain by the basaltic units. Furthermore, a weak zone on the left abutment could form a potential leakage zone for future reservoirs.

According to in-situ packer permeability test results, the rock on the left abutment contains Lugeon values ranging from less than 10 Lu (25–30 m depth) to 23 Lu (30–34.7 m depth) at the minimum and maximum, respectively. So the degree of permeability of the rock mass at the indicated depth was categorized as low to medium permeability according to Lashkaripour and Ghafoori (2002), as shown in (table 2.2). The first packer test value was in the depth of 16 to 20.4 with a lugeon value of 1 Lu, which was categorized as impervious, and the second test value was 10 Lu in the depth of 20.4 to 25 m, where medium permeability degrees of rock mass were identified. The in-situ falling head soil permeability test result of the left abutment was categorized from low to very low permeability according to Bell F.G. (2007) and impervious to semi-impervious according to the USBR rating from table (2.1).

Right Abutment Seepage Condition

The right abutment comprises a thick superficial quaternary deposit that was overlaid on the basaltic unit. Those soils were weathered products of existing parent basaltic rocks, and the maximum thickness of the right abutment reaches 30 m. As indicated in table (5.6), the soil mass in situ falling head permeability result ranged from 8.63*10-7cm/sec to $3.54*10^{-6}$ cm/sec. Therefore, the degree of permeability was categorized from very low to low (Bell, 2007) and impervious to semi-pervious as per the USBR classification table (2.1).

From FD-BH3, the packer test value was lower than the left abutment, which means that the permeability of the rock mass at the right abutment is less pervious in comparison to the left abutment. The packer test performed at a depth of 37–42 m resulted in a lugeon value of 6 Lu; therefore, the rock mass was categorized as having a low degree of permeability.

5.4.1 Recommended Seepage controlling Methods of Foundation

To decide the choice of seepage controlling techniques, in-situ tests, laboratory test results, and information from geophysical surveys were the key parameters kept in consideration. For the present study, soil and rock laboratory or in-situ test results were used for foundations and abutments to analyse the characteristics of the materials in case of seepage. Based on the results of laboratory tests conducted on foundation material, a suitable seepage-control method was selected. The selection was based on the falling head permeability test and packer/lugeon permeability tests performed in boreholes at the site.

Many researchers recommended that two approaches be followed around the world in general in order to control dam foundation and embankment failure due to seepage problems. The first was a preventive approach, while the second was curative. In the preventive approach, the water was kept out as much as possible, and in the curative approach, an appropriate outlet (drainage) of water was provided. In the earth dam, both approaches were used in combination. These approaches were applied through the following activities:

5.4.1.1 Excavation of Unsuitable Materials

The geology of the proposed dam site at the river was composed of basalt units at the centre of the dam axis. The unit is extended into abutments horizontally, and it was overlain by residual soil. Highly weathered materials are removed by excavation. For example, the topmost part of the soil that contains organic materials must be removed. Accordingly, low-strength and compressible materials were removed. Because of those materials, the foundation level of the dam proposed was 0.5 to 12.8m below ground level. So materials that were highly susceptible to erosion were removed.

5.4.1.2 Consolidation (blanket) Grouting

Consolidation grouting was done in areas influenced by low shear strength and high deformation characteristics of the rock mass. It was used to close all the openings of fissures, joints, and cracks found at the foundation of the dam. Consolidation grouting was required to improve the strength of the foundation by making it strong enough and also improving the seepage problems. It required low pressures because it was done near the surface, below the impervious core of the embankment.

Fato Dam foundation consolidation grouting was selected as one possible alternative for improving the foundation. Therefore, the consolidation (blanket) grouting should be provided up to a depth of 10m beneath the impervious core of the embankment to strengthen the foundation. The depth of grouting was based on the standard (USBR, 2014) recommendation for embankment dam foundation grouting. According to the standards, for embankments 30m and above, 5 to 10 m of blanket grouting of the foundation beneath the core of the embankment dam should be provided based on site-specific geological conditions. Therefore, for the present study, a blanket grouting depth of 10m is recommended because the geology of the left abutment was highly fractured and weathered rock formations existed. The spacing of the blanket grouting primary was 40ft (12.192m) based on the geology of the site (USBR, 2014). For the present study, a primary spacing of 12 m is recommended. Secondary and tertiary may not be mandatory from an economic point of view because curtain grouting in three rows in combination with 12m spacing of blanket grouting is applied for the purpose of permeability and strength improvement.

5.4.1.3 Curtain Grouting

From the geophysical and borehole data of the site, the bedrock in the area was predominantly volcanic, with weathered and fractured rock. Consequently, very high permeability is expected in the left abutment. Curtain grouting is required to provide a screen to prevent seepage and leakages at dam foundations and abutments. Therefore, the depth of the grout curtain was proposed to satisfy primarily the hydraulic criteria (the depth required to increase the seepage line below the dam foundation so that seepage would not be excessive). The depth of curtain grouting was taken from (ICOLD, 2005 Bullet 129). The bullet states that the acceptable depth of curtain grouting is from 0.35 H to 0.75 H. The attempt was based on Houlsby's (1990) suggestion that when the Lugeon values are below 3, no grouting is required; when between 3 and 10, a single row of grouting is required; and when the values are above 10, a grout curtain should include three rows of grouting holes. Accordingly, three rows of curtain grouting holes were recommended as the foundation Lugeon value was dominantly above 10 Lu.

The principles of closure were adopted to design the grouting pattern and spacing based on the site condition. For weak formation of Fato dam, the spacing of the primary curtain is recommended at 12m (Houlsby, 1990). Likewise, primary (P), secondary (S), and tertiary (T) rows are recommended to reach the requirements. Their hole spacing was 12m, 6 m, and 3 m, respectively. It states that the grouting should result in a reduction in lugeon value to 3 Lu. Therefore, Fato Dam foundation grouting in that value range was considered standard for closure and refusal to grout. Grouting should be carried out sequentially to achieve this predetermined standard of water tightness.

5.4.1.4 Dental Treatment

The foundation contains weak fractured rocks in the left abutment, which are mostly sheared and partly crushed zones. Those zones were altered or weathered rock, as described from geophysics and borehole data at the site. The weak formation that extends from 12 to 31 m on the left abutment of Fato Dam requires this type of treatment. Therefore, a dental treatment was required for localized weak or compressible features extending into an acceptable depth of foundation rock. So dental treatment was another possible alternative required to improve low shear strength areas of the foundation, especially in the left abutment. The treatment was done according to Shasta's formula for dental treatment in order to backfill with lean concrete. This treatment was recommended to decrease the seepage challenges that may occur at the site due to localized fractured zones.

The Shasta's formula of dental treatment is given by:

$D = 0.00656H*b + 1.526$

Where, D is depth of excavation of weak zone below the surface adjoining sound rock in meters ,H is the height of dam above the general foundation level in meters and b is the width of the weak zones in meters.

Figure 5.11 Grouting test in Weak formation at left abutment

Therefore, this treatment is good for 4.7 to 9.7m depth for weak zones extended from 12 to 32m in the left abutment.

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CHAPTER VI CHARACTERIZATIONS OF CONSTRUCTION MATERIALS FOR FATO DAM SITE

6.1 General

For any dam project, the selection of appropriate construction materials is very important to characterize its quality as well as its quantity. Especially the characterization of materials for various zones must be made in detail and with care in order to construct a safe and durable dam project. Thus, the occurrence of pervious and impervious materials in sufficient quantities with standard quality and an economic distance from the dam site are the main factors that are taken into consideration when selecting a dam type. Especially for embankment dams, the quality and quantity of construction materials are the main factors that influence the function of a dam.

6.2 Selection of the potential construction Materials

The selection of construction materials may be different for different projects based on its purpose and the standards required for each unique engineering work. Therefore, the characterization of construction materials is the most primary phenomenon considered during site selection for any construction project. The quality and quantity of construction materials must be analysed in addition to its distance from the project. Construction materials should be as close to the project as possible. This needs to decrease the cost of transportation of the project.

Fato Dam site was selected to build the embankment dam with zoned rocks and soils alternatively, as per the project requirement. The selected zones were based on topography, foundation condition, and the availability of construction materials. After site investigation, they approved the zoned dam with central impervious clay cores (zone 1), followed by filters F1, F2, and transition (zones 2A, 2B, and 2C), rock fill (zone 3), riprap (zone 4), and rock toe (zone 5). The selected zone was mainly based on the availability of construction materials. Therefore, to meet the requirements of each zone, materials were proposed for further analysis based on standards. Each material has its own standards of acceptance for both soil and rock fill materials.

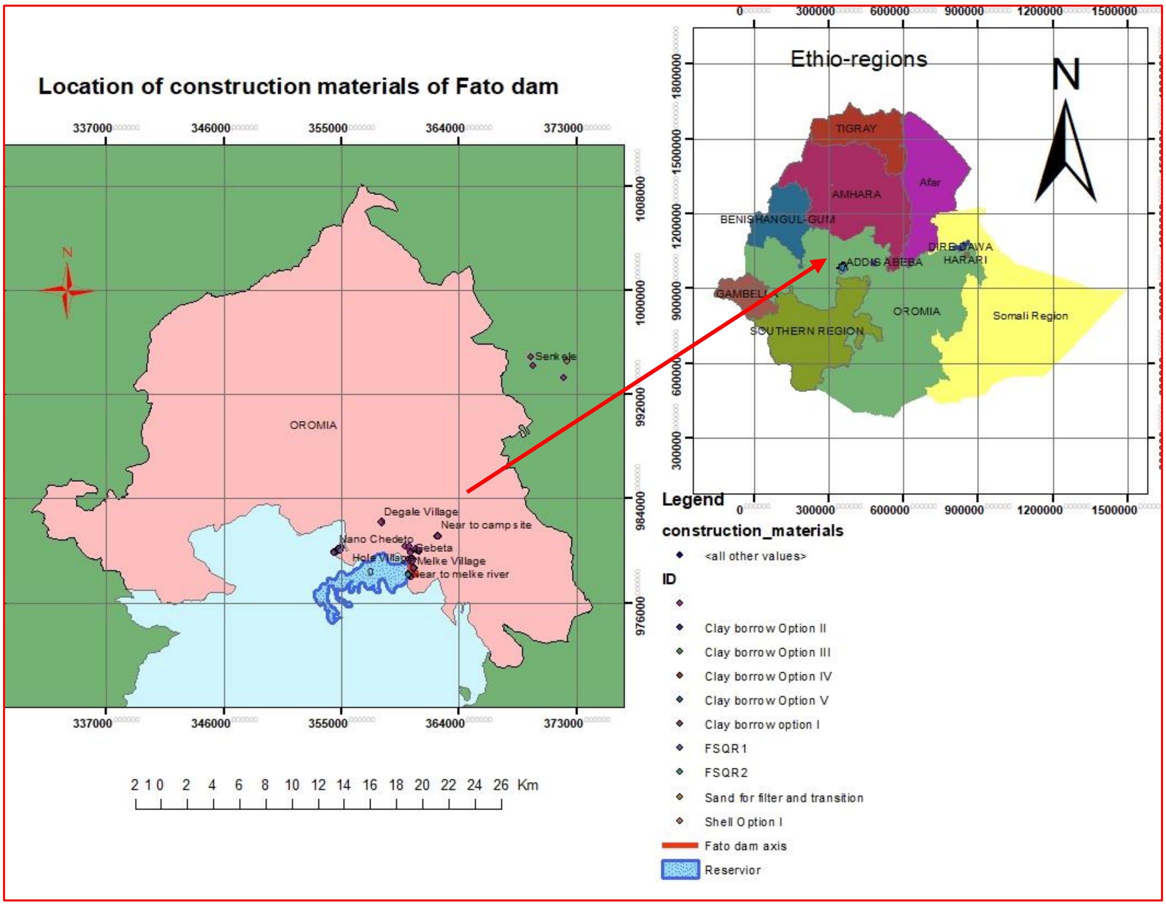


Figure 6.1 Map of construction material proposed for Fato Dam project

6.2.1 Construction materials for Clay core of the dam

Six sites are identified as potential clay borrow areas to be used in core materials (table). To characterize the physical and engineering properties of the residual soil deposits selected for core materials, seven samples were collected and analyzed in a laboratory. The summary of the results of the laboratory tests that are useful in engineering design is presented in Table 6.1.

A compaction test done on the sample of clay borrow shows that the maximum dry density ranges from 1.292g/cm^3 to 1.5 g/cm^3 . The lower value of maximum dry density was due to the fine-grained nature of the clay soils. Optimum moisture content ranges from 24.5 to 37.5%, and natural moisture content ranges from 19 to 22%. The degree of permeability of clay materials was very low (1.07x10^{-7} cm/s to 5.49 x 10^{-8} cm/s).The Atterberg result showed that the soil had medium compressibility and also medium to high plasticity properties. The shear strength parameters of the clay materials were 42 to 60 Kpa cohesion and 11.31^o to 23.3^o friction angle, while the unconfined compressive strength (UCS) ranged from 233 to 360 Kpa. The grain size analysis test result of this clay material fulfilled the criteria for core materials. Percent of soil finer than grain size 0.005mm showed that value of 31 to 41% (Figure 6.2). Therefore, the proposed clay borrow was suitable for core because its gradation meets the standard criteria for core materials.

Table 6.1 Laboratory test result of Clay materials

Location	Depth(m)	Sample ID	Compaction(Standard Proctor) Maximum dry density MDD(gm/cc	Compaction(Standard Proctor) Optimum Moisture Content OMC (%)	Direct shear test – Shear strength parameters Cohesion (Kpa)	Direct shear test – Shear strength parameters Friction angle(°)	Unit weight(gm/cc)	Maximum Unconfined Compression Strength Test UCS(Kpa)	Organic content (%)	Permeability(cm/sec)
E 360857 N 980031	0.90-2.40	CBTP-1BDS1	1.292	37.50	58	23.30°	2.54	271.23	0.55	5.02×10^{-8}
E 360994 N 980040	0.0-3.0	CTP-2BDS1	1.378	24.50	43.50	17.48°	2.35	359.24	0.78	5.49×10^{-8}
E 360651 N 980145	0.40-2.0	CBTP-3BDS1	1.300	35.40	59.33	19.29°	2.37	274.86	3.88	3.19×10^{-9}
E 360249 N 980382	0.40-3.0	CBTP-5BDS1	1.319	37.30	60.33	18.77°	2.49	218.33	0.84	1.07×10^{-7}
E 360537 N 978765	0.0-3.0	FCB-5-TP-1BDS1	1.431	34.00	42.00	11.31°	2.41	233.87	0.76	3.09×10^{-7}

Table 6.2 Grain size analyses, natural moisture content, Atterberg L.

Location	Depth	Sample ID	Grain size analysis Clay (%)	Grain size analysis Silt (%)	Grain size analysis Sand (%)	Grain size analysis Gravel size (%)	NMC (%)	Atterberg Limits LL (%)	Atterberg Limits PL (%)	Atterberg Limits PI (%)	Free swell (%)
E 360994 N 980040	0.50-1.40	CBTP-2SDS1	42.2	39.6	16.2	0	19.53	56.35	27.8	28.55	52.5
E 360651 N 980145	0.40-2.0	CBTP-3SDS1	19.09	70.51	10.4	0	21.17	53.44	33.16	20.28	62.5
E 360399 N 979376	0.0-3.0	CBTP-4SDS1	34.4	43.88	21.72	0		41.7	21.21	20.49	50

Table 6.3 Summary of Combined Sieve and hydrometer laboratory test result

CBTP-4SDS1 Grain size (mm)	Percent of finer (%)	CBTP-3SDS1 Grain size(mm)	Percent of finer (%)	CBTP-2SDS1 Grain size(mm)	Percent of finer (%)
2	100	2	100	2	100
1.18	99.3	1.18	99.7	1.18	99.7
0.6	94.9	0.6	97.9	0.6	96.8
0.3	87.8	0.3	95.2	0.3	92.1
0.15	82.8	0.15	92.4	0.15	89.4
0.075	78.3	0.075	89.6	0.075	83.8
0.0322	64.75	0.0332	52.24	0.0311	71.33
0.0206	61.72	0.0213	49.23	0.0199	67.31
0.0121	55.65	0.0125	44.2	0.0118	61.28
0.0088	49.58	0.009	39.18	0.00084	57.26
0.0062	47.55	0.0064	35.16	0.0061	53.25
0.0032	37.44	0.0033	21.1	0.003	47.22
0.0013	31.37	0.0014	17.08	0.0013	41.19

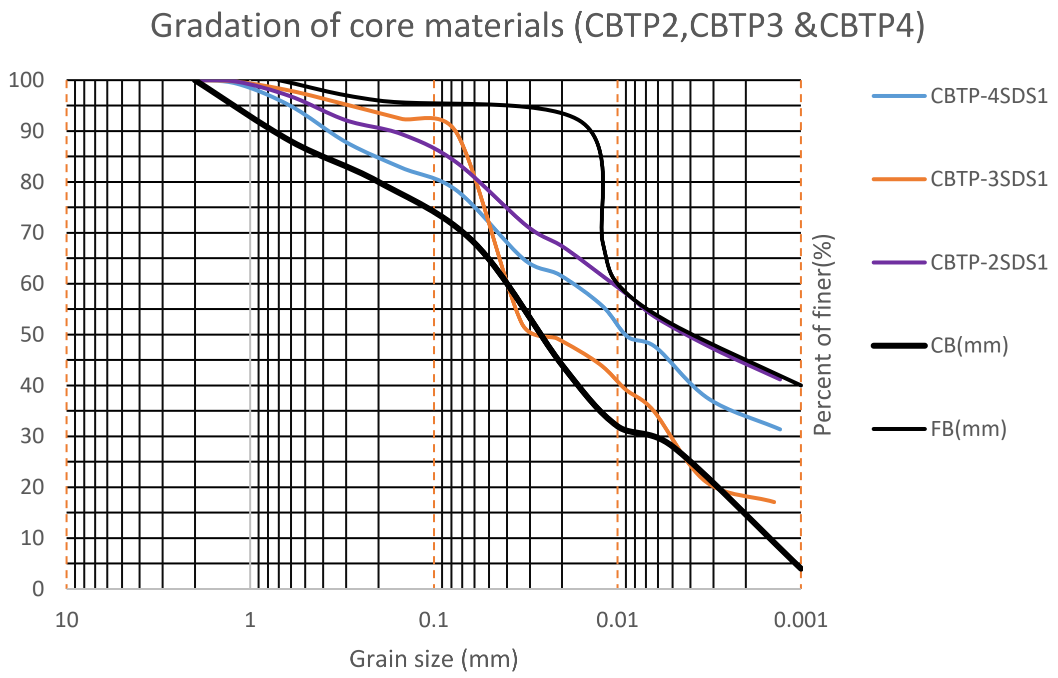


Figure 6.2 Gradation Curve of Coarse boundary (CB) and Fine boundary (FB)

6.2.2 Selection of Materials for Filter zone

The sand materials assessed in the river bed of the project site were not sufficient for the objective of the project. Hence, the only identified sand source for filter and transition zones was located around the Ambo area Senkelle sandstone. So a total of six sand samples were collected from different borrow pits for laboratory testing and analysis.

UGSS-04 was a blend of UGSS-01, UGSS-02, and UGSS-03. Summaries of test results are shown in the table (6.4) below. The samples show different proportions of fines and gravel content. The fine content varies from 10% to 15% on average. However, the quality of sand varies from place to place within the Senkelle locality. Therefore, consideration should be given accordingly. The blended sand sample (UGSS-04) shows about 13% fine content, which can be taken as the average fine content. So it was very high when compared to the filter criteria (it should be <5%). Because of this, the filter and transition materials were produced by grinding basalt at the dam site, as there was an abundant fresh basalt source near the project.

Table 6.4 summary of laboratory test results of sand materials.

Parameter		UGS-01-R1	UGS-01-R2	UGS-01-R3	UGSS-01	UGSS-02	UGSS-03	UGSS-04
Grain Size Analysis	Silt and clay content (%)	9.84	10.51	15	10.82	11.65	15.96	13.01
	Sand %	77.84	60.39	57.97	79.99	83.76	81.14	81.52
	Gravel %	12.32	29.10	26.36	9.19	4.59	2.90	5.46
Specific Gravity		2.43	2.57	2.65	2.80	2.59	2,60	2.68
Bulk unit weight gm/cc		ND	ND	ND	39.08	38.38	32.70	35.88

In the case of concrete works, the project had two alternative sources of sand identified, i.e., the first option was by processing sand material (transporting and washing) from a borrow site located around Senkelle, 30 km from the dam site. The second option was to grind basalt at the dam site, as there was an abundant basalt source near the project site. The alternative was based on a cost-benefit analysis between the two sources. The geotechnical cost analysis proved that the cost of transportation from Senkelle was calculated to be 500.00 birr/m^3 while the cost of grinding the fresh basalt at the dam site was calculated to be around 718.85 birr/m3. Therefore, the cost of transportation is decreased by 218.85 birr/m^3 compared to grinding the basalt found near the site. Therefore, sand materials from Senkelle are suitable for concrete because it require a lower cost than processing basalt at the site. The sand materials initially proposed were used as concrete preparation because its transportation cost was lower than the cost of basalt preparation at the site by grinding.

