Assessment of Causes for Partial settlement of Gidabo Dam, Southern Ethiopia.

Atakti Hagos

A Thesis Submitted to

School of Earth Sciences

Presented In Partial Fulfillment of Requirement for the Degree of Masters of Science (in Geology Engineering)

ADDIS ABABA UNIVERSITY

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DECLARATION

I hereby declare that this thesis is my original work that has been carried out under the supervision of Dr. Tarun Tarun Kumar Raghuvanshi, School of Earth science, Addis Ababa University during the year 2017 as part of the Master of Science program in Engineering Geology in accordance with the rule and regulation of the institute. I further declare that this work has not been submitted to any other university of institution for the award of any degree or diploma and all sources of materials used for the thesis have duly acknowledged.

Ataklti Hagos

Signature __________________________________

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

May 2017

ABSTRACT

Assessment of causes for partial settlement of Gidabo Dam, Southern Ethiopia.

Atakltі Hagos

Addis Ababa University, 2017

The present study was carried out at Gidabo Dam, which is proposed on Gidabo River in Oromia Regional State, about 375 km from Addis Ababa, the capital city of Ethiopia. Gidabo Dam has faced settlement at the conduit outlet foundation during the construction time which was measured to be about 0.4 m. The main objectives of this study were to assess the possible causes of partial settlement and to estimate the amount of potential future settlement of the dam. The general methodology followed for the present study was based on thorough literature review, field investigations and data collection, analysis and evaluation of various soil parameters of settlement. For the present study immediate and primary settlement analysis was carried out. Elastic theory for cohesive soils, Janbu’s approach and one dimensional settlement analysis were applied to estimate the settlement amount of the upper part of backfill foundation unit and compressible silty clay layer of the dam foundation. For the granular soil foundation at the bottom immediate settlement was estimated from in-situ standard penetration test (SPT) results.

The present study results showed excessive settlement. The estimated settlement is more than the expected settlement as anticipated in the design of the dam. The differential settlement is also expected at the contact of the backfill material, at outlet conduit and in between the intake tower and the outlet conduit. As investigated in the present study, the primary causes of the settlement are related to unsuitable backfill material comprising alluvium backfill and clay cutoff, compressible silty clay layer (organic) below the excavation and due to in-appropriate excavation method (dewatering process) followed during the construction stage. Besides, granular type of soil in the foundation has also contributed for the settlement of the dam in general, and of conduit section in particular. The study also showed that this settlement also continue in future. Therefore, it is strongly recommended to adopt appropriate measures, as suggested through the present study, so that possible safety and stability of the dam can be ensured during the performance stage.

Key words: Gidabo dam; Settlement analysis; Janbu Settlement analysis; Consolidation

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CHAPTER- 1 INTRODUCTION

1.1. General

Embankment dams have been built since early times. The general philosophy to design these dams is to utilize locally available geological materials. According to Novak et al. (2007) embankment dams are numerically dominant for technical and economic reasons, and account for an estimated 85–90% of all dams built. It is older and simpler in structural concept than the early masonry dams; the embankment dam utilizes locally available untreated materials. In addition to this, embankment dams have proved to be increasingly adaptable to a wide range of site circumstances. In contrast, concrete dams and their masonry predecessors are more demanding in relation to foundation conditions. Historically, they have also proved to be dependent upon relatively advanced and expensive construction skills.

All embankment dams in service, regardless of their age, should be systematically evaluated for their safe performance under all operational conditions. The principal requirement for dam safety evaluation is to protect public safety, property and life. The structural safety of an embankment dam is dependent primarily on the absence of excessive deformations and pore fluid pressure buildup under all conditions of environments and operation, the ability of to pass flood flows, and control of seepage to prevent migration of materials and thus preclude adverse effects on stability. All embankment dams are deformed and settle in their service life. Deformations of embankment dams may result in aesthetically unacceptable surficial appearance. However, excessive deformations indicate distress of the dam, and can result in reduction (loss) of free board and/or internal and/or external cracks. Either of these two consequences of settlements