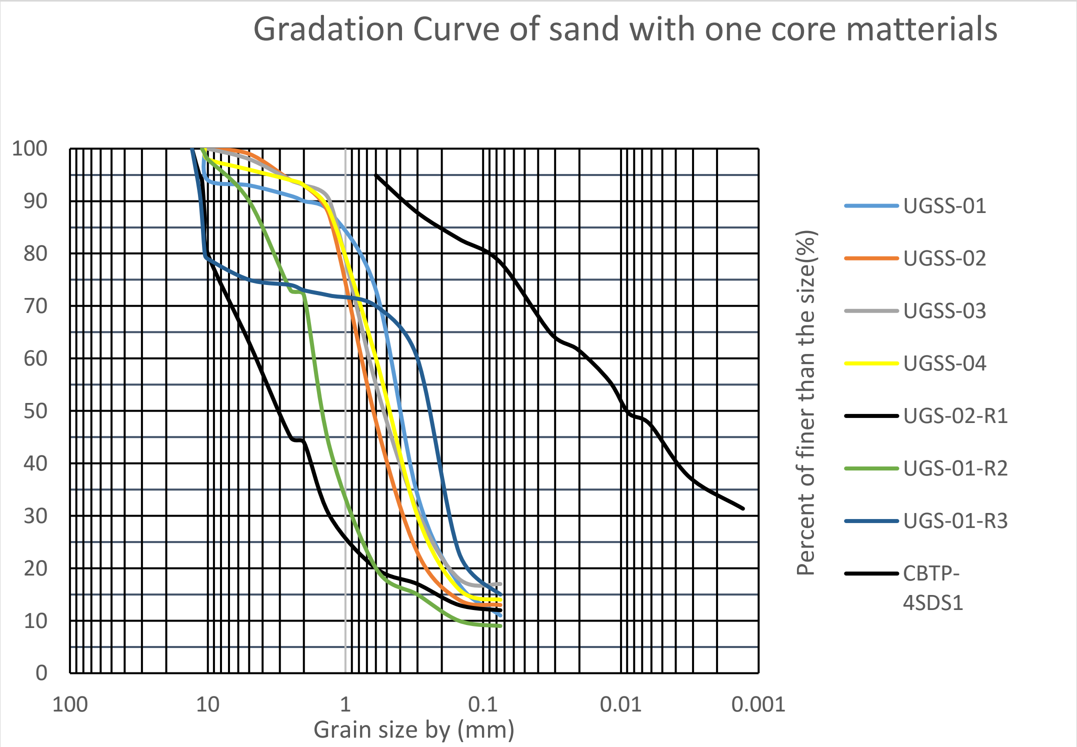


Figure 6.3 Gradation Sand proposed for filter and transition with core materials.

The filter materials of the present project include: fine filters (F1), coarse filters (F2), and transition materials (2C). From the above gradation curve and tables (6.5 and 6.6), filter criteria are interpreted as follows: The analyzed sand materials with core materials here did not satisfy even one of the three fully. Therefore, the sand material proposed from the Senkelle area was not suitable for filter and transition materials.

Table 6.5 particle sizes of sand proposed for filter and sample core materials

Particle size (mm)	UGSS-01	UGSS-02	UGSS-03	UGSS-04	UGS-02-R1	UGS-01-R2	UGS-01-R3	Core materials(CBTP-4SDS1
D_{10}	>10% fines	>10 fines	>10 fines	>10 % fine	>10 %fines	0.15	>10 % fines	>10% fines
D_{15}	0.15	0.18	>15 % fines	0.14	0.22	0.3	0.075	>15% finer
D_{50}	0.4	0.63	0.53	0.48	3.1	1.3	0.25	0.009
D_{60}	0.48	0.88	0.7	0.6	4.5	1.8	0.3	0.018
D_{85}	1.2	1.3	0.9	1.3	8.5	4	1.3	0.2

$C_U = \frac{D6O}{D10}$determined for UGS-01-R2 but not satisfy USBR criteria because 12> (3-4).

The others contained fines greater than 10%. Therefore, we could not determine the coefficient of uniformity. Generally, sand selected from the Senkelle area is not effective because it doesn’t satisfy the standards proposed by different researchers. Due to this, filter material should be processed from locally available basalt material within the standards of filter criteria.

Table 6.6 Terzaghi, Indian standard and USBR Filter criteria analysis

Criteria	Characteristic		Filter materials							
			UGSS-01		UGSS-02		UGSS-03		UGSS-04	
			R	C	R	C	R	C	R	C
TERZAGHI	$\frac{D15\ of filter}{D85\ 0f\ protected\ layer} < 4 to 5$		0.75	S	0.9	S	NA	NS	0.7	S
	$\frac{D15\ of filter}{D15\ of protected layer} > 4 to 5$		NA	NS	NA	NS	NA	NS	NA	NS
INDIAN standard	$\frac{D15\ of filter}{D85\ of base} < 5$		0.75	S	0.9	S	NA	NS	0.7	S
	$\frac{D15\ of filter}{D15\ of base} > 4 \& < 20$		NA	NS	NA	NS	NA	NS	NA	NS
	$\frac{D50\ of filter}{D50\ of base} < 25$		44.4	NS	70	NS	58.8	NS	53.3	NS
USBR	$R50 = \frac{D50\ of fiter}{D50\ of\ base}$	1. R_{50} CU(5 to 10)	44.4	NS	70	NS	58.8	NS	53.3	NS
		2. WG-PG sub-rounded R_{50} (12 to 58)	44.4	S		NS		S		S
		3. WG-PG angular particles R_{50}(4 to 30)	44.4	NS	70	NS	58.8	NS	53.3	NS
	Ratio $R_{15} = \frac{D15\ of\ filter}{D15\ 0f\ base}$	1. WG-PG (Sub-rounded) R15(12-40)	NA	NS	NA	NS	NA	NS	NA	NS
		2. WG-PG(angular particles) R_{15}(6-18)		NS		NS		NS	NA	NS
	$CU = \frac{D60}{D10} = 3 - 4$		NA	NS	NA	NS	NA	NS	NA	NS

Key:

C= Criteria, R= Result, NA= No answer, NS = Non Satisfied, S= Satisfied

The option of filter and transition materials was prepared from basalt rocks dominantly available near the dam site within standard gradation. Gradation of filter materials was achieved through procedures that balanced the permeability between the finer side and the coarser-grained particle distributions. So based on standards, the preparation limits were set in consideration of being fine enough to be filtered properly and coarse enough to be effective in permeability cases. This was done based on determining D15F .Maximum

D15F to prevent particle movement, while the minimum D15F is selected to provide the permeability required.

General order followed to analysis Fine filter (F1) and Coarse filters (F2) using USBR, 2011.

Fine filter (f1)

- ✓ D15 B (average) = 0.0022mm from the gradation curve of base soil plotted (Figure 6.2).
- ✓ Size adjustment was not required because no size was greater than (4.75mm).
- ✓ Determination of the soil category of base soil was category (I), table 2.7, and figure 6.2.

  Average passing sieve No. 200 was greater than 85% (Figure 6.2).
- ✓ Determination of Maximum $D_{15}F$ =cannot less than 0.1mm according to standard (Table 2.7) $D_{15}F \leq 9xD_{85}B$; $D_{15}F \leq 9X0.123=1.107$

  For maximum F1 take $D_{15}F$ upper limit =0.5mm
- ✓ Minimum $D_{15}F$ designed for providing sufficient permeability. $D_{15}F \geq 5*D_{15}B$ but not less than 0.1mm. 5*0.001=0.005 which was <0.1mm. Therefore take $D_{15}F$ lower limit =0.15mm
- ✓ Determine $D_{60}F$ and $D_{10}F$ for upper and lower limit of f1

  By finding coefficient of uniformity $D_{60}F/D_{10}F \leq 6$

  $D_{10}F1$ upper limit$=\frac{\text{D15F upper limit}}{D100B\ Coarser}$

  =0.5/1.2= 0.42mm.

  $D_{10}F1$ lower limit$=\frac{\text{D15F lower limit}}{1.2}$ =0.15/1.2= 0.13mm.

  $Cu=\frac{D60\ F1\ of\ upper\ limit}{D10\ F1\ of\ upper\ limit}=6$

  $D_{60}F$ of upper limit = $6*D_{10}F$ of upper limit

  = 6*0.42mm

  = 2.52mm

  $Cu=\frac{D60\ F1\ of\ lower\ limit}{D10\ F1\ of\ lower\ limit} = 6$

  $D_{60}F1$ of lower limit = $6*D_{10}$ F1 of lower limit

  = 6*0.13=0.78mm.
- ✓ Determining of D_{90} by using (table 2.9)

  For D_{10} F1 lower=0.13mm < 0.50mm---------------take $D_{90}F$ of lower = 5mm

  For D_{10} F1 upper = 0.42mm <0.50mm--------------take $D_{90}F$ of upper = 10 mm.

Coarse Filter (F2)

Determine of $D_{15}F2$ soil base within the same in category I similarly with fine filter (F1). The D_{15} F1 maximum of F1 was became $D_{15}B$ for the lower F2.

Therefore $D_{15}F$ lower $=5*D_{15}B$ =5*0.5mm

= 2.5mm

In order to prevent the possible gap graded the ratio of $D_{15}F2$ upper to $D_{15}F2$ lower should be less than or equal to 5.

Cu = $\frac{D15F2\ upper\ limit}{D15\ F2\ lower\ limit} \leq$ 4 to 5------------take a ratio of 4.

$D_{15}F2$ upper limit = $D_{15}F2$ lower limit * Cu=2.5mm*4= 10.0mm.

Determine $D_{60}F2$ and $D_{10}F2$ from the Cu= $D_{60}F2/D_{10}F2 \leq 6$

$D_{10}F2$ upper limit =$\frac{\text{D15F2 upper limit}}{1.2}$ = 10/1.2=8.33mm..

$D_{10}F2$ lower= $\frac{\text{D15F2 lower}}{1.2}$ = 2.5/1.2=2.08

Then take Cu=4

$D_{60}F2$ upper limit=4*$D_{10}F2$ upper=4*8.33=33.32mm

$D_{60}F2$ lower limit= 4*$D_{10}F2$ lower=4*2.08= 8.32mm

Determining the limit of $D_{10}F2$ and $D_{90}F2$

For $D_{10}F2$ lower= 2.08mm--------take $D_{90}F2$ lower =20mm

For $D_{10}F2$ upper =8.3mm----------take $D_{90}F2$ upper= 50mm

Table 6.7 Summary of Gradation of fine filter (F1)

D_{f1}	Lower boundary of fine filter		Upper boundary of Fine filter	
	size(mm)	Percentage of pass (%)	size(mm)	Percentage of pass (%)
D_0	0.08	0	0.3	0
D_{10}	0.13	10	0.42	10
D_{15}	0.15	15	0.5	15
D_{60}	0.78	60	2.52	60
D_{90}	5	90	10	90
D_{100}	10	100	15	100

Table 6.8 Summary of gradation coarser filter (F2)

Gradation of Coarse Filter(F2)				
D_{f2}	For D_f lower limit		For D_f upper limit	
	size(mm)	Percentage of pass (%)	size(mm)	Percentage of pass (%)
D_0	1.5	0	4.75	0
D_{10}	2.08	10	8.3	10
D_{15}	2.5	15	10	15
D_{60}	8.3	60	33.2	60
D_{90}	20	90	50	90
D_{100}	25	100	70	100

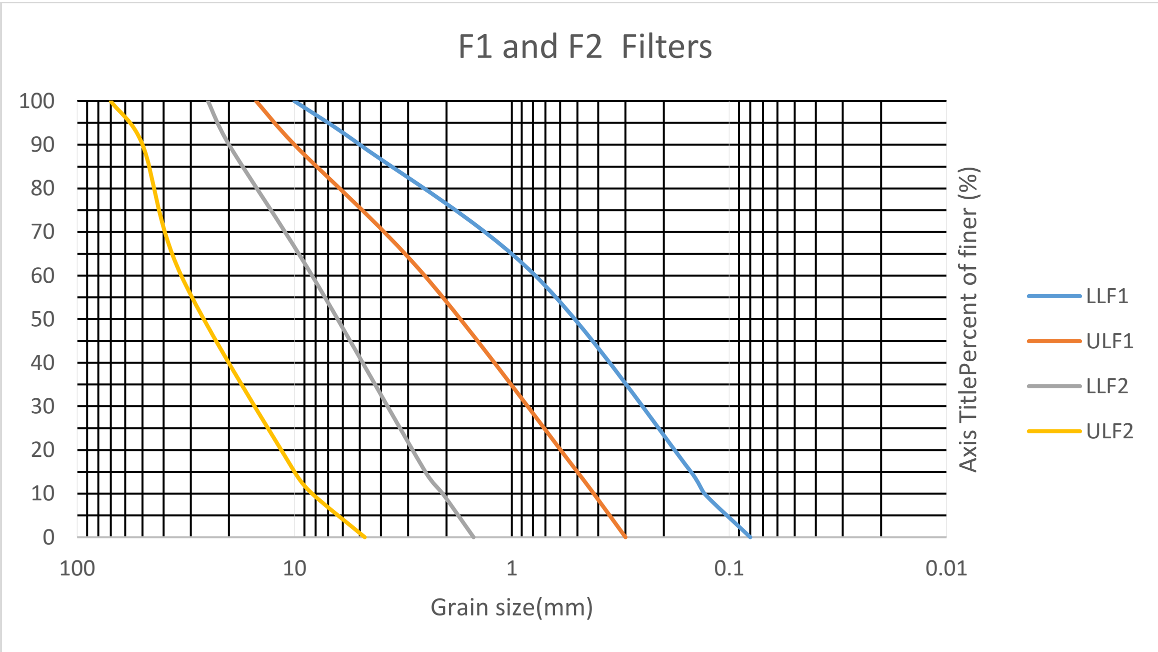


Figure 6.4 Upper and lower limits of F1 &F2

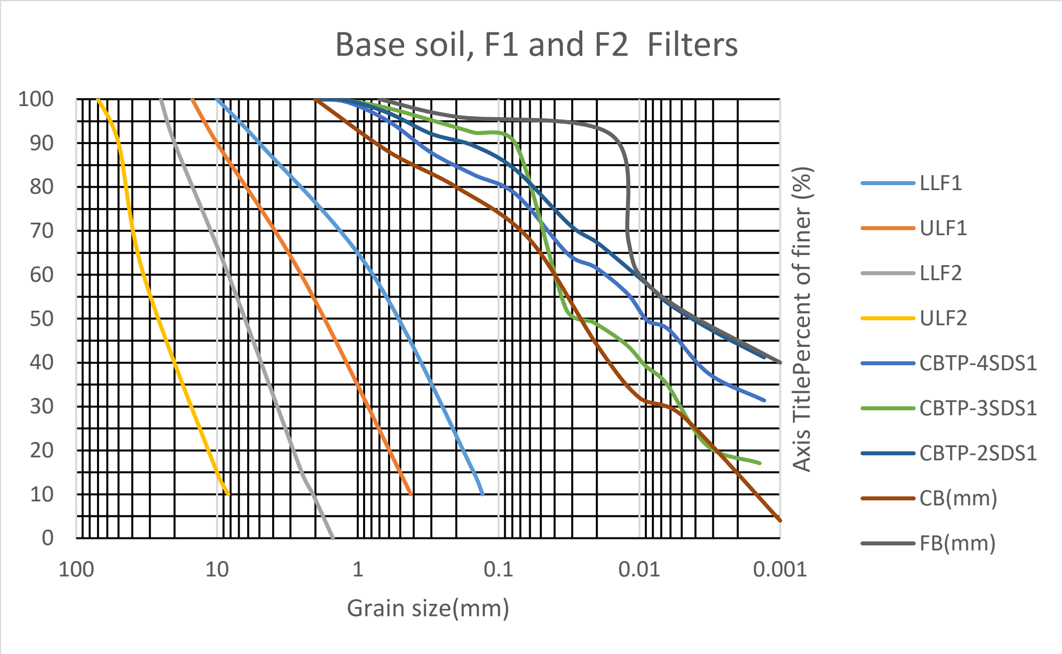


Figure 6.5 Gradation Curve of Fine F1, coarse F2 and base soil

Table 6.9 Summary of Limit of fine (F1) and Coarse (F2) filter limit

Particle Size(mm)	F1 filter		F2 filter	
	% of finer	% of finer	% of finer	% of finer
100	-	-	-	100
75	-	-	100	100
60	-	-	100	95
35	-	100	100	63
20	-	100	90	34
15	-	100	77	25
10	100	90	64	15
9	98	86	62	11
6	92	74	45	3.5
4	83	66	30	0
2	69	43	19	0
0.6	47	12	0	
0.2	19	0		
0.01	0			

Transition zone

For the Fato Dam project, there were no appropriate shell or transition materials within a reasonable distance. Therefore, the option was to prepare shell materials of adequate size from locally available basaltic rocks. This part was the transition zone of the embankment. The size of the transition zone material was prepared with a maximum size of 60 mm by using the criteria of USSD, 2011.

6.2.4 Rock fill Materials, Riprap and Crushed Coarse Aggregate

Using rocks for filling considered that weak rocks were used in less critical zones while strong rocks were used in areas where strength was required. From the laboratory test, the specific gravity value was greater than 2.6 for FSRQ1 and FSRQ2, which indicated that sound-quality rocks could be stable in place (USBR, 2014). These rocks also had a water absorption test value of less than 1%. The bulk density and point load test results were higher when compared to FSRQ3. Therefore, the two quarry site samples indicated that it was used for rock fill parts based on order of importance in their strength and permeability. In the case of FSRQ3, the specific gravity value was less than 2.6, which means that the rock was less durable with a higher potential for displacement added by wave action, and it was used for riprap according to the USBR design standard.

The source of material for riprap was similar to the rock fill except that the size was slightly coarser than the rock fill materials. It can be from the same quarry sources if it meets the criteria required. Table 6.10 shows coarser bands (gradation) for both rock fill and riprap materials.

Table 6.10 Laboratory test results for rock and crushed aggregate

Parameters	Sample ID					
	FSRQ1 BDS1	Location	FSRQ2 RS2	Location	FSRQ3 RS3	Location
Specific gravity	2.97	E 355223 N 982250	3.08	E 357121 N 981469	2.56	Fato river from Surface
Bulk density(gm/cc)	2.55		2.42		2.83	
Water absorption (%)	0.77		0.61		1.3	
Point load(Mpa)	8.78		7.65		5.34	

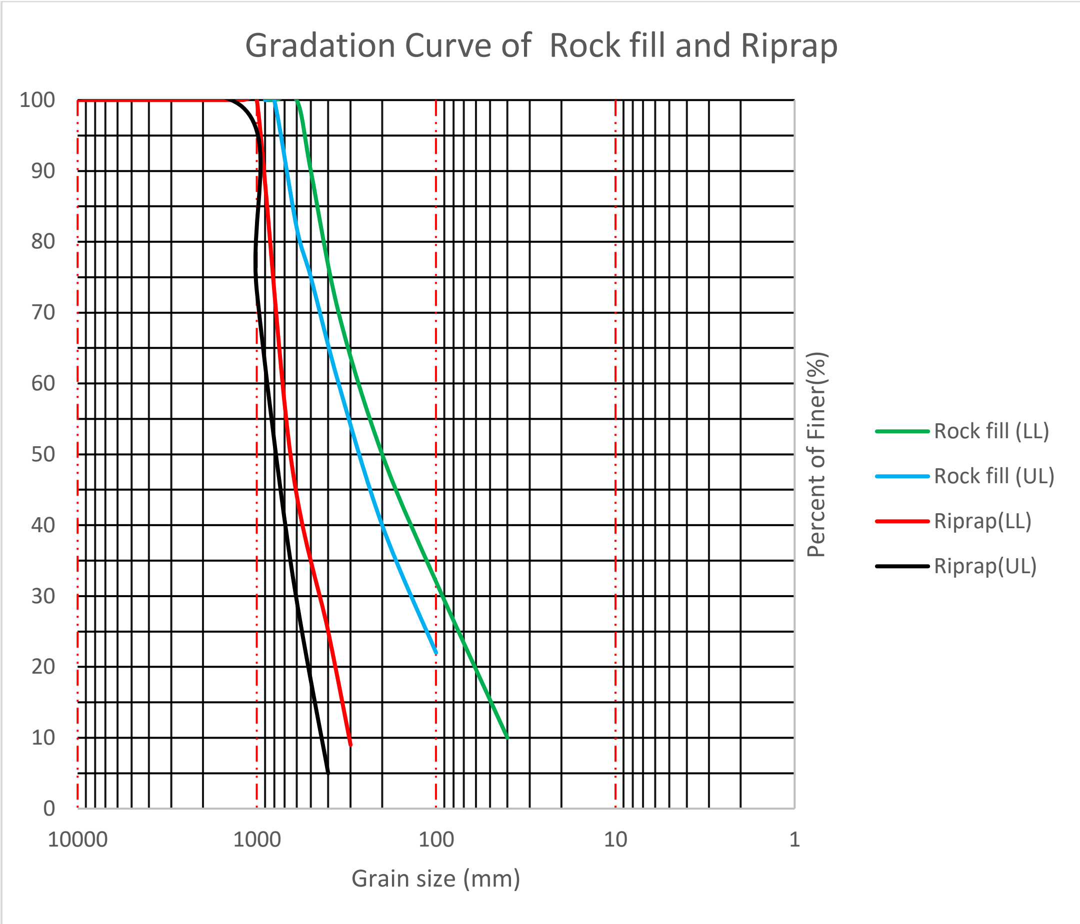


Figure 6.6 Gradation Curve of Riprap and Rock fill based on their size

Table 6.11 Rock fill and riprap particle gradation

Particle size(mm)		900	800	600	500	350	200	100	40
Rock fill	% finer	100	100	100	90	70	50	32	10
	% finer	100	100	82	75	60	40	22	0
Particle size(mm)		250	300	400	650	1000	1200	15000	
Riprap	% finer	0	9	25	50	100	100	100	
	% finer	0	0	5	35	73	99	100	

6.3 Overall Suitability of the Construction Materials

The suitability of construction materials is mainly measured by three parameters. These are the quality, quantity, and distance of transportation. The quantity of materials required is estimated depending on how large the project is. The quality of materials is based on the standards of the materials performed worldwide, and the distance is analyzed by considering the cost of transportation. A large ridge-forming basaltic unit was available on the site. So there was no shortage of materials for processing filter materials. But it requires standard processing, as it is not naturally available in well-graded or filter-form sizes.

The construction materials for most of the embankment were located within less than 1 km. Therefore, the occurrence of materials near the site made it suitable for the project to minimize the cost of transportation. However, the sand materials proposed initially for the filter and transitional zones were far from the site, that is, around 30 km from the dam site.

From the eqn 2.9 coefficient of the permeability of filter calculated based on coefficient of uniformity (1.5 to 8) and table 6.12 (Cu = 4-6). Therefore, the coefficient of permeability (k) was:

$K = 0.35(D15f)^2$

$K = 0.35(0.15)^2$ for f1 lower limit

$K = 0.007875 cm/s = 7.875 * 10^{-3} cm/s$. While for upper limit of f2 $k=0.35(10)^2=35 cm/s$.

Rock fill, riprap and crushed for each part the strength point load strength index (5 to 9 Mpa) uniaxial compressive strength varies from (95 to 178 Mpa) converted by using Karman and Kesimal, 2012 empirical relation equation of various rocks below.

UCS = 20.42(PLI (Map))-5.146 -------------------------Eqn 6.1.

Therefore, the strength of rock fill, riprap, and crushed rocks for each zone had high strength, as stated by the USSD (2011) range (UCS from 70 to 200 MPa). From the above gradation curve (figure 6.6), the size of riprap materials is larger. This was because it was used at the slope of the embankment to prevent the wave action of water.

Table 6.12 Analysis of filter criteria of the site from gradation curve

Criteria	Characteristic		Filter materials							
			LLF1		ULF1		LLF2		ULF2	
			R	C	R	C	R	C	R	C
TERZAGHI	$\frac{D15\ of filter}{D85\ 0f\ protected\ layer} < 4to5$		0.0429	S	0.148	S	0.3125	S	0.923	S
	$\frac{D15\ of filter}{D15\ of protectedlayer} > 4to5$		68.18	S	3.33	NS	5	S	4	S
INDIAN standard	$\frac{D15\ of filter}{D85\ of base} < 5$		0.0429	S	0.148	S	0.3125	S	0.923	S
	$\frac{D15\ of filter}{D15\ of base} > 4\ \&\ < 20$		68.18	NS	3.33	NS	5	S	4	S
	$\frac{D50\ of filter}{D50\ of base} < 25$		20.8	S	3.46	S	3.61	S	4	S
USBR	$R50=\frac{D50\ of fiter}{D50\ of\ base}$	1 R_{50} CU(5 to 10)	20.8	NS	3.46	NS	3.61	NS	4	NS
		2 WG-PG sub-rounded R_{50} (12 to 58)	20.8	S		NS		NS		NS
		3 WG-PG angular particles R_{50}(4 to 30)	20.8	S		NS		NS		S
	Ratio R_{15}= $\frac{D15\ of\ filter}{D15\ 0f\ base}$	WG-PG (Sub-rounded) R15(12-40)	68.18	NS	3.33	NS	5	NS	4	NS
		WG-PG(angular particles) R_{15}(6-18)		NS		NS		NS	4	NS
	$CU = \frac{D60}{D10} = 3 - 4$		6	NS	6	NS	3.99	S	4	S

6.4 Seepage flow analysis by plaxis software

In the present study, plaxis 2D version 8.5 was used to model the seepage of the Fato Dam Foundation and the materials used in each zone of embankment. So the results of the analysis expressed in the model of seepage flow through the foundation and through the embankment body of the dam are discussed in the following explanation. The flow in each medium was governed by Darcy's law for plaxis 2D. Therefore, the equations used in analyzing different parameters described in the plane of X-Y as follows.

$q_x = K_x \frac{\partial \phi}{\partial x}$……………………………………………………………………..Eqn5.5

$q_y = K_y \frac{\partial \Phi}{\partial y}$...Eqn5.6

$\phi = Y + \frac{P}{\Upsilon w}$……………………………………………………………………………Eqn5.7

Where y=vertical elevation, ф= Ground water head, q= discharge along horizontal and vertical(X, Y) direction.

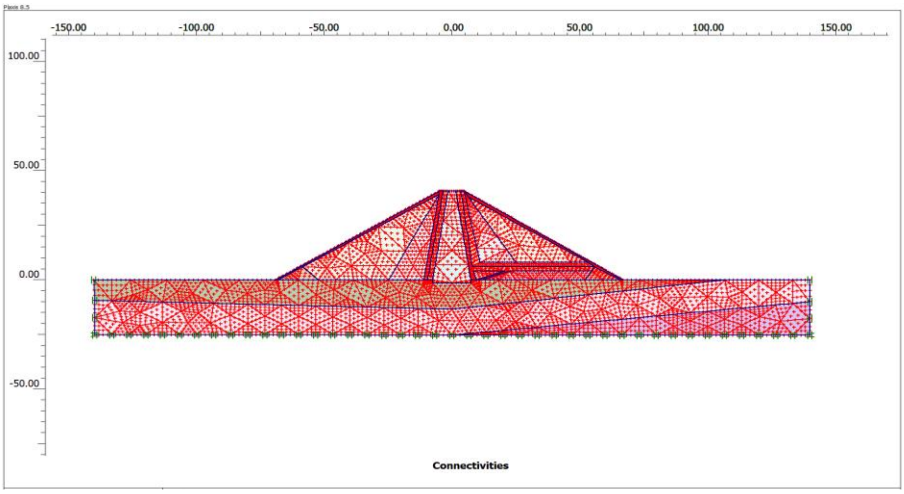


Figure 6.7 Finite element model of 2D illustration of Fato embankment dam

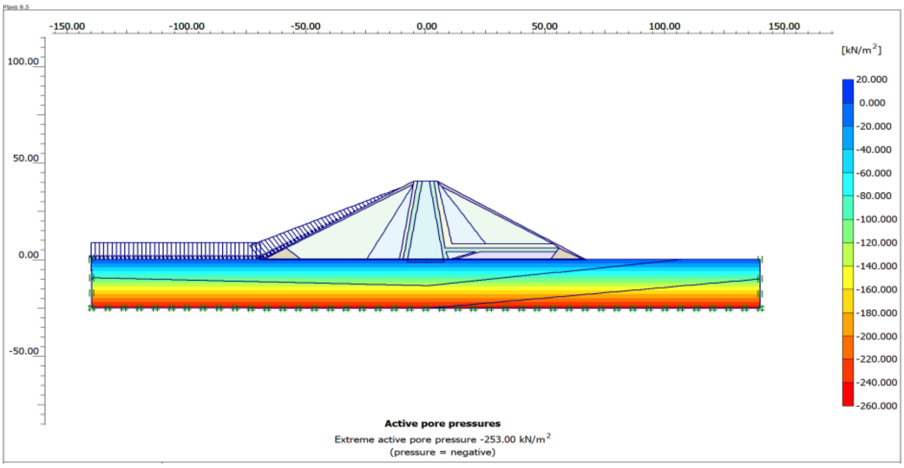


Figure 6.8 Active pore pressure distribution of foundation bed

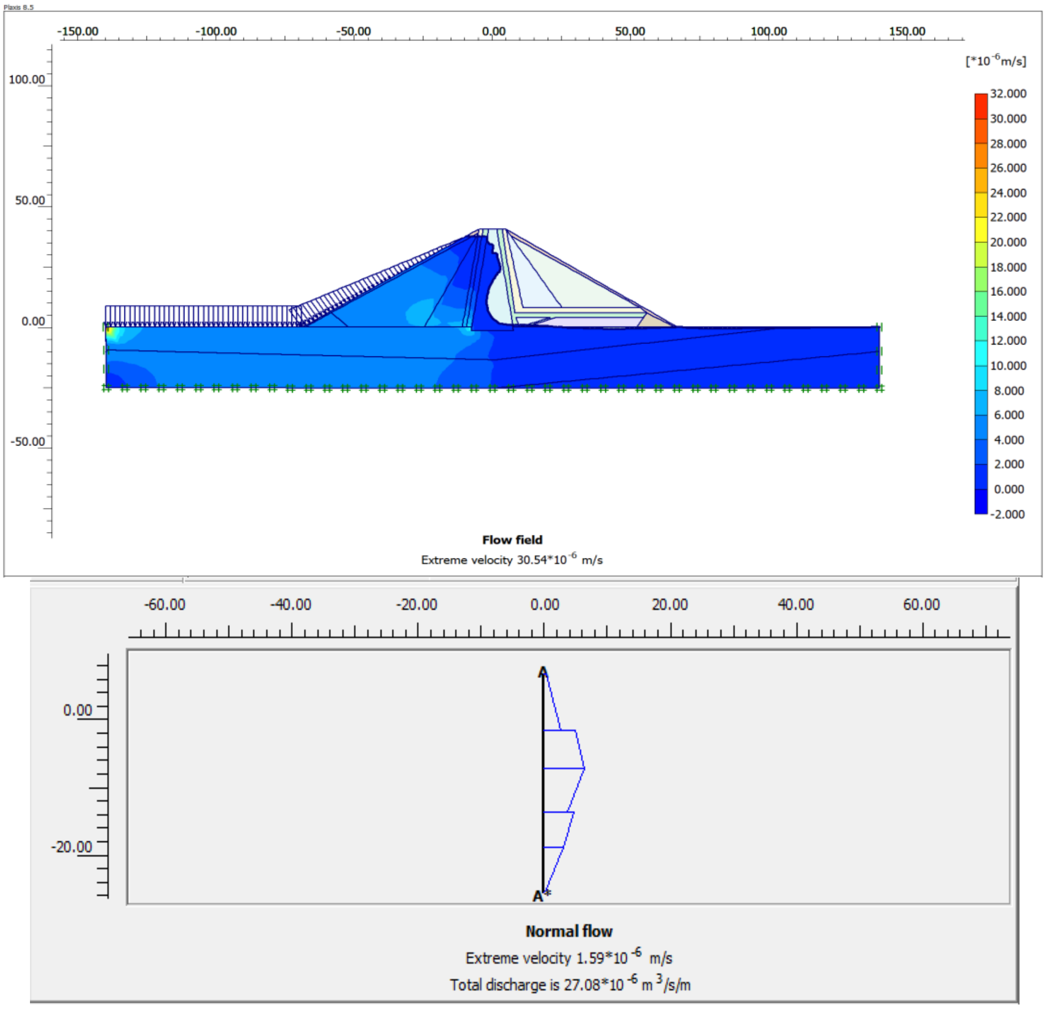
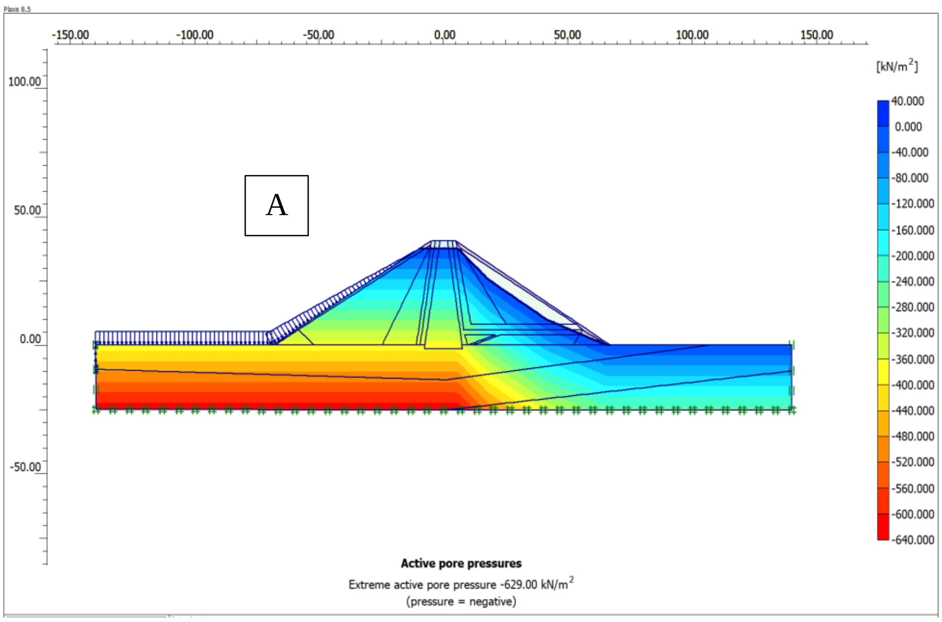


Figure 6.9 seepage flow (Steady state) when phreatic line at dam foundation level



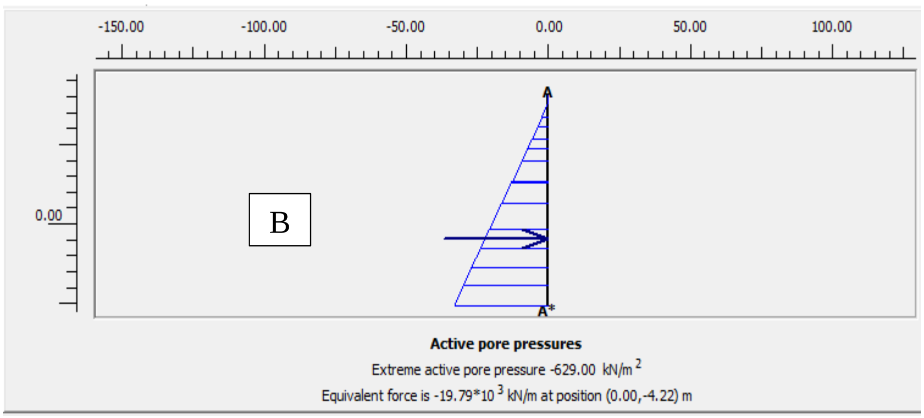


Figure 6.10 Distribution (A) and Cross section (B) of Active pore pressure at phreatic level at RFL

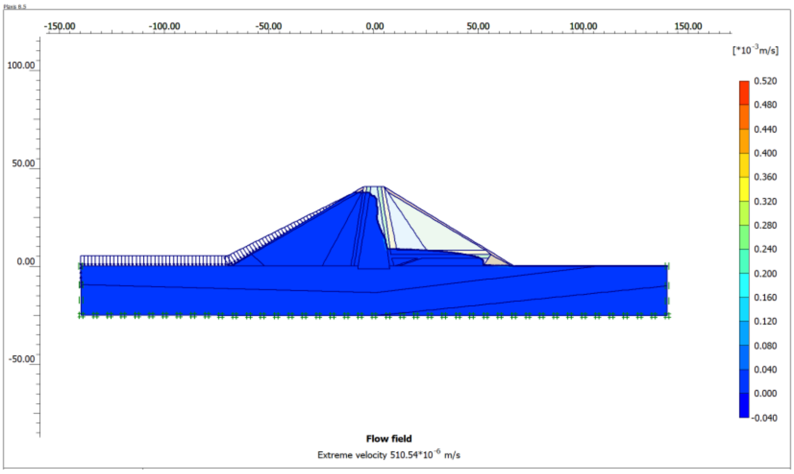


Figure 6.11 seepage flow (steady state) when phreatic line location at RFL

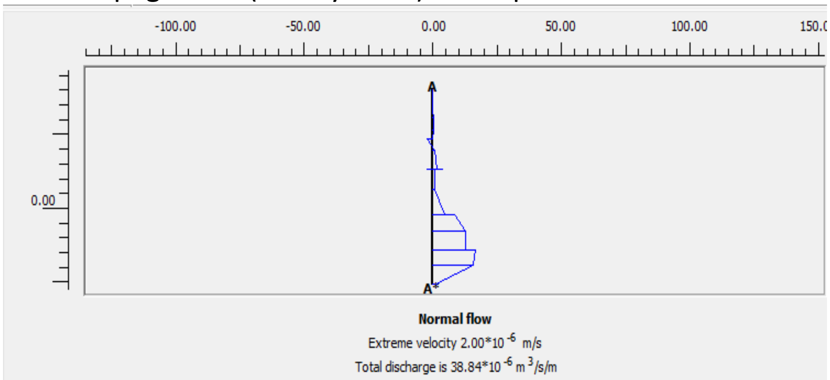


Figure 6.12 Seepage flow quantity in embankment dam body and foundation

Seepage flow when phreatic line 25 m above foundation level

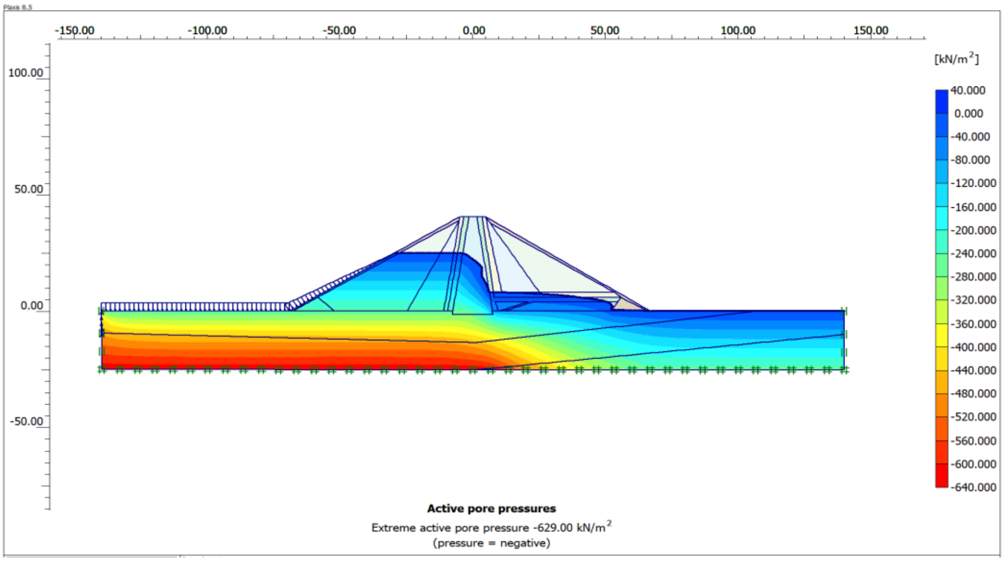


Figure 6.13 Active pore pressure distribution when phreatic line at 25m

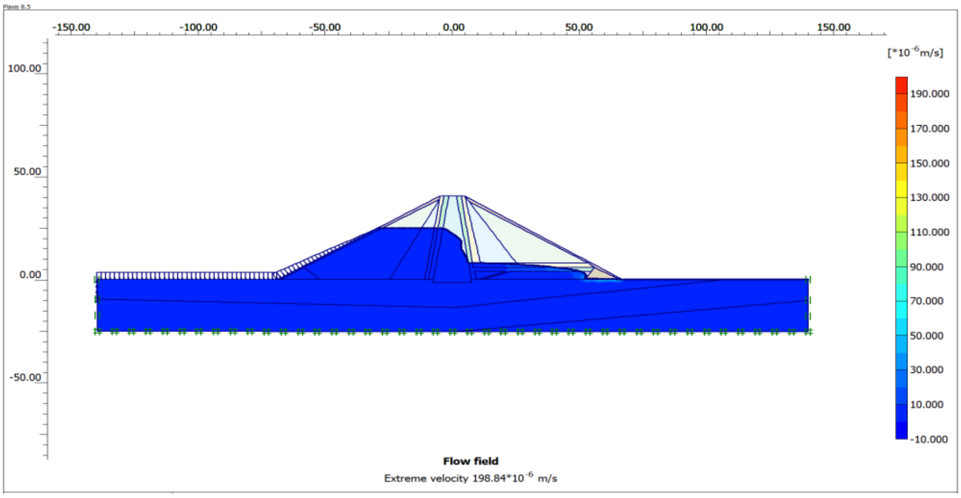


Figure 6.14 Seepage flow when GW head at 25m above foundation

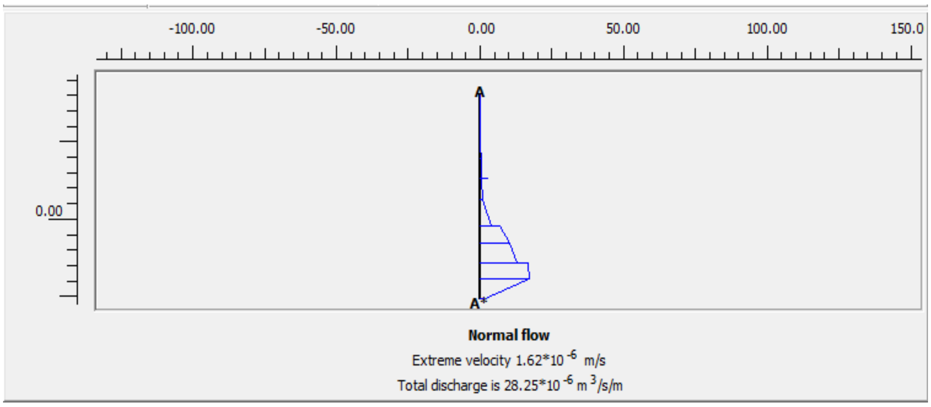


Figure 6.15 Seepage quantities for phreatic line at 25m above foundation.

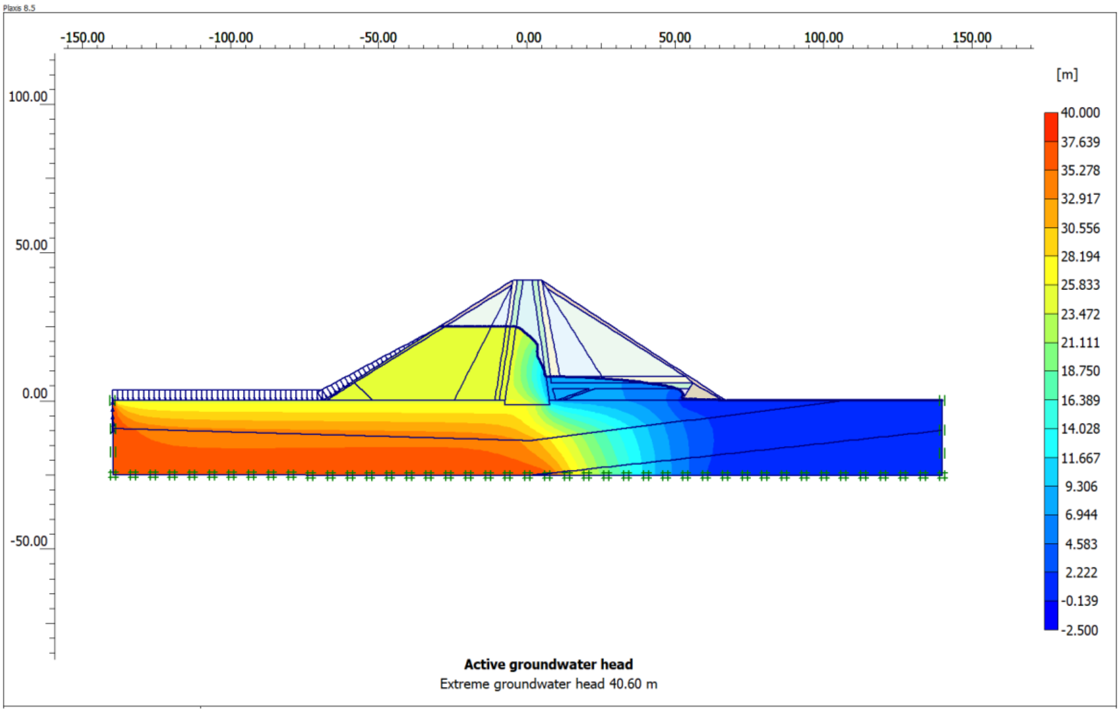


Figure 6.16 Ground water head models for phreatic line at 25m

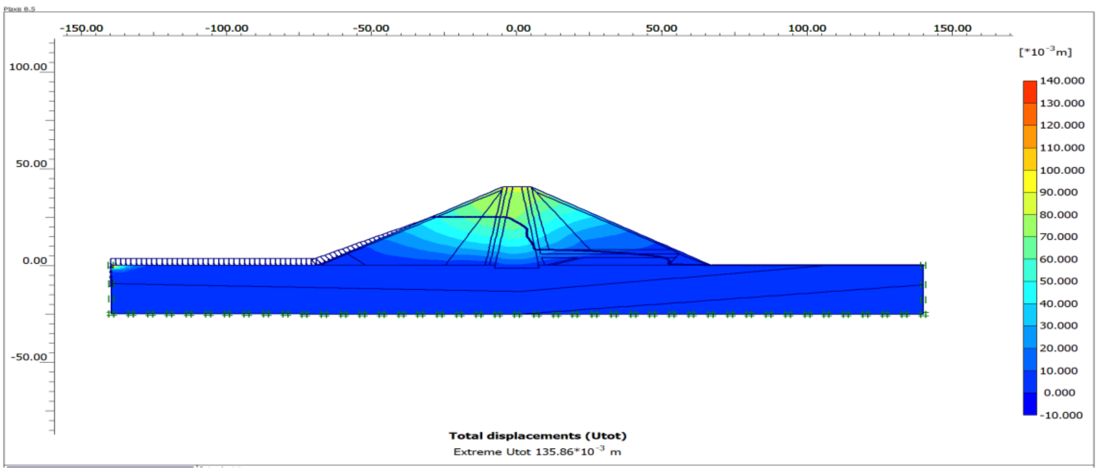


Figure 6.17 total displacement steady state Seepage when GW head at 25m

In this case flow was slowly reduced from higher (RFL) level to lower level in phreatic reduction.

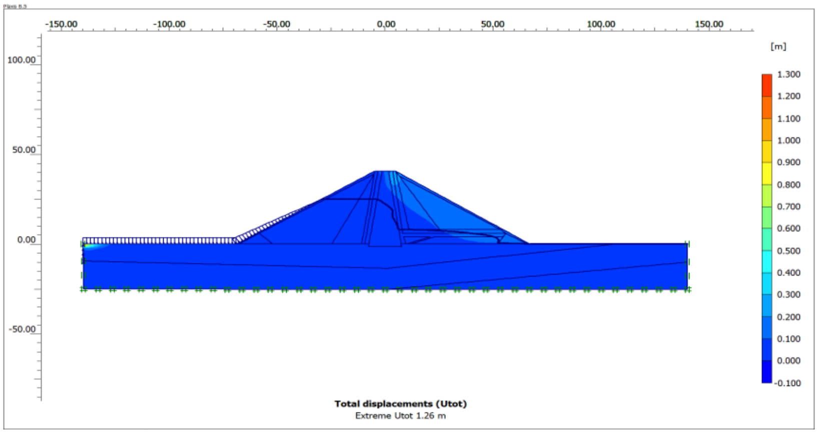


Figure 6.18 Total displacements in phreatic reduction flow type

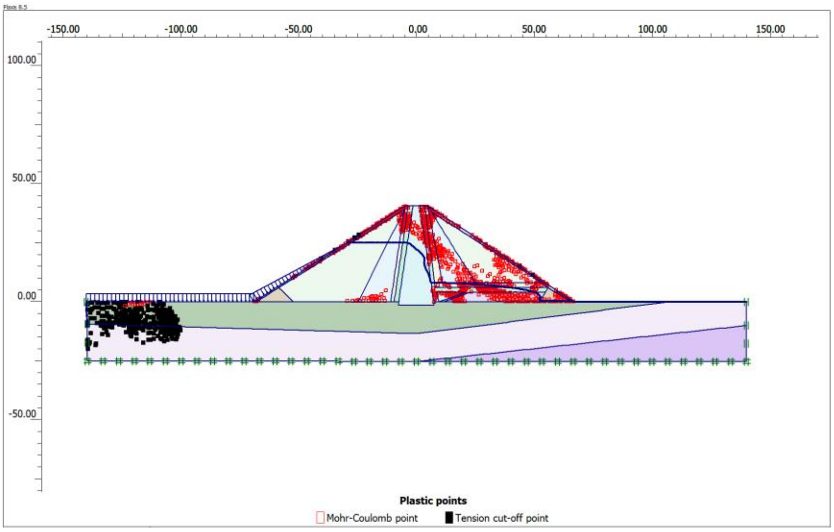


Figure 6.19 Plastic points at 25m GW head

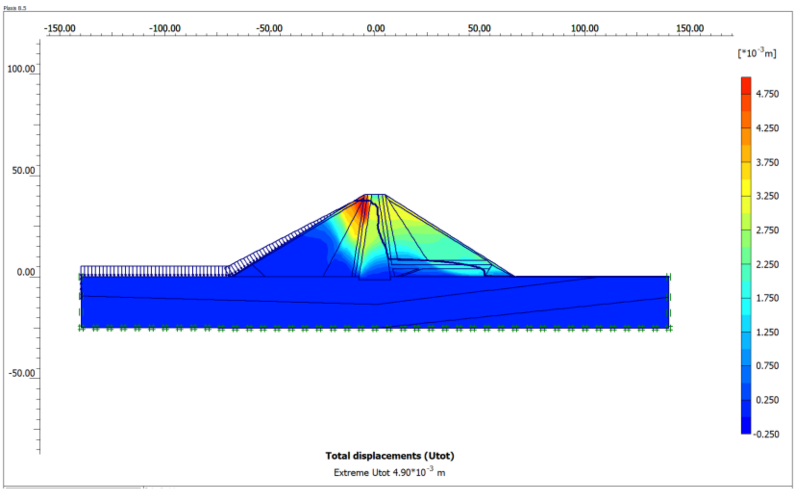


Figure 6.20 total displacement in dynamic condition

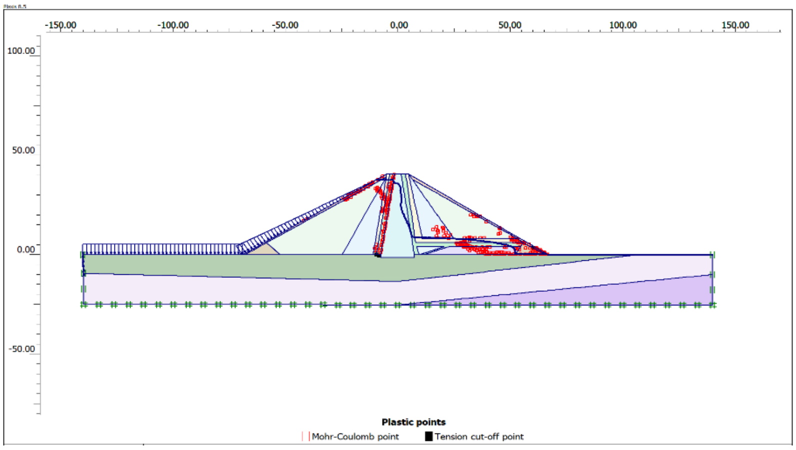


Figure 6.21 Plastic point in dynamic condition

6.5 Summary of Seepage flow analysis from plaxis results

From the analysis performed above, the result of seepage flow at the foundation level was based on 592 elements, 4929 nodes, 7104 stress points, and an average element size of $5.58*10^0$. The result of the seepage analysis of the present study is summarized in the next table (6.14).

The seepage modeling was based on the parameters determined from boreholes and construction material tests. Each material was identified by cluster areas. The parameters of each individual cluster are taken as homogenous. The parameters used through seepage modeling were Young's modulus (E), permeability (k), bulk unit weight (Y_γ), shear strength parameters C & ɸ) and Poisson ratio (v) of each material.

Table 6.13Material clusters and their specified parameters for seepage Modelling

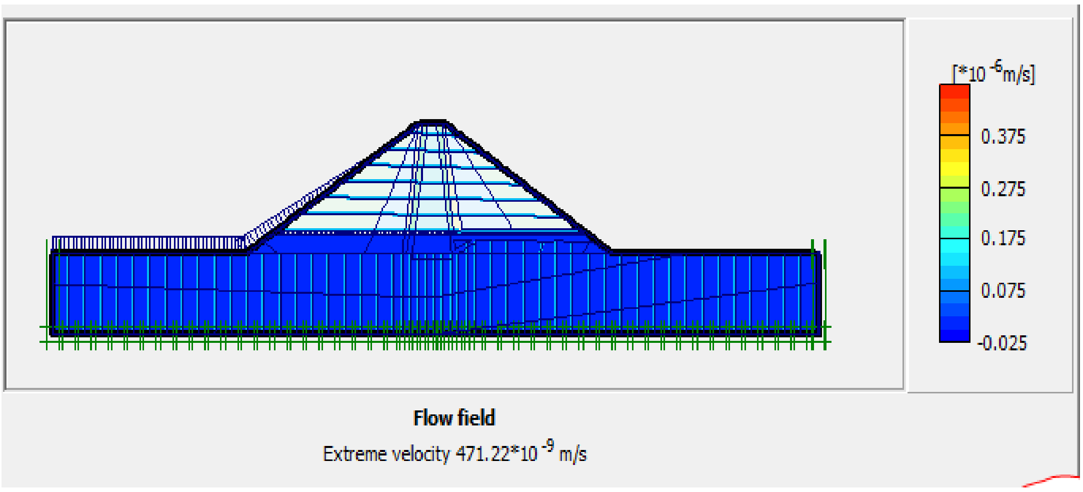
Materials /Clusters	Unit weight($KN/m^{3)}$	Permeability K(m/s)	Modulus E(Kpa)	Angle of friction ɸ(degree)	Cohesion C(Kpa)	Poisson Ratio(V)	Constant KO=1-Sinɸ
Clay core	18	$5.38 * 10^{-8}$	20000	20	20	0.35	0.65
Fine filter	19	$1.82 * 10^{-4}$	44000	32	0	0.37	0.47
Coarse filter	21	$2.03 * 10^{-4}$	44300	34	0	0.3	0.44
Transition zone	22	$1.678 * 10^{-4}$	44600	34	0	0.25	0.44
Rock fill	22	$1.0 * 10^{-2}$	45000	42	0	0.2	0.33
Riprap	22	$1.0 * 10^{-2}$	45000	42	0	0.2	0.33
Rock Toe	22	$1.0 * 10^{-2}$	45000	44	0	0.2	0.30
Top soil	16	$2.63 * 10^{-5}$	31000	20	20	0.35	0.65
Vesicular basalt	25.7	$1.944 * 10^{-6}$	5000000	38.6	280		0.37
Amygdaloidal basalt	26.2	$6.181 * 10^{-6}$	4100000	38	330	0.2	0.38
Aphanitic basalt	26.8	$7.546 * 10^{-6}$	4400000	38	360	0.2	0.38

Table 6.14 Summary of Seepage analysis result by plaxis 2D software without remedial measures

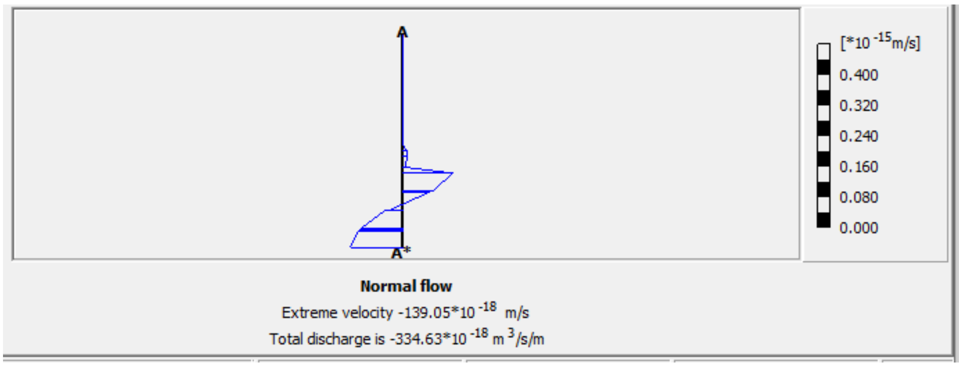
Phreatic level	Parameters	Result	Unit
Foundation	Active pore pressure	-253	KN/m^2
	Extreme velocity	$30.54 * 10^{-6}$	m/s
	Top to bottom x-section of seepage velocity and discharge(q)	$1.59 *10^{-6}$	m/s
		$27.08 * 10^{-6}$	$m^3/s/m$
Reservoir full level(RFL)	Active pore pressure	-629	KN/m^2
	Extreme velocity	$510.54 * 10^{-6}$	m/s
	x-section from top to bottom extreme velocity and total discharge	$2.00 *10^{-6}$	m/s
		$38.84 * 10^{-6}$	$m^3/s/m$
At 25m above foundation	Active pore pressure	-629	KN/m^2
	Extreme velocity	$198.84*10^{-6}$	m/s
	x-section from top to bottom extreme velocity and total discharge	$1.62*10^{-6}$	m/s
		$28.25*10^{-6}$	$m^3/s/m$

The discharge seepage quantity must be lower than 0.56 L/min/m for embankment dams taller than 40m (Look, 2014). In another case, Rezagholilou and Khorasani (2007) conclude that the seepage quantity beneath the dam is not tolerable when it is greater than 2.66*10-4 m3/s/m. Fato Dam is taller than 40m at its maximum height. From the above analysis, seepage quantity varies from 27.08 * 10-6 m3/s/m (1.62 L/min/m) to 38.84 * 10-6 m3/s/m (2.33 L/min/m). Therefore, it does not satisfy the Seepage Quantity Criteria (Look, 2014). According to Rezagholilou and Khorasani (2007), the seepage quantity is satisfied because 2.708*10-7 m3/s/m is less than 2.66*10-4 m3/s/m. Overall, the foundation of the dam had seepage problems due to localized fracturing of bedrock, especially in the left abutment, which was very critical.

After using remedial measures of curtain grouting (secondary in less critical area, tertiary in around river bed) and consolidating grouting in some places where fractured materials were dominant. Then the discharge was reduced to $3.4*10^{-16}m^3/s/m$ and normal flow was reduced to $4.71*10^{-7}m/s$. Therefore, by using remedial measures the criteria of seepage from plaxis standard was satisfied.



A



B

Figure 6.22 Seepage flow (A) and seepage quantity (B) after using remedial measures at RFL

As observed from the model and cross-section, the amount of seepage flow was decreased after using improvement methods by plaxis software. The amount of seepage discharge was became less than 0.56L/min/m. Hence, the criteria of seepage quantity was reached to tolerable standards of plaxis software, after remedial measures used for improvement.

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CHAPTER VII CONCLUSION AND RECOMMENDATION

7.1 Conclusion

The present study was done on the Fato Dam foundation and the construction materials used for each part of the embankment zones. The foundation was mainly evaluated in terms of seepage condition, while the characterization of construction materials was based on its suitability according to the standards required for each zone.

The geology of the foundation from bore holes, laboratory data, a geophysical survey, and site observation generally concluded as follows:

- ✓ Residual soils were overburdening the rock formation of the site.
- ✓ Three types of basaltic rocks occupied the area: vesicular, amygdaloidal, and aphanitic, from top to bottom, horizontally extended to both abutments of the dam axis.

Left abutment

- The soil thickness below the earth's surface was around 8.6 m.
- Weathered and fractured vesicular basalt until the depth of 12 m, following the soils.
- Amygdaloidal basalt with medium strength and filled with calcite (to a depth of 36.15 m) and Aphanitic basalt with the properties of strong and healed joint structures was associated with this unit. It covered from 36.15 to 42.2 m of the final core drilled.

Right abutment

- ✓ Residual soil thickness was about 31.15m from the earth's surface, and the top part included organic material formed from the decay of plants.
- ✓ Below the soils, vesicular basalt extended to a depth of 40.30 m, and an aphanitic basalt unit was then observed in the final core drilling run (42.6m).

Center of the dam axis

- Residual soils with a thickness of 3.40m
- Vesicular basalt (3.40 to 13.5 m) with the same properties within the left abutment
- Amygdaloidal (to a depth of 25m) and aphanitic until the core run of 31.35 m

Foundation intake area

- <1 m of alluvial soil overburdens the vesicular basalt with a thickness of 0.60 to 5.90 m.
- Amygdaloidal basalt, 5.90 to 10.30 m, and aphanitic basalt, 10.30 to 22.10 m

  Soil foundation of the site:
- Composed of gravel, sand, silt, and clay in different proportions; however, silt and clay dominate the area. It was medium-stiff on average.
- The overall range of the permeability range of the soil was 1.16*10-5 to 8.68*10-7cm/s which was categorized as semi-pervious to impervious (Falling head test result, Table 5.7 of the USBR standard).

- Unconfined strength estimated from the correlation of SPT values ranged from 50–200 Kpa through different depths of boreholes.
- Top part has low shear strength due to decay of plant. The strength of the soils shows somehow increasing around the contact with parent rock.
- The engineering soil classes as per as USCS dominantly Low to high plasticity silt (ML to MH), clay sands (SC), and low to high plasticity clay (CL to CH).
- Grain size shows from gravel size to clay size particles available on both sides of the abutments.
- Atterberg limit test result indicated that medium to high plasticity(table 5.4)

Rock foundation

Strength of rock foundation was rang from low to very strong (42 to 478 Mpa) uniaxial compressive test due to localized weathered and fractured rock formation of the site.
Therefore the result of strength test deformation

- ✓ Rock Quality Designation (RQD) value less than 25 to 100 %(Very poor to good).
- ✓ UCS test result from 42 to 478 Mpa
- ✓ Rock Mass Rating (RMR) values ranged from 47 to 76 %(table 5.10, Fair to good quality).
- ✓ Weathering condition slightly to medium especially in left abutment.
- ✓ Vertical electrical Resistivity interpretation and imaging shows that localized fluctuation of strength of rocks and occurrence of weak zones around left abutments.
- ✓ Water absorption percent greater than 2% table 5.8 (left abutment) indicates poor quality rocks.

The permeability of foundation rock is characterized as medium to very high, dominantly through different depths (Packer test results, table (5.11). The water absorption percent test result was relatively high in the left abutment (FDBH1 table 5.8). Therefore, the rock foundation has a high probability of seepage problems. Because the packer test result indicated that it was above 3 Lu (lugeon value), which was the minimum requirement for foundation, the Paxis 2D flow model analysis result in Chapter 6 also indicated that seepage flow velocity was higher. in bed rocks than the dam body. From analysis of seepage the higher the water level, indicated that the higher active pore pressure and seepage discharge. So the seepage quantity was above 0.56L/min/m without remedial measures used in plaxis modeling. But, after remedial measures of curtain grouting and consolidation grouting used, the seepage reduced tolerable standard.

Overall the gap between strength from low to high, localized fractured and weathered formation and water absorption nature of basalt formation results in deformation and seepage problems of foundation conditions.

Construction materials Most of selected quarries of construction materials localized almost within reasonable distance (<1km from the site) and it was good reduction of cost of transportation. Only the Senkelle area sand borrows was 30 km from the site.

Materials (clay) selected for impervious core was suitable because if fulfilled most criteria required by standards. From (compaction, UCS, grain size analysis, Permeability, OMC, MDD and Atterberg limits tests)

- Cohesion 42 to 60 Kpa and friction angle(φ) was from 11.31 to 23.3°.
- UCS ranged from 233 to 360 Kpa.
- Permeability was very low(10^{-7} to 10^{-8} order)
- MDD was 1.29 to 1.5g/cm^3
- OMC was 24.5 to 37.5%
- NMC was 19 to 22%.
- Percent of finer than 0.005 was 31 to 41%.
- Plasticity index shows that from medium to high plasticity.

Filter materials and transition proposed from Senkelle did not fulfil the filter criteria since it contained greater than 5% fines. The second option was processing filter material from locally available basalt was accepted because it was processed (Grinding and washing) based on the standards.

Rock fill, riprap and other aggregate fill materials selected were had Gs(2.5 to 3.08),Bulk density(2.4 to 2.84 gm/cm^3), water absorption (1.3 to 0.77) and Point load test result(5 to 9 Mpa) or 95 to 178 Mpa in UCS. So the test satisfies the standard of rock fill and riprap by ASTM, 1993 which limits:

- ✓ Maximum water absorption 2 to 6%
- ✓ Minimum specific gravity 2.3 to 2.5
- ✓ Minimum bulk unit weight 2.24 to 2.64

7.2 Recommendation

The major challenge was from deep localized weak zones, as explained from geophysical interpretations and borehole cross-sections. Therefore, the following measures should be taken to improve the strength and water-tightness of foundations and construction materials:

- ➢ Appropriate preparation of filter and transition materials
- ➢ The compaction of the materials should be done as per the requirements for each part.
- ➢ Curtain grouting with tertiary rows with primary 12m, secondary 6 m, and tertiary 3m(river bed area under core zone) hole spacing is recommended, and combined with consolidation grouting, it improves the water tightness and strength of rocks.
- ➢ Consolidation grouting with primary rows of 12m spacing and 10m depth for weak zones in left abutments beneath the impervious core
- ➢ Concrete improvement around low shear strength in the left abutment is recommended.
- ➢ Further detail investigation is needed on the foundation, especially weak zones in the left abutment.

***

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Appendix

1 Precipitation of nearest station of Fato Dam site

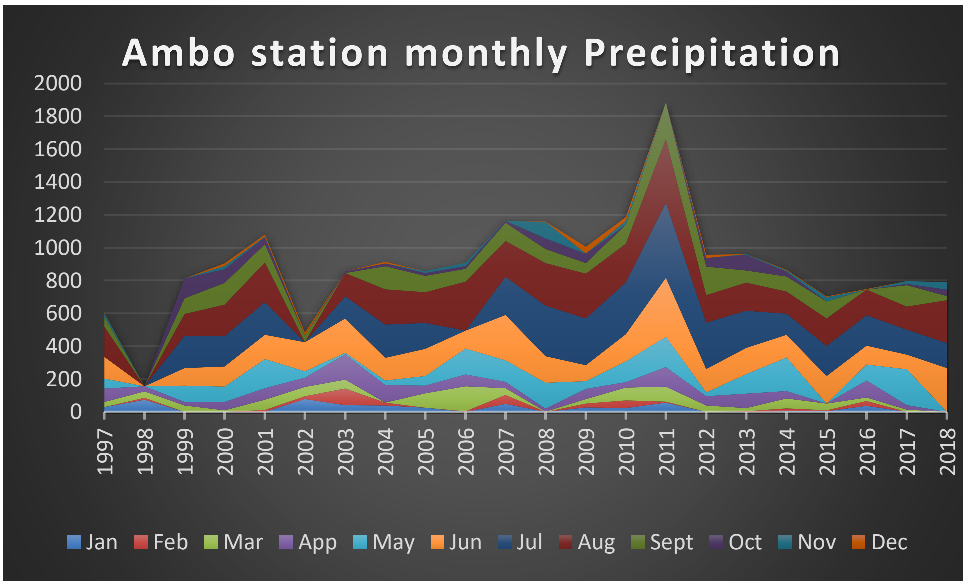


Chart shows monthly mean precipitation of Ambo station from 1997-2018

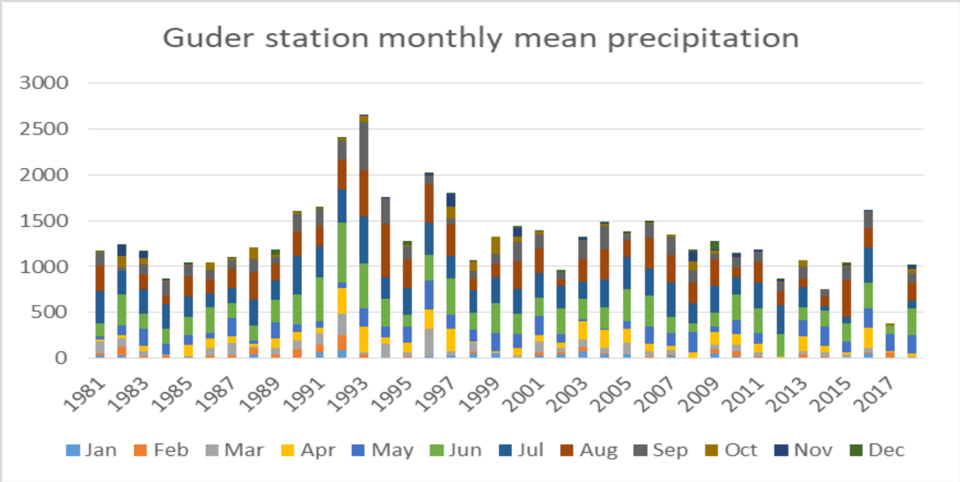


Figure monthly mean precipitation (1981-2017)

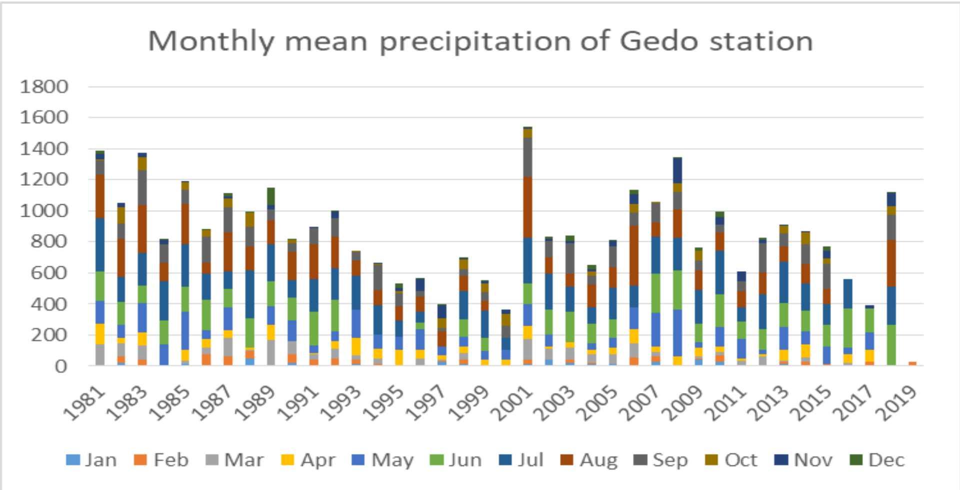


Figure Chart shows monthly precipitation of Gedo station

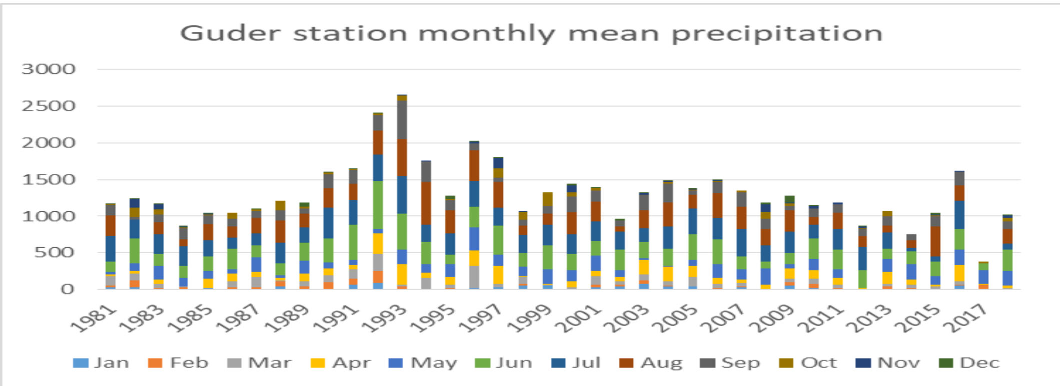


Figure Charts of Guder monthly mean precipitation from 1981 to 2018

2 Geophysical VES Model

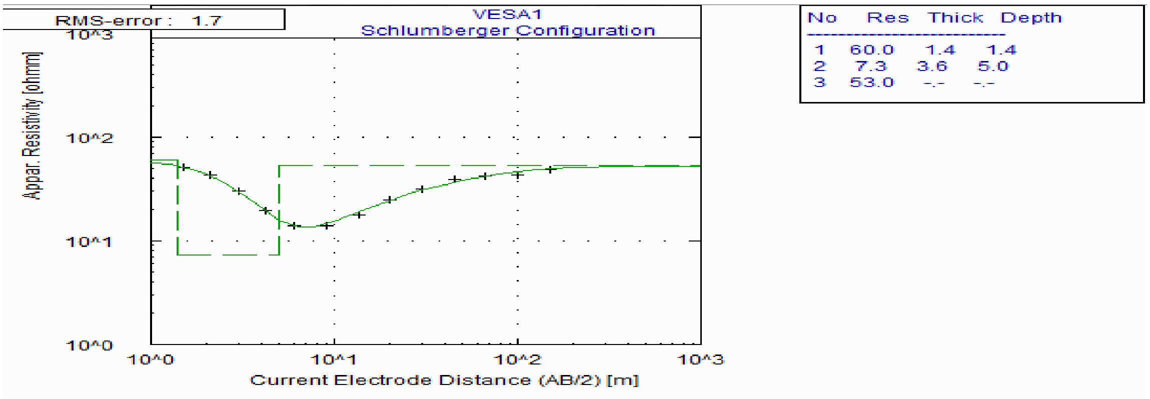


Figure Interpretation of VES model for VESA-1

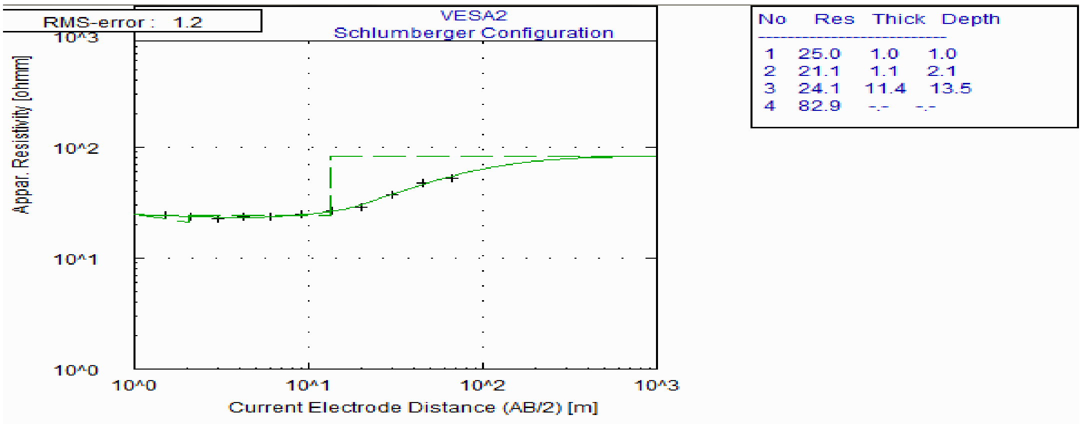


Figure Interpretation of VES model for VESA-2

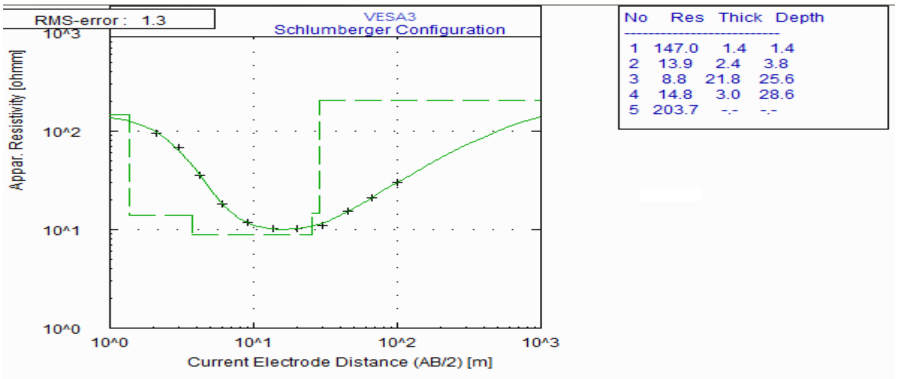


Figure Interpretation of VES model for VESA-3

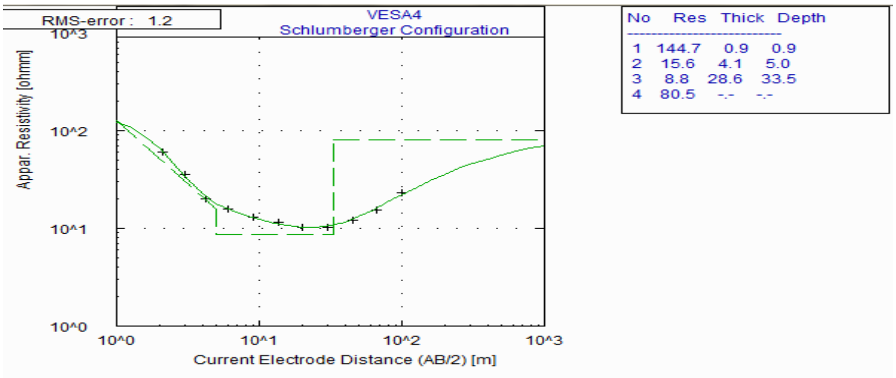


Figure Interpretation of VES model for VESA-4

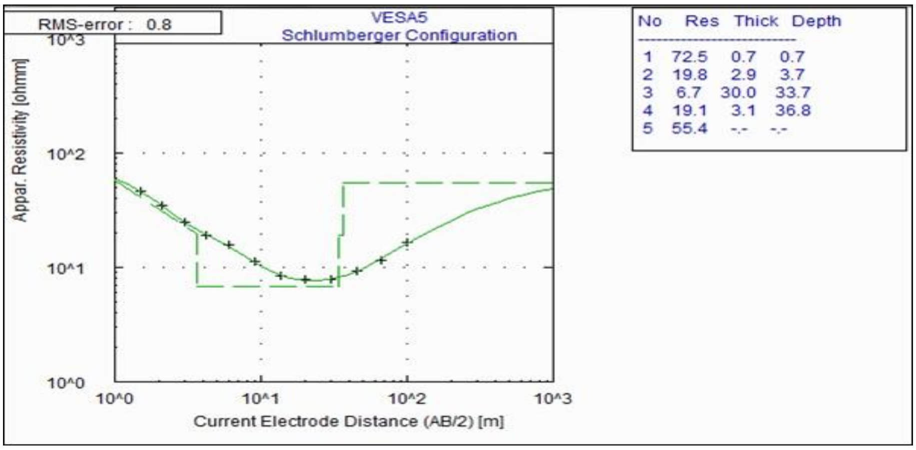


Figure Interpretation of VES model for VESA-5

3, Foundation insitu and Laboratory test results

Packer test result of boreholes

	Company Name: በኢትዮጵያ የኮንስትራክሽን ዲዛይንና ሱፐርቪዥን ሥራዎች ኮርፖሬሽን የውሃና ኢነርጂ ዲዛይንና ቁጥጥር ሥራዎች ዘርፍ **Ethiopian Construction Design & Supervision Works Corporation** **Water & Energy Design & Supervision Works Sector**		
Title: **Borehole Packer Permeability Test**	Document No: OF/WWDSE/406	Issue No. 1	Page No.

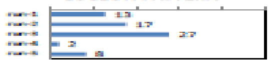
Project:	Halele-Werabesa HEPP	Guage Height from top of hole (m)	1.1
Client:	MOWIE	Swivel Height (m)	1.1
Consultant:	WWDSE	Packer position (m)	20.4
Contractor:		Packer Type	pneumatic To
Bore hole ID:	FDBH-1	Testing Method	Single
Bore hole Location:	Dam axis, Left A.	Bore hole Diam. (mm):	76
Easting (m)	360069	Groundwater level from surface (m):	9.2
Northing(m)	979608	Tested by:	
Top Elevation (a.m.s.l.)	2418	Supervised by:	Asebe & hailu
Bore hole Depth (m):	25	Test No.	2
Test Section (m):	20.4-25	Testing Date:	28/01/2016
Test Section length (m):	4.6		

Testing Time			Water Meter Reading (Lit)		Rate (lit/Min)	Testing Pressure (bars)			Lugeon (each Run)
Start (HH:MM)	Finish (HH:MM)	Elapsed (Min)	Start	Finish		Applied Gauge Pressure	Column Pressure	Total Pressure	
		2	404525	404554	14.5	1.50	0.92	2.42	13
		2	404554	404571	8.5				
		2	404571	404600	14.5				
		2	404600	404637	18.5				
		2	404637	404665	14				
		2	404680	404733	26.5	2.20	0.92	3.12	17
		2	404733	404773	20				
		2	404773	404835	31				
		2	404835	404888	26.5				
		2	404888	404930	21				
		2	405000	405102	51	2.90	0.92	3.82	27
		2	405102	405195	46.5				
		2	405195	405291	48				
		2	405291	405386	47.5				
		2	405386	405481	47.5				
		2	405597	405533	-32	2.20	0.92	3.12	2
		2	405533	405556	11.5				
		2	405556	405580	12				
		2	405580	405602	11				
		2	405602	405624	11				
		2	405633	405652	9.5	1.50	0.92	2.42	8
		2	405652	405670	9				
		2	405670	405689	9.5				
		2	405689	405706	8.5				
		2	405706	405724	9				

Representative Lugeon value (Lu): 10

Lu = (10*Q)/(P*L)
Lu = Lugeon Unit
Q = average water intake rate in lit/min
P = total pressure in bar
L = length of test section in meter

Parameter	Run-1	Run-2	Run-3	Run-4	Run-5
P	2.42	3.12	3.82	3.12	2.42
Q	14	25	48.1	2.7	9.1
L	4.6	4.6	4.6	4.6	4.6
Lu	13	17	27	2	8

LUGEON PATTERN


Dilation

Permeability (k, cm/s): 1.09E-04

Remark

Prepared by	Checked by	Approved by

	Company Name: በኢትዮጵያ የኮንስትራክሽን ዲዛይንና ሱፐርቪዥን ሥራዎች ኮርፖሬሽን የውሃና ኢነርጂ ዲዛይንና ቁጥጥር ሥራዎች ዘርፍ **Ethiopian Construction Design & Supervision Works Corporation** **Water & Energy Design & Supervision Works Sector**		
Title: **Borehole Packer Permeability Test**	Document No: OF/WWDSE/406	Issue No. 1	Page No.

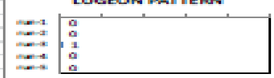
Project:	Halele-Werabesa HEPP	Guage Height from top of hole (m)	1
Client:	MOWIE	Swivel Height (m)	0.5
Consultant:	WWDSE	Packer position (m)	25
Contractor:		Packer Type	pneumatic
Bore hole ID:	FDBH-1	Testing Method	Single
Bore hole Location:	Dam axis, Left A.	Bore hole Diam. (mm):	76
Easting (m)	360069	Groundwater level from surface (m):	9.2
Northing(m)	979608	Tested by:	
Top Elevation (a.m.s.l.)	2418	Supervised by:	Asebe and Hailu
Bore hole Depth (m):	30	Test No.	3
Test Section (m):	25-30	Testing Date:	
Test Section length (m):	5		

Testing Time			Water Meter Reading (Lit)		Rate (lit/Min)	Testing Pressure (bars)			Lugeon (each Run)
Start (HH:MM)	Finish (HH:MM)	Elapsed (Min)	Start	Finish		Applied Gauge Pressure	Column Pressure	Total Pressure	
		2	406117	406117	0	1.80	0.92	2.72	0
		2	406117	406117	0				
		2	406117	406117	0				
		2	406117	406117	0				
		2	406117	406117	0				
		2	406124	406124	0	2.70	0.92	3.62	0
		2	406124	406124	0				
		2	406124	406124	0				
		2	406124	406124	0				
		2	406124	406124	0				
		2	406130	406131	0.5	3.60	0.92	4.52	1
		2	406131	406132	0.5				
		2	406132	406135	1.5				
		2	406135	406139	2				
		2	406139	406142	1.5				
		2	406141	406141	0	2.70	0.92	3.62	0
		2	406141	406141	0				
		2	406141	406141	0				
		2	406141	406141	0				
		2	406141	406141	0				
		2	406141	406141	0	1.80	0.92	2.72	0
		2	406141	406141	0				
		2	406141	406141	0				
		2	406141	406141	0				
		2	406141	406141	0				

Representative Lugeon value (Lu): <1

Lu = (10*Q)/(P*L)
Lu = Lugeon Unit
Q = average water intake rate in lit/min
P = total pressure in bar
L = length of test section in meter

Parameter	Run-1	Run-2	Run-3	Run-4	Run-5
P	2.72	3.62	4.52	3.62	2.72
Q	0	0	1.2	0	0
L	5	5	5	5	5
Lu	0	0	1	0	0

LUGEON PATTERN


Laminar

Permeability (k, cm/s): 1.13E-05

Remark

Prepared by	Checked by	Approved by

Company Name:

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Ethiopian Construction Design & Supervision Works Corporation

Water & Energy Design & Supervision Works Sector

Title:	Document No:	Issue No.	Page No.
Borehole Packer Permeability Test	OF/WWDSE/406	1	

Project:	Halele-Werabesa HEPP	Guage Height from top of hole (m)	1
Client:	MOWIE	Swivel Height (m)	0.5
Consultant:	WWDSE	Packer position (m)	30
Contractor:		Packer Type	pneumatic
Bore hole ID:	FDBH-1	Testing Method	Single
Bore hole Location:	Dam axis, Left A.	Bore hole Diam. (mm):	76
Easting (m)	360369	Groundwater level from surface (m):	9.2
Northing(m)	979608	Tested by:	
Top Elevation (a.m.s.l.)	2418	Supervised by:	Asebe & Hailu
Bore hole Depth (m):	34.7	Test No.	4
Test Section (m):	30-34.7	Testing Date:	14/02/2016
Test Section length (m):	4.7		

Testing Time			Water Meter Reading (Lit)		Rate (lit/Min)	Testing Pressure (bar)			Lugeon (each Run)
Start (HH:MM)	Finish (HH:MM)	Elapsed (Min)	Start	Finish		Applied Gauge Pressure	Column Pressure	Total Pressure	
		2	406213	406257	22	1.80	0.92	2.72	14
		2	406257	406289	16				
		2	406289	406322	16.5				
		2	406322	406356	17				
		2	406356	406389	16.5				
		2	406426	406502	38	2.70	0.92	3.62	20
		2	406502	406567	32.5				
		2	406567	406630	31.5				
		2	406630	406697	33.5				
		2	406697	406771	37				
		2	407143	407235	46	3.60	0.92	4.52	22
		2	407235	407332	48.5				
		2	407332	407428	48				
		2	407428	407523	47.5				
		2	407523	407616	46.5				
		2	407689	407761	36	2.70	0.92	3.62	19
		2	407761	407828	33.5				
		2	407828	407898	35				
		2	407898	407958	30				
		2	407958	408019	30.5				
		2	408118	408183	32.5	1.80	0.92	2.72	23
		2	408183	408239	28				
		2	408239	408295	28				
		2	408295	408348	26.5				
		2	408348	408408	30				

Representative Lugeon value (Lu)	23

$$Lu = (10*Q)/(P*L)$$

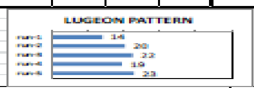
Lu = Lugeon Unit

Q = average water intake rate in lit/min

P = total pressure in bar

L = length of test section in meter

Parameter	Run-1	Run-2	Run-3	Run-4	Run-5
P	2.72	3.62	4.52	3.62	2.72
Q	17.6	34.5	47.3	33	29
L	4.7	4.7	4.7	4.7	4.7
Lu	14	20	22	19	23



Washout

Permeability (k, cm/s)	2.60E-04

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Title:	Document No:	Issue No.	Page No.
Borehole Packer Permeability Test	OF/WWDSE/406	1	

Project:	Halele-Werabesa HEPP	Guage Height from top of hole (m)	2
Client:	MOWIE	Swivel Height (m)	1.1
Consultant:	WWDSE	Packer position (m)	7.4
Contractor:		Packer Type	pneumatic
Bore hole ID:	FDBH-4	Testing Method	Single
Bore hole Location:	Upstream/Intake	Bore hole Diam. (mm):	76
Easting (m)	360265	Groundwater level from surface (m):	18.25
Northing(m)	979585	Tested by:	
Top Elevation (a.m.s.l.)	2399	Supervised by:	Asebe & Hailu
Bore hole Depth (m):	11.8	Test No.	2
Test Section (m):	7.4-11.8	Testing Date:	
Test Section length (m):	4.4		

Testing Time			Water Meter Reading (Lit)		Rate (lit/Min)	Testing Pressure (bar)			Lugeon (each Run)
Start (HH:MM)	Finish (HH:MM)	Elapsed (Min)	Start	Finish		Applied Gauge Pressure	Column Pressure	Total Pressure	
		2	419220	419347	63.5	0.60	0.96	1.56	84
		2	419347	419453	53				
		2	419453	419568	57.5				
		2	419568	419682	57				
		2	419682	419794	56				
		2	419862	419980	59	0.90	0.96	1.86	74
		2	419980	420099	59.5				
		2	420099	420219	60				
		2	420219	420350	65.5				
		2	420350	420468	59				
		2	420567	420712	72.5	1.20	0.96	2.16	75
		2	420712	420853	70.5				
		2	420853	420994	70.5				
		2	420994	421135	70.5				
		2	421135	421277	71				
		2	421355	421481	63	0.90	0.96	1.86	77
		2	421481	421613	66				
		2	421613	421735	61				
		2	421735	421863	64				
		2	421863	421986	61.5				
		2	422077	422188	55.5	0.60	0.96	1.56	80
		2	422188	422298	55				
		2	422298	422409	55.5				
		2	422409	422518	54.5				
		2	422518	422629	55.5				

Representative Lugeon value (Lu)	75

$$Lu = (10*Q)/(P*L)$$

Lu = Lugeon Unit

Q = average water intake rate in lit/min

P = total pressure in bar

L = length of test section in meter

Parameter	Run-1	Run-2	Run-3	Run-4	Run-5
P	1.56	1.86	2.16	1.86	1.56
Q	57.4	60.6	71	63.1	55.2
L	4.4	4.4	4.4	4.4	4.4
Lu	84	74	75	77	80

Turbulent

Permeability (k, cm/s)	8.48E-04

Remark

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Title:	Document No:	Issue No.	Page No.
Borehole Packer Permeability Test	OF/WWDSE/406	1	

Project:	Halele-Werabesa HEPP	Guage Height from top of hole (m)	0.5
Client:	MOWIE	Swivel Height (m)	2
Consultant:	WWDSE	Packer position (m)	12
Contractor:		Packer Type	pneumatic
Bore hole ID:	FDBH-4	Testing Method	Single
Bore hole Location:	Upstream/Intake	Bore hole Diam. (mm):	76
Easting (m)	360265	Groundwater level from surface (m):	18.25
Northing(m)	979585	Tested by:	
Top Elevation (a.m.s.l.)	2399	Supervised by:	Asebe & Hailu
Bore hole Depth (m):	17	Test No.	3
Test Section (m):	12-17	Testing Date:	
Test Section length (m):	5		

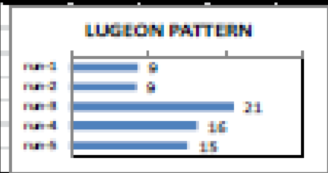
Testing Time			Water Meter Reading (Lit)		Rate (lit/Min)	Testing Pressure (bar)			Lugeon (each Run)
Start (HH:MM)	Finish (HH:MM)	Elapsed (Min)	Start	Finish		Applied Gauge Pressure	Column Pressure	Total Pressure	
		2	4237757	4237787	15	1.36	1.45	2.81	9
		2	4237787	4238813	13				
		2	4238813	4238836	11.5				
		2	4238836	4238858	11				
		2	4238858	4238879	10.5				
		2	4238890	4238921	15.5	2.04	1.45	3.49	9
		2	4238921	4238950	14.5				
		2	4238950	4238980	15				
		2	4238980	4240010	15				
		2	4240010	4240039	14.5				
		2	4240064	4241148	42	2.73	1.45	4.18	21
		2	4241148	4242232	42				
		2	4242232	4243326	47				
		2	4243326	4244414	44				
		2	4244414	4245503	44.5				
		2	4245584	4246645	30.5	2.04	1.45	3.49	16
		2	4246645	4247704	29.5				
		2	4247704	4247784	30				
		2	4247784	4248821	28.5				
		2	4248821	4248870	24.5				
		2	4248910	4248955	22.5	1.36	1.45	2.81	15
		2	4248955	4248997	21				
		2	4248997	4250040	21.5				
		2	4250040	4250082	21				
		2	4250082	4251123	20.5				

Representative Lugeon value (Lu)	12

$$Lu = (10*Q)/(P*L)$$

Lu = Lugeon Unit
Q = average water intake rate in lit/min
P = total pressure in bar
L = length of test section in meter

Parameter	Run-1	Run-2	Run-3	Run-4	Run-5
P	2.81	3.49	4.18	3.49	2.81
Q	12.2	14.9	41.9	28.6	21.3
L	5	5	5	5	5
Lu	9	9	21	16	15



Dilation

Permeability (k, cm/s)	1.41E-04

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Title: **Borehole Packer Permeability Test**		Document No: OF/WWDSE/406	Issue No. 1	Page No.

Project:	Halele-Werabesa HEPP	Guage Height from top of hole (m)	0.5
Client:	MOWIE	Swivel Height (m)	2
Consultant:	WWDSE	Packer position (m)	17
Contractor:		Packer Type	pneumatic
Bore hole ID:	FDBH-4	Testing Method	Single
Bore hole Location:	Upstream/Intake	Bore hole Diam. (mm):	76
Easting (m)	360265	Groundwater level from surface (m):	18.25
Northing(m)	979585	Tested by:	
Top Elevation (a.m.s.l.)	2399	Supervised by:	Asebe & Hailu
Bore hole Depth (m):	22.1	Test No.	4
Test Section (m):	17-22.1	Testing Date:	
Test Section length (m):	5.1		

Testing Time			Water Meter Reading (Lit)		Rate (lit/Min)	Testing Pressure (bars)			Lugeon (each Run)
Start (HH:MM)	Finish (HH:MM)	Elapsed (Min)	Start	Finish		Applied Gauge Pressure	Column Pressure	Total Pressure	
		2	425333	425420	43.5	1.66	1.825	3.485	57
		2	425420	425639	109.5				
		2	425639	425872	116.5				
		2	425872	426116	122				
		2	426116	426352	118				
		2	426452	426847	197.5	2.49	1.825	4.315	81
		2	426847	427179	166				
		2	427179	427535	178				
		2	427535	427900	182.5				
		2	427900	428225	162.5				
		2	428374	428782	204	3.33	1.825	5.155	79
		2	428782	429245	231.5				
		2	429245	429620	187.5				
		2	429620	430034	207				
		2	430034	430450	208				
		2	430606	430980	187	2.49	1.825	4.315	87
		2	430980	431360	190				
		2	431360	431718	179				
		2	431718	432106	194				
		2	432106	432529	211.5				
		2	432667	432966	149.5	1.66	1.825	3.485	90
		2	432966	433298	166				
		2	433298	433624	163				
		2	433624	433943	159.5				
		2	433943	434262	159.5				

Representative Lugeon value (Lu) 90

$$Lu = (10*Q)/(P*L)$$

Lu = Lugeon Unit
Q = average water intake rate in lit/min
P = total pressure in bar
L = length of test section in meter

Parameter	Run-1	Run-2	Run-3	Run-4	Run-5
P	3.485	4.315	5.155	4.315	3.485
Q	101.9	177.3	207.6	182.3	159.5
L	5.1	5.1	5.1	5.1	5.1
Lu	57	81	79	87	90

Washout

Permeability (k, cm/s) 1.02E-03

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Standard penetration test (SPT) Result

Company Name:

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Ethiopian Construction Design & Supervision Works Corporation
Water & Energy Design & Supervision Works Sector

Title:	Document No:	Issue No.	Page No.
Bore Hole Standard Penetration Test (SPT)	OF/WWDSE/409	1	1 of 1

Project	UGMP FATO DAM DETAIL DESIGN GEOTECHNICAL INVESTIGATION PROJECT		
Client	MOWIE	Borehole Location	LEFT ABUTUMENT
Consultant	WWDSE	Easting (UTM)	360369
Contractor	WATER WORKS DESIGN & SUPERVISION ENTERPRISE (WWDSE)	Northing (UTM)	979608
Borehole ID	FDBH-1	OGL (UTM)	2418

Test No.	Testing Depth (m)	BH Diam. (mm)	Rod Type	Rod Length (m)	Hammer Type	Sampler With Liner	Sampler Without Liner	Soil Type Description	No. of blows N_1 0-15 cm	N_2 15-30 cm	N_3 30-45 cm	N 15-45 cm	Remarks	Testing Date	Tested By	Supervisor
1	1.8	101	BW	2.7	Safety		√	Silty CLAY	3	5	6	11		10/5/2008	Kebede	Asebe & Hailu
2	3.5	101	BW	4.4	Safety		√	Silty CLAY	3	4	5	9		>>	>>	>>
3	6	96	BW	6.7	Safety		√	Sandy Silty Clay	6	,11	11	22		12/5/2008	>>	>>
4	8	96	BW	8.7	Safety		√	Gravely Sandy SILT	16	30	38	68		>>	>>	>>

Company Name:

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Water & Energy Design & Supervision Works Sector

Title:	Document No:	Issue No.	Page No.
Bore Hole Standard Penetration Test (SPT)	OF/WWDSE/409	1	1 of 1

Project	UGMP FATO DAM DETAIL DESIGN GEOTECHNICAL INVESTIGATION PROJECT		
Client	MOWIE	Borehole Location	LEFT ABUTUMENT
Consultant	WWDSE	Easting (UTM)	360325
Contractor	WATER WORKS DESIGN & SUPERVISION ENTERPRISE (WWDSE)	Northing (UTM)	979541
Borehole ID	FDBH-2	OGL (UTM)	2401

Test No.	Testing Depth (m)	BH Diam. (mm)	Rod Type	Rod Length (m)	Hammer Type	Sampler With Liner	Sampler Without Liner	Soil Type Description	No. of blows N_1 0-15 cm	N_2 15-30 cm	N_3 30-45 cm	N 15-45 cm	Remarks	Testing Date	Tested By	Supervisor
1	1.5	101	BW	2.7	Safety		√	sandy Silty Clay	3	3	4	7		21/4/08	Kebede C	Asebe & Hailu

Company Name:

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Title:	Document No:	Issue No.	Page No.
Bore Hole Standard Penetration Test (SPT)	OF/WWDSE/409	1	1 of 1

Project	UGMP FATO DAM DETAIL DESIGN GEOTECHNICAL INVESTIGATION PROJECT		
Client	MOWIE	Borehole Location	LEFT ABUTUMENT
Consultant	WWDSE	Easting (UTM)	360225
Contractor	WATER WORKS DESIGN & SUPERVISION ENTERPRISE (WWDSE)	Northing (UTM)	979409
Borehole ID	FDBH-3	OGL (UTM)	2430

Test No.	Testing Depth (m)	BH Diam. (mm)	Rod Type	Rod Length (m)	Hammer Type	Sampler With Liner	Sampler Without Liner	Soil Type Description	No. of blows N_1 0-15 cm	N_2 15-30 cm	N_3 30-45 cm	N 15-45 cm	Remarks	Testing Date	Tested By	Supervisor
1	1.5	101	BW	2.7	Safety		√	sandy silt	3	6	7	13		16.03.08	Kebede C	Asebe & Hailu
2	3.5	101	BW	4.7	Safety		√	Silty Clay	2	3	3	6		>>	>>	>>
3	6.6	>>	BW	7.7	Safety		√	sandy silty CLAY	6	8	9	17		18.03.08	>>	>>
4	8.05	>>	BW	8.7	Safety		√	clayey SAND and SILT	6	10	10	20		>>	>>	>>
5	11.3	>>	BW	12.2	Safety		√	sandy SILT with some clay	1	2	4	6		19.03.08	>>	>>
6	13.35	>>	BW	14.2	Safety		√	sandy silty CLAY	6	6	5	11		>>	>>	>>
7	15	>>	BW	15.7	Safety		√	sandy silty CLAY	5	7	8	15		20.03.08	>>	>>
8	17.1	96	BW	18.2	Safety		√	silty clayey SAND	2	5	5	10		21.03.08	>>	>>
9	19	>>	BW	20.2	Safety		√	silty CLAY with some sand	3	7	7	14		22.03.0	>>	>>
10	20.45	>>	BW	22.7	Safety		√	clayey SILT with some sand	3	5	6	11		23.03.08	>>	>>
11	22.4	96	BW	23.2	Safety		√	calyey sandy SILT	2	3	3	6		23.03.08	>>	>>
12	24.55	>>	BW	26.2	Safety		√	clayey silty SAND	4	9	0	19		>>	>>	>>
13	26.5	>>	BW	27.7	Safety		√	clayey SILT and SAND	4	6	7	13		24.03.08	>>	>>
14	29	>>	BW	30.2	Safety		√	clayey SILT and SAND	4	11	14	25		24.03.08	>>	>>

Combined sieve and hydrometer analysis result of FDBH1

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Water Works Design and Supervision Enterprise Laboratory Service Sub Process

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Soil & Material Testing Section

Project : Upper Guder Multi Purpose Project
Client : MWIE
Location : _
Test Pit : FD-BH-1SPT-2
Depth (m): 3.50-3.95

Sample Type : Disturbed
Test Type : Hydrometer
Date :22/3/2016
Lab-No : - 585/08

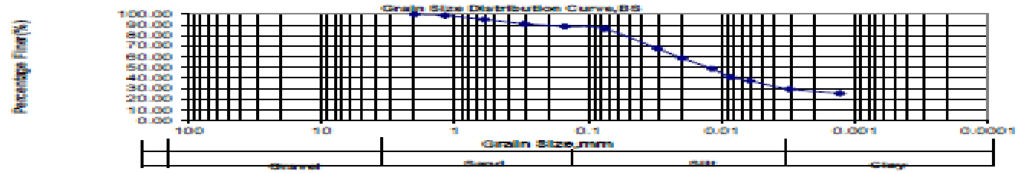
Total mass of sample, g 50

Sieve No	Sieve Opening(mm)	Mass of Sieve(g)	Mass of sieve + Ret .soil(g)	Mass of Ret. soil (g)	Percentage Retained	Cumulative % Retained	Percentage Passing
No 10	2	551.1	551.1	0.00	0.000	0.00	100.0
No 16	1.18	538.9	539.5	0.55	1.100	1.10	98.9
No 30	0.6	516.7	518.6	1.86	3.720	4.82	95.2
No 50	0.3	488.2	490.1	1.94	3.880	8.70	91.3
No 100	0.15	481.9	483.2	1.29	2.580	11.28	88.7
No 200	0.075	459.2	460.3	1.13	2.260	13.54	86.5
pan		425.5	425.5	0.00	0.000	0.00	

Hydrometer Analysis

Specific Gravity of soil 2.75 Test Temperature, deg.c 22.0

Elapsed Time (min)	Actual Hydrometer Reading	Temprature deg.c	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Percentage Finer Combined
2	40.5000	22.0	34.5000	10.65	0.01309	0.0302	67.51
5	36.0000	22.0	30.0000	11.40	0.01309	0.0198	58.71
15	31.0000	22.0	25.0000	12.20	0.01309	0.0118	48.92
30	27.0000	22.0	21.0000	12.90	0.01309	0.0086	41.09
60	25.0000	22.0	19.0000	13.20	0.01309	0.0061	37.18
250	21.0000	22.0	15.0000	13.80	0.01309	0.0031	29.35
1440	19.0000	22.0	13.0000	14.20	0.01309	0.0013	25.44



Tested by :____________________ Checked by :____________________
Processed by: ____________________ Approved by :____________________

የውሃ ሥራዎች ዲዛይንና ቁጥጥር ድርጅት
የላቦራቶሪ አገልግሎት ንዑስ የሥራ ሂደት

Water Works Design and Supervision Enterprise Laboratory Service Sub Process

የአፈር ምህንድስና ክፍል

Soil & Material Testing Section

Project : Upper Guder Multi Purpose Project
Client : MWIE
Location : _
Test Pit : FD-BH-1SPT-3
Depth (m): 8.80-8.45

Sample Type : Disturbed
Test Type : Hydrometer
Date :22/3/2016
Lab-No : - 586/08

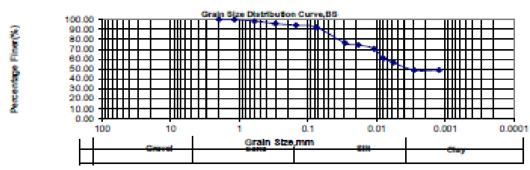
Total mass of sample, g 50

Sieve No	Sieve Opening(mm)	Mass of Sieve(g)	Mass of sieve + Ret .soil(g)	Mass of Ret. soil (g)	Percentage Retained	Cumulative % Retained	Percentage Passing
No 10	2	551.1	551.1	0.00	0.000	0.00	100.0
No 16	1.18	538.9	539.0	0.08	0.160	0.16	99.8
No 30	0.6	516.7	517.5	0.76	1.520	1.68	98.3
No 50	0.3	488.2	489.4	1.22	2.440	4.12	95.9
No 100	0.15	481.9	482.7	0.84	1.680	5.80	94.2
No 200	0.075	459.2	460.2	1.03	2.060	7.86	92.1
pan		425.5	425.5	0.00	0.000	0.00	

Hydrometer Analysis

Specific Gravity of soil 2.76 Test Temperature, deg.c 22.0

Elapsed Time (min)	Actual Hydrometer Reading	Temprature deg.c	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Percentage Finer Combined
2	45.0000	22.0	39.0000	9.90	0.01305	0.0290	76.16
5	44.0000	22.0	38.0000	10.10	0.01305	0.0185	74.21
15	42.0000	22.0	36.0000	10.40	0.01305	0.0109	70.30
30	37.0000	22.0	31.0000	11.20	0.01305	0.0080	60.54
60	35.0000	22.0	29.0000	11.50	0.01305	0.0057	56.63
250	31.0000	22.0	25.0000	12.20	0.01305	0.0029	48.82
1440	31.0000	22.0	25.0000	12.90	0.01305	0.0012	48.82



Tested by :____________________ Checked by :____________________
Processed by: ____________________ Approved by :____________________

የውሃ ሥራዎች ዲዛይንና ቁጥጥር ድርጅት
የላቦራቶሪ አገልግሎት ንዑስ የሥራ ሂደት

Water Works Design and Supervision Enterprise Laboratory Service Sub Process

የአፈር ምህንድስና ክፍል

Soil & Material Testing Section

Project : Upper Guder Multi Purpose Project
Client : MWIE
Location : _
Test Pit : FD-BH-1BPT-4
Depth (m): 6.0-6.45

Sample Type : Disturbed
Test Type : Hydrometer
Date :22/3/2016
Lab-No : - 587/08

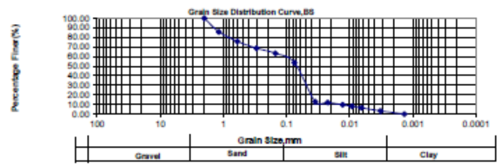
Total mass of sample, g 60

Sieve No	Sieve Opening(mm)	Mass of Sieve(g)	Mass of sieve + Ret .soil(g)	Mass of Ret. soil (g)	Percentage Retained	Cumulative % Retained	Percentage Passing
No 10	2	551.1	551.1	0.00	0.000	0.00	100.0
No 16	1.18	538.9	547.4	8.50	14.167	14.17	85.8
No 30	0.6	516.7	522.7	5.99	9.983	24.15	75.9
No 50	0.3	488.2	492.5	4.28	7.133	31.28	68.7
No 100	0.15	481.9	485.1	3.21	5.350	36.63	63.4
No 200	0.075	459.2	465.1	5.91	9.850	46.48	53.5
pan		425.5	425.5	0.00	0.000	0.00	

Hydrometer Analysis

Specific Gravity of soil 2.82 Test Temperature, deg.c 22.0

Elapsed Time (min)	Actual Hydrometer Reading	Temprature deg.c	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Percentage Finer Combined
2	14.0000	22.0	8.0000	15.00	0.01283	0.0351	12.86
5	13.5000	22.0	7.5000	15.10	0.01283	0.0223	12.06
15	12.0000	22.0	6.0000	15.30	0.01283	0.0130	9.65
30	11.0000	22.0	5.0000	15.50	0.01283	0.0092	8.04
60	10.0000	22.0	4.0000	15.60	0.01283	0.0065	6.43
250	8.0000	22.0	2.0000	16.00	0.01283	0.0032	3.22
1440	6.0000	22.0	0.0000	16.30	0.01283	0.0014	0.00



Tested by :____________________ Checked by :____________________
Processed by: ____________________ Approved by :____________________

5. Laboratory results of construction materials

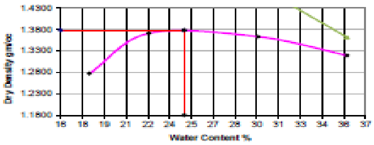
Compaction clay materials

Soil & Material Testing Section

Tel. 251 - 116 - 18 55 16/61 45 01 P.O.Box 2561
Fax 251 - 116 - 61 53 71/61 08 98 Addis Ababa
e-mail w.w.d.s.e@ethionet.et Ethiopia

Project : Upper Guder Multi Purpose Project
Client : MWIE Sample type : Disturbed
Location : E:360994[illegible]:9800408 Test type : Standard Proctor
BH No : CBTP-2BDS1B Date : 25/03/16
Depth(m) : 0.0-3.0 Lab No: 596/08

Lab.test No.	1	2	3	4
Water in cc	200	300	400	500
Wt.of mould + Wet sample(g)	3058	3217	3303	3327
Wt.of mould (g)	1635.4	1635.4	1635.4	1635.4
Wt.of wet soil(g)	1422.6	1581.6	1667.6	1691.6
Volume of mould cm3	944	945	944	945
Wet Density gm /cm3	1.51	1.67	1.77	1.79
Moisture content				
Tin No.	11	M	34	28
Wet soil + tin (g)	65.16	82.23	129.22	159.57
Dry soil + tin (g)	57.80	70.36	103.50	121.98
Wt of tin (g)	16.88	16.59	16.6	16.73
Wt of Water (g)	7.36	11.87	25.72	37.59
Wt of Dry soil (g)	40.92	53.77	86.9	105.25
Moisture content %	17.99	22.08	29.60	35.71
Dry Density gm /cm3	1.2773	1.3710	1.3631	1.3190
Zero Air Voids 100%	1.79	1.67	1.49	1.36



24.50 1.180
24.50 1.378
1.378 16.00
1.378 24.50

MDD 1.378 gm/cc
OMC 24.60 %

TESTED BY:- CHECKED BY :-
PROCESSED BY :- APPROVED BY :-

Water Works Design and Supervision Enterprise Laboratory ServiceSub Process
Soil & Material Testing Section

Water Works Design and Supervision Enterprise Laboratory ServiceSub Process
Soil & Material Testing Section

Tel. 251 - 116 - 18 55 16/61 45 01 P.O.Box 2561
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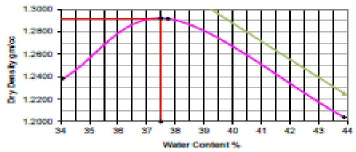
e-mail w.w.d.s.e@ethionet.et Ethiopia

Project : Upper Guder Multi Purpose Project
Client : MWIE Sample type : Disturbed
Location : E:360857[illegible]:9800318 Test type : Standard Proctor
BH No : CBTP-1BDS1B Date : 25/03/16
Depth(m) : 0.90-2.40 Lab No: 593/08

Lab.test No.	1	2	3
Water in cc	400	500	600
Wt.of mould + Wet sample(g)	3202.7	3314.8	3270.5
Wt.of mould (g)	1635.4	1635.4	1635.4
Wt.of wet soil(g)	1567.3	1679.4	1635.1
Volume of mould cm3	944	944	944
Wet Density gm /cm3	1.66	1.78	1.73
Moisture content			
Tin No.	N	M	11
Wet soil + tin (g)	116.54	140.42	118.54
Dry soil + tin (g)	91.19	106.49	87.53
Wt of tin (g)	16.8	16.59	16.88
Wt of Water (g)	25.35	33.93	31.01
Wt of Dry soil (g)	74.39	89.9	70.65
Moisture content %	34.08	37.74	43.89
Dry Density gm /cm3	1.2383	1.2916	1.2037
Zero Air Voids 100%	1.39	1.32	1.23



37.50 1.200
37.50 1.292
1.292 29.50
1.292 37.50

MDD 1.292 gm/cc
OMC 37.60 %

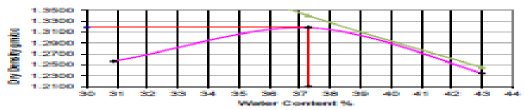
TESTED BY:- CHECKED BY :-
PROCESSED BY :- APPROVED BY :-

Water Works Design and Supervision Enterprise Laboratory ServiceSub Process
Soil & Material Testing Section

Tel. 251 - 116 - 18 55 16/61 45 01 P.O.Box 2561
Fax 251 - 116 - 61 53 71/61 08 98 Addis Ababa
e-mail w.w.d.s.e@ethionet.et Ethiopia

Project : Upper Guder Multi Purpose Project
Client : MWIE Sample type : Disturbed
Location : E:360249[illegible]:9803828 Test type : Standard Proctor
BH No : CBTP-5BDS1D Date : 25/03/16
Depth(m) : 0.40-3.0 Lab No: 594/08

Lab.test No.	1	2	3
Water in cc	330	430	530
Wt.of mould + Wet sample(g)	3188.0	3343.9	3301.7
Wt.of mould (g)	1635.4	1635.4	1635.4
Wt.of wet soil(g)	1552.6	1708.5	1666.3
Volume of mould cm3	944	944	944
Wet Density gm /cm3	1.64	1.81	1.77
Moisture content			
Tin No.	FD	MSN	15
Wet soil + tin (g)	73.55	101.50	115.07
Dry soil + tin (g)	60.10	78.45	85.61
Wt of tin (g)	16.56	16.64	17.1
Wt of Water (g)	13.45	23.05	29.46
Wt of Dry soil (g)	43.54	61.81	68.51
Moisture content %	30.89	37.29	43.00
Dry Density gm /cm3	1.2565	1.3183	1.2344
Zero Air Voids 100%	1.47	1.34	1.25



37.30 1.210
37.30 1.319
1.319 30.00
1.319 37.30

MDD 1.319 gm/cc
OMC 37.30 %

TESTED BY:- CHECKED BY :-
PROCESSED BY :- APPROVED BY :-

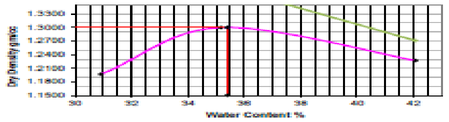
Water Works Design and Supervision Enterprise Laboratory ServiceSub Process

Tel. 251 - 116 - 18 55 16/61 45 01 P.O.Box 2561
Fax. 251 - 116 - 61 53 71/61 08 98 Addis Ababa
e-mail w.w.d.s.e@ethionet.et Ethiopia

Project : Upper Guder Multi Purpose Project
Client : MWIE
Location : E:360651N:980145E
BH No : CBTP-3BDS1
Depth(m) : 0.40-2.0

Sample type : Disturbed
Test type : Standard Proctor
Date : 25/03/16
Lab No: 598/08

Lab.test No.	1	2	3
Water in cc	330	430	530
Wt.of mould + Wet sample(g)	3114.3	3293.7	3280.5
Wt.of mould (g)	1635.4	1635.4	1635.4
Wt.of wet soil(g)	1478.9	1658.3	1645.1
Volume of mould cm3	944	944	944
Wet Density gm /cm3	1.57	1.76	1.74
Moisture content			
Tin No.	23	57	24
Wet soil + tin (g)	72.62	100.16	117.08
Dry soil + tin (g)	59.36	78.43	87.37
Wt of tin (g)	16.47	16.63	16.75
Wt of Water (g)	13.26	21.73	29.71
Wt of Dry soil (g)	42.89	61.8	70.62
Moisture content %	30.92	35.16	42.07
Dry Density gm /cm3	1.1967	1.2997	1.2266
Zero Air Voids 100%	1.48	1.39	1.27



35.40	1.150
35.40	1.300
1.300	26.00
1.300	35.40

MDD 1.300 gm/cc
OMC 35.40 %

TESTED BY:- ____________ CHECKED BY :- ____________
PROCESSED BY :- ____________ APPROVED BY :- ____________

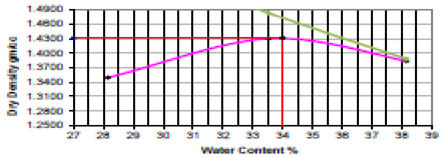
Water Works Design and Supervision Enterprise Laboratory Service Sub Process
Soil & Material Testing Section

Tel. 251 - 116 - 18 55 16/61 45 01 P.O.Box 2561
Fax. 251 - 116 - 61 53 71/61 08 98 Addis Ababa
e-mail w.w.d.s.e@ethionet.et Ethiopia

Project : Upper Guder Multi Purpose Project
Client : MWIE
Location : E:360537N:978765E
BH No : FCB-5-TP-1BDS1E
Depth(m) : 0.0-3.0

Sample type : Disturbed
Test type : Standard Proctor
Date : 25/03/16
Lab No: 599/08

Lab.test No.	1	2	3	4
Water in cc	300	400	500	600
Wt.of mould + Wet sample(g)	3188	3365.2	3379.5	3332.1
Wt.of mould (g)	1635.4	1635.4	1635.4	1635.4
Wt.of wet soil(g)	1552.6	1729.8	1744.1	1696.7
Volume of mould cm3	945	944	944	944
Wet Density gm /cm3	1.64	1.83	1.85	1.80
Moisture content				
Tin No.	57	54	7	97
Wet soil + tin (g)	71.02	140.50	145.92	142.00
Dry soil + tin (g)	61.31	113.29	113.42	107.38
Wt of tin (g)	16.63	16.63	16.52	16.66
Wt of Water (g)	9.71	27.21	32.5	34.62
Wt of Dry soil (g)	44.68	96.66	96.9	90.72
Moisture content %	21.73	28.15	33.54	38.16
Dry Density gm /cm3	1.3497	1.4299	1.3835	1.3009
Zero Air Voids 100%	1.80	1.61	1.48	1.39



34.00	1.170
34.00	1.431
1.431	27.00
1.431	34.00

MDD 1.431 gm/cc
OMC 34.00 %

TESTED BY:- ____________ CHECKED BY :- ____________
PROCESSED BY :- ____________ APPROVED BY :- ____________

Direct shear strength clay materials

Water Works Design and Supervision Enterprise Laboratory ServiceSub Process

Soil & Material Testing Section

Tel. 251 - 116 - 18 55 16/61 45 01 P.O.Box 2561
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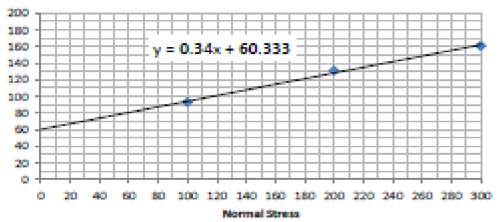
e-mail w.w.d.s.e@ethionet.et Ethiopia

Prject : Upper Guder Multi Purpose Project
Client : MWIE
Location : E:360249N:980382 Test Type : Direct Shear
T.Pit. No. CBTP-5BDS1 Date :25/4 /2016
Depth(m) : 0.40-3.0 Lab-No :594 /08

Normal Stress	Shear Stress
0	0
100	93
200	131
300	161

Shear Stress



C= 60.33 kPa
& =18.77^{0}

TASTED BY:_______________ CHECKED BY:_______________
PROCESSED BY:_______________ APPROVED BY:_______________

Water Works Design and Supervision Enterprise Laboratory ServiceSub Process

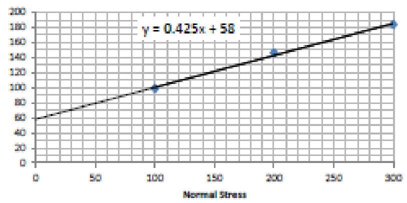
Soil & Material Testing Section

Tel. 251 - 116 - 18 55 16/61 45 01 P.O.Box 2561
Fax. 251 - 116 - 61 53 71/61 08 98 Addis Ababa
e-mail w.w.d.s.e@ethionet.et Ethiopia

Prject : Upper Guder Multi Purpose Project
Client : MWIE
Location : E:360857N:980031 Test Type : Direct Shear
T.Pit. No. CBTP-1BDS1 Date : 25/4/2016
Depth(m) : 0.90-2.40 Lab-No : -593 /08

Normal Stress	Shear Stress
0	0
100	99
200	146
300	184

Shear Stress



C= 58kPa
F =23.30^{0}

TASTED BY:_______________ CHECKED BY:_______________
PROCESSED BY:_______________ APPROVED BY:_______________

Water Works Design and Supervision Enterprise Laboratory ServiceSub Process

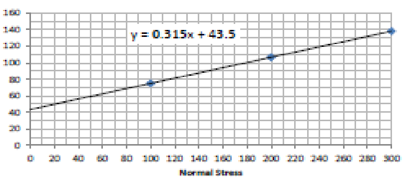
Soil & Material Testing Section

Tel. 251 - 116 - 18 55 16/61 45 01 P.O.Box 2561
Fax. 251 - 116 - 61 53 71/61 08 98 Addis Ababa
e-mail w.w.d.s.e@ethionet.et Ethiopia

Prject : Upper Guder Multi Purpose Project
Client : MWIE
Location : E:360994N:980040 Test Type : Direct Shear
T.Pit. No. CBTP-2BDS1 Date : 26/4/2016
Depth(m) : 0.0-3.0 Lab-No : -596/08

Normal Stress	Shear Stress
0	0
100	75
200	107
300	138

Shear Stress



C=43.50 kPa
& =17.48^{0}

TASTED BY:_______________ CHECKED BY:_______________
PROCESSED BY:_______________ APPROVED BY:_______________

Water Works Design and Supervision Enterprise Laboratory ServiceSub Process

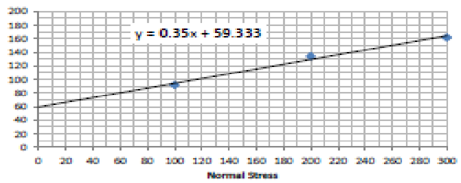
Soil & Material Testing Section

Tel. 251 - 116 - 18 55 16/61 45 01 P.O.Box 2561
Fax. 251 - 116 - 61 53 71/61 08 98 Addis Ababa
e-mail w.w.d.s.e@ethionet.et Ethiopia

Prject : Upper Guder Multi Purpose Project
Client : MWIE
Location : E:360651N:980145 Test Type : Direct Shear
T.Pit. No. CBTP-3BDS1 Date :25/4/2016
Depth(m) : 0.40-2.0 Lab-No : -598/08

Normal Stress	Shear Stress
0	0
100	92
200	134
300	162

Shear Stress



C= 59.33 kPa
& =19.29^{0}

TASTED BY:_______________ CHECKED BY:_______________
PROCESSED BY:_______________ APPROVED BY:_______________

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Water Works Design and Supervision Enterprise Laboratory ServiceSub Process

Soil & Material Testing Section

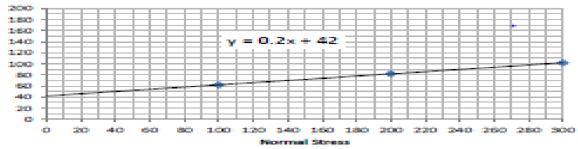
Tel. 251 - 116 - 18 55 16/81 45 01
Fax. 251 - 116 - 61 53 71/61 08 98
e-mail w.w.d.s.e@ethionet.et

P.O.Box 2561
Addis Ababa
Ethiopia

Project : Upper Guder Multi Purpose Project
Client : MWIE
Location : E:360537N:978765
T.Pit. No. FCB-5-TP-1BDS1
Depth(m) : 0.0-3.0

Test Type : Direct Shear
Date :25/4/2016
Lab-No : -599/08

Normal Stress	Shear Stress
0	0
100	62
200	82
300	102



C= 42.00 kPa
& =11.31°

TASTED BY:______________________ CHECKED BY:______________________
PROCESSED BY:______________________ APPROVED BY:______________________

Combined sieve and hydrometer analysis of clay materials

የውሃ ሥራዎች ዲዛይንና ቁጥጥር ድርጅት የላቦራቶሪ አገልግሎት ንዑስ የሥራ ሂደት

[illegible]

Water Works Design and Supervision Enterprise Laboratory Service Sub Process

Soil & Material Testing Section

Project : Upper Guder Multi Purpose Project
Client : MWIE
Location : E:360399N:979376
Test Pit : CBTP-4BDS1
Depth (m): 0.0-3.0

Sample Type : Disturbed
Test Type : Hydrometer
Date :22/3/2016
Lab-No : - 592/08

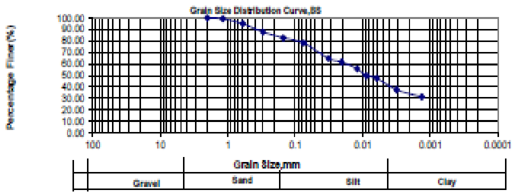
Total mass of sample, g 50

Sieve No	Sieve Opening(mm)	Mass of Sieve(g)	Mass of sieve + Ret. soil(g)	Mass of Ret. soil (g)	Percentage Retained	Cumulative % Retained	Percentage Passing
№ 10	2	551.1	551.1	0.00	0.000	0.00	100.0
№ 16	1.18	538.9	539.3	0.35	0.700	0.70	99.3
№ 30	0.6	516.7	518.9	2.18	4.360	5.06	94.9
№ 50	0.3	488.2	491.8	3.56	7.120	12.18	87.8
№ 100	0.15	481.9	484.4	2.52	5.040	17.22	82.8
№ 200	0.075	459.2	461.5	2.25	4.500	21.72	78.3
pan		425.5	425.5	0.00	0.000	0.00	

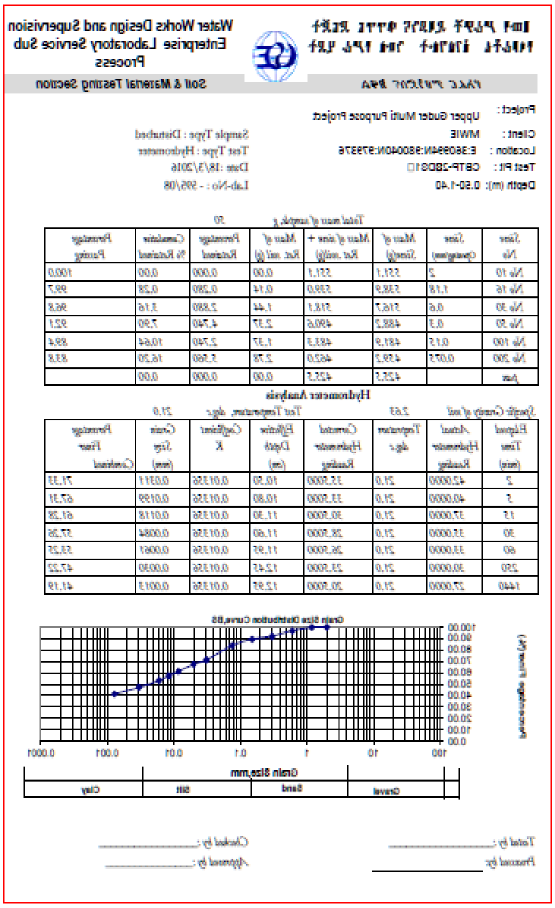
Hydrometer Analysis

Specific Gravity of soil 2.60 *Test Temperature, deg.c* 21.0

Elapsed Time (min)	Actual Hydrometer Reading	Temperature deg.c	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Percentage Finer Combined
2	38.5000	21.0	32.0000	11.10	0.01369	0.0322	64.75
5	37.0000	21.0	30.5000	11.30	0.01369	0.0206	61.72
15	34.0000	21.0	27.5000	11.80	0.01369	0.0121	55.65
30	31.0000	21.0	24.5000	12.30	0.01369	0.0088	49.58
60	30.0000	21.0	23.5000	12.45	0.01369	0.0062	47.55
250	25.0000	21.0	18.5000	13.25	0.01369	0.0032	37.44
1440	22.0000	21.0	15.5000	13.75	0.01369	0.0013	31.37



*Tested by :*______________ *Checked by :*______________
*Processed by:*______________ *Approved by :*______________



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Water Works Design and Super Enterprise Laboratory Service Process

Soil & Material Testing Sectio

Project : Upper Guder Multi Purpose Project
Client : MWIE
Location : E:360651N:980145
Test Pit : CBTP-3 SDSI
Depth (m): 0.40-2.0

Sample Type : Disturbed
Test Type : Hydrometer
Date :18/3/2016
Lab-No : - 597/08

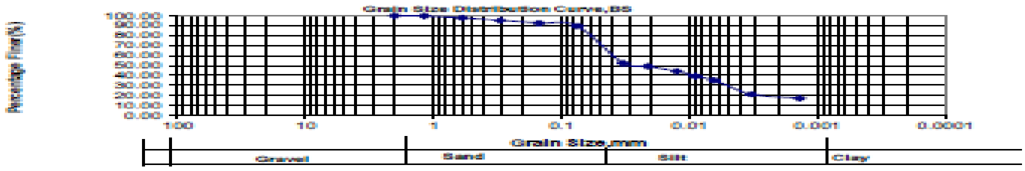
Total mass of sample, g 50

Sieve No	Sieve Opening(mm)	Mass of Sieve(g)	Mass of sieve + Ret. soil(g)	Mass of Ret. soil (g)	Percentage Retained	Cumulative % Retained	Percentage Passing
No 10	2	551.1	551.1	0.00	0.000	0.00	100.0
No 16	1.18	538.9	539.1	0.15	0.300	0.30	99.7
No 30	0.6	516.7	517.6	0.91	1.820	2.12	97.9
No 50	0.3	488.2	489.6	1.35	2.700	4.82	95.2
No 100	0.15	481.9	483.3	1.37	2.740	7.56	92.4
No 200	0.075	459.2	460.6	1.42	2.840	10.40	89.6
pan		425.5	425.5	0.00	0.000	0.00	

Hydrometer Analysis

Specific Gravity of soil 2.63 Test Temperature, deg.c 21.0

Elapsed Time (min)	Actual Hydrometer Reading	Temperature deg.c	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Percentage Finer Combined
2	32.5000	21.0	26.0000	12.00	0.01356	0.0332	52.24
5	31.0000	21.0	24.5000	12.30	0.01356	0.0213	49.23
15	28.5000	21.0	22.0000	12.70	0.01356	0.0125	44.20
30	26.0000	21.0	19.5000	13.10	0.01356	0.0090	39.18
60	24.0000	21.0	17.5000	13.40	0.01356	0.0064	35.16
250	17.0000	21.0	10.5000	14.60	0.01356	0.0033	21.10
1440	15.0000	21.0	8.5000	14.90	0.01356	0.0014	17.08



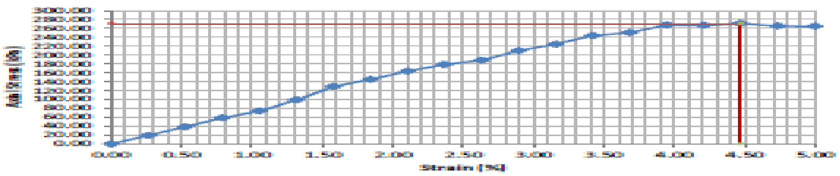
Tested by :______________________
Processed by: ______________________

Checked by :______________________
Approved by :______________________

Unconfined compressive strength Lab. test results

Unconfined Compression Test Data

Deformation Dial Reading	Load Dial Reading	Sample Deformation, ΔL(mm)	Strain(ε)	% Strain	Corrected Area A'	Load (kN)	Stress (kPa)
0	0	0.00	0.00	0.00	1133.540	0.000	0.00
20	2	0.20	0.003	0.263	1136.523	0.022	19.27
40	3	0.40	0.005	0.526	1139.506	0.044	38.44
60	5	0.60	0.008	0.789	1142.489	0.067	58.78
80	6	0.80	0.011	1.053	1145.472	0.085	73.93
100	8	1.00	0.013	1.316	1148.455	0.114	99.16
120	10	1.20	0.016	1.579	1151.438	0.149	129.33
140	12	1.40	0.018	1.842	1154.421	0.168	145.44
160	13	1.60	0.021	2.105	1157.404	0.190	163.99
180	14	1.80	0.024	2.368	1160.387	0.207	178.66
200	15	2.00	0.026	2.632	1163.370	0.219	188.25
220	17	2.20	0.029	2.895	1166.353	0.245	210.30
240	18	2.40	0.032	3.158	1169.336	0.263	224.74
260	20	2.60	0.034	3.421	1172.319	0.286	244.10
280	20	2.80	0.037	3.684	1175.302	0.295	250.93
300	22	3.00	0.039	3.947	1178.285	0.315	267.64
320	22	3.20	0.042	4.211	1181.268	0.315	266.97
340	22	3.40	0.045	4.474	1184.251	0.321	271.23
360	22	3.60	0.047	4.737	1187.234	0.315	265.63
380	22	3.80	0.050	5.000	1190.217	0.315	264.96



Tested by: -______________________
Cecked by: -______________________

Processed by:-______________________
Approved by: -______________________

UNCONFINED COMPRESSION TEST DATA SHEET

Sample Data:

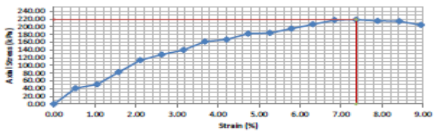
Project: -	Upper Guder Multi Purpose Project
BH No: -	CBTP-5BDS1
Depth(m)	0.40-3.0
Dia(mm)=	38.00
Length(mm)=	76.00
Rate (mm/min)	1.25
Ring Factor (kN/div)=	0.0146
Mass of Soil(gm)	154.55

BH No: -	CBTP-5BDS1	
Depth(m)	0.40-3.0	
Container No	B	
Wt of Wet Sample + Container(gm)	154.55	
Wt of Dry Sample + Container(gm)	112.72	
Wt. Of Water(gm)	41.83	
Wt. Of Container (gm)	16.51	
Wt of Dry Sample (gm)	96.21	
Water Content(%)	43.48	
Average Water Content(%)		

Area, A_0 (mm^2)=	1133.54
Volume, (mm^3)=	86149.04
Bulk Density(gm/cc)=	1.79
Dry Density(gm/cc)=	1.79

Unconfined Compression Test Data

Deformation Dial Reading	Load Dial Reading	Sample Deformation ΔL(mm)	Strain (ε)	% Strain	Corrected Area A'	Load (kN)	Stress (kPa)
0	0	0.00	0.00	0.00	1133.540	0.000	0.00
20	3	0.20	0.005	0.526	1139.506	0.047	41.00
40	4	0.40	0.011	1.053	1145.472	0.058	50.98
60	7	0.60	0.016	1.579	1151.438	0.095	82.42
80	9	0.80	0.021	2.105	1157.404	0.131	113.53
100	10	1.00	0.026	2.632	1163.370	0.149	128.01
120	11	1.20	0.032	3.158	1169.336	0.164	139.84
140	13	1.40	0.037	3.684	1175.302	0.190	161.49
160	14	1.60	0.042	4.211	1181.268	0.197	166.85
180	15	1.80	0.047	4.737	1187.234	0.216	182.00
200	15	2.00	0.053	5.263	1193.200	0.219	183.54
220	16	2.20	0.058	5.789	1199.166	0.234	194.80
240	17	2.40	0.063	6.316	1205.132	0.248	205.95
260	18	2.60	0.068	6.842	1211.098	0.263	216.99
280	18	2.80	0.074	7.368	1217.064	0.266	218.33
300	18	3.00	0.079	7.895	1223.030	0.263	214.88
320	18	3.20	0.084	8.421	1228.996	0.263	213.83
340	17	3.40	0.089	8.947	1234.962	0.253	204.52
360	17	3.60	0.095	9.474	1240.928	0.253	203.54



Tested by: -______________________ Processed by:-______________________

Cecked by: -______________________ Approved by: -______________________

UNCONFINED COMPRESSION TEST DATA SHEET

Sample Data:

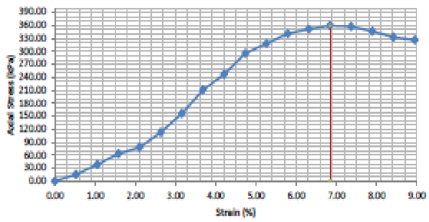
Project: -	Upper Guder Multi Purpose Project
BH No: -	CBTP-2BDS1
Depth(m)	0.0-3.0
Dia(mm)=	38.00
Length(mm)=	76.00
Rate (mm/min)	1.25
Ring Factor (kN/div)=	0.0146
Mass of Soil(gm)	166.84

BH No: -	CBTP-2BDS1	
Depth(m)	0.0-3.0	
Container No	F12	
Wt of Wet Sample + Container(gm)	166.84	
Wt of Dry Sample + Container(gm)	130.06	
Wt. Of Water(gm)	36.78	
Wt. Of Container (gm)	16.5	
Wt of Dry Sample (gm)	113.56	
Water Content(%)	32.39	
Average Water Content(%)		

Area, A_0 (mm^2)=	1133.54
Volume, (mm^3)=	86149.04
Bulk Density(gm/cc)=	1.94
Dry Density(gm/cc)=	1.94

Unconfined Compression Test Data

Deformation Dial Reading	Load Dial Reading	Sample Deformation, ΔL(mm)	Strain (ε)	% Strain	Corrected Area A'	Load (kN)	Stress (kPa)
0	0	0.00	0.00	0.00	1133.540	0.000	0.00
20	1	0.20	0.005	0.526	1139.506	0.018	15.38
40	3	0.40	0.011	1.053	1145.472	0.044	38.24
60	5	0.60	0.016	1.579	1151.438	0.073	63.40
80	6	0.80	0.021	2.105	1157.404	0.091	78.21
100	9	1.00	0.026	2.632	1163.370	0.131	112.95
120	13	1.20	0.032	3.158	1169.336	0.183	156.07
140	17	1.40	0.037	3.684	1175.302	0.248	211.18
160	20	1.60	0.042	4.211	1181.268	0.292	247.19
180	24	1.80	0.047	4.737	1187.234	0.350	295.14
200	26	2.00	0.053	5.263	1193.200	0.380	318.14
220	28	2.20	0.058	5.789	1199.166	0.409	340.90
240	29	2.40	0.063	6.316	1205.132	0.423	351.33
260	30	2.60	0.068	6.842	1211.098	0.435	359.24
280	30	2.80	0.074	7.368	1217.064	0.435	357.48
300	29	3.00	0.079	7.895	1223.030	0.423	346.19
320	28	3.20	0.084	8.421	1228.996	0.409	332.63
340	28	3.40	0.089	8.947	1234.962	0.403	326.29
360	27	3.60	0.095	9.474	1240.928	0.394	317.67



Tested by: -______________________ Processed by:-______________________

Cecked by: -______________________ Approved by: -______________________

UNCONFINED COMPRESSION TEST DATA SHEET

Sample Data:

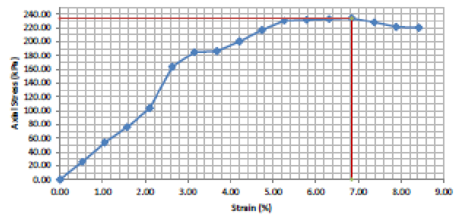
Project: -	Upper Guder Multi Purpose Project
BH No: -	FCB-5-TP-1BDS1
Depth(m)	0.0-3.0
Dia(mm)=	38.00
Length(mm)=	76.00
Rate (mm/min)	1.25
Ring Factor (kN/div)=	0.0146
Mass of Soil(gm)	165.72

BH No: -	FCB-5-TP-1BDS1	
Depth(m)	0.0-3.0	
Container No	214	
Wt of Wet Sample + Container(gm)	165.72	
Wt of Dry Sample + Container(gm)	128.23	
Wt. Of Water(gm)	37.49	
Wt. Of Container (gm)	16.27	
Wt of Dry Sample (gm)	111.96	
Water Content(%)	33.49	
Average Water Content(%)		

Area, A_o (mm^2)=	1133.54
Volume, (mm^3)=	86149.04
Bulk Density(gm/cc)=	1.92
Dry Density(gm/cc)=	1.92

Unconfined Compression Test Data

Deformation Dial Reading	Load Dial Reading	Sample Deformation,Δ L(mm)	Strain (ε)	% Strain	Corrected Area A'	Load (kN)	Stress (kPa)
0	0	0.00	0.00	0.00	1133.540	0.000	0.00
20	2	0.20	0.005	0.526	1139.506	0.029	25.63
40	4	0.40	0.011	1.053	1145.472	0.061	53.53
60	6	0.60	0.016	1.579	1151.438	0.088	76.08
80	8	0.80	0.021	2.105	1157.404	0.120	103.44
100	13	1.00	0.026	2.632	1163.370	0.190	163.15
120	15	1.20	0.032	3.158	1169.336	0.216	184.79
140	15	1.40	0.037	3.684	1175.302	0.219	186.34
160	16	1.60	0.042	4.211	1181.268	0.237	200.23
180	18	1.80	0.047	4.737	1187.234	0.257	216.44
200	19	2.00	0.053	5.263	1193.200	0.274	230.04
220	19	2.20	0.058	5.789	1199.166	0.277	231.33
240	19	2.40	0.063	6.316	1205.132	0.280	232.61
260	19	2.60	0.068	6.842	1211.098	0.283	233.87
280	19	2.80	0.074	7.368	1217.064	0.277	227.93
300	19	3.00	0.079	7.895	1223.030	0.270	220.84
320	19	3.20	0.084	8.421	1228.996	0.270	219.77



Tested by: -____________________ Processed by:-____________________

Cecked by: -____________________ Approved by: -____________________

UNCONFINED COMPRESSION TEST DATA SHEET

Sample Data:

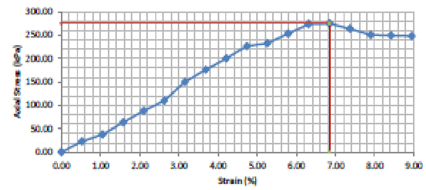
Project: -	Upper Guder Multi Purpose Project
BH No: -	CBTP-3BDS1
Depth(m)	0.40-2.0
Dia(mm)=	38.00
Length(mm)=	76.00
Rate (mm/min)	1.25
Ring Factor (kN/div)=	0.0146
Mass of Soil(gm)	160.5

BH No: -	CBTP-3BDS1	
Depth(m)	0.40-2.0	
Container No	800	
Wt of Wet Sample + Container(gm)	160.95	
Wt of Dry Sample + Container(gm)	118.95	
Wt. Of Water(gm)	42	
Wt. Of Container (gm)	16.32	
Wt of Dry Sample (gm)	102.63	
Water Content(%)	40.92	
Average Water Content(%)		

Area, A_o (mm^2)=	1133.54
Volume, (mm^3)=	86149.04
Bulk Density(gm/cc)=	1.86
Dry Density(gm/cc)=	1.86

Unconfined Compression Test Data

Deformation Dial Reading	Load Dial Reading	Sample Deformation, ΔL(mm)	Strain (ε)	% Strain	Corrected Area A'	Load (kN)	Stress (kPa)
0	0	0.00	0.00	0.00	1133.540	0.000	0.00
20	2	0.20	0.005	0.526	1139.506	0.026	23.06
40	3	0.40	0.011	1.053	1145.472	0.044	38.24
60	5	0.60	0.016	1.579	1151.438	0.073	63.40
80	7	0.80	0.021	2.105	1157.404	0.102	88.30
100	9	1.00	0.026	2.632	1163.370	0.128	110.44
120	12	1.20	0.032	3.158	1169.336	0.175	149.83
140	14	1.40	0.037	3.684	1175.302	0.207	176.40
160	16	1.60	0.042	4.211	1181.268	0.237	200.23
180	18	1.80	0.047	4.737	1187.234	0.269	226.27
200	19	2.00	0.053	5.263	1193.200	0.277	232.48
220	21	2.20	0.058	5.789	1199.166	0.304	253.24
240	23	2.40	0.063	6.316	1205.132	0.330	273.80
260	23	2.60	0.068	6.842	1211.098	0.333	274.86
280	22	2.80	0.074	7.368	1217.064	0.321	263.91
300	21	3.00	0.079	7.895	1223.030	0.307	250.69
320	21	3.20	0.084	8.421	1228.996	0.307	249.47
340	21	3.40	0.089	8.947	1234.962	0.307	248.27



Tested by: -____________________ Processed by:-____________________

Cecked by: -____________________ Approved by: -____________________

Summary of Laboratory tests results of construction materials

Parameter	CBTP-1 BDS1 E:360857 N:980031 0.90-2.40 Lab No: 593/08	CBTP-5 BDS1 E:360249 N:980382 0.40-3.0 Lab No: 594/08	CBTP-2 SDS1 E:360994 N:980040 0.50-1.40 Lab No: 595/08	CBTP-2 BDS1 E:360994 N:980040 0.0-3.0 Lab No: 596/08	CBTP-3 SDS1 E:360651 N:980145 0.40-2.0 Lab No: 597/08	CBTP-3 BDS1 E:360651 N:980145 0.40-2.0 Lab No: 598/08	FCB-5-TP-1 BDS1 E:360537 N:978765 0.0-3.0 Lab No: 599/08	FSQ1 BDS1 E:355223 N:982250 River deposit Lab No: 600/08	FSRQ2 RS2 E:357121 N:981469 From surface Lab No: 601/08	FSRQ3 RS3 Fato river From surface Lab No: 602/08
Specific gravity	-	-	-	-	-	-	-	2.97	3.08	2.56
Grain Size Analysis Clay % Silt % Sand % Gravel %	-	-	44.20 39.60 16.20 -	-	19.09 70.51 10.40 -	-	-	-	-	-
Atterberg Limits Liquid Limit Plastic Limit Plastic Index	-	-	56.35 27.80 28.55	-	53.44 33.16 20.28	-	-	-	-	-
Standard Proctor MDD (gm/cc) OMC (%)	1.292 37.50	1.319 37.30	-	1.378 24.50	-	1.300 35.40	1.431 34.00	-	-	-
Direct Shear C(kPa) Φ (°)	58.00 23.30	60.33 18.77	-	43.50 17.48	-	59.33 19.29	42.00 11.31	-	-	-
Permeability, cm/sec	5.02X10^{-8}	1.07X10^{-7}	-	5.49X10^{-8}	-	3.19X10^{-8}	3.09X10^{-7}	-	-	-
Oedometer Consolidation, Cc	0.240	0.200	-	0.170	-	0.380	0.270	-	-	-
Point load (MPa)	-	-	-	-	-	-	-	8.78	7.65	5.34
NMC, %	-	-	19.53	-	21.17	-	-	-	-	-
Free Swell %	-	-	52.50	-	62.50	-	-	-	-	-
Organic Content (%)	0.55	0.84	-	0.78	-	3.88	0.76	-	-	-
Dispersion by pinhole	ND1	ND1	-	ND1	-	ND1	ND1	-	-	-
Unit Weight gm/cc	2.54	2.49	-	2.35	-	2.37	2.41	-	-	-
Water Absorption	-	-	-	-	-	-	-	0.77	0.61	1.30
UCS (kPa)	271.23	218.33	-	359.24	-	274.86	233.87	-	-	-
Bulk density	-	-	-	-	-	-	-	2.55	2.42	2.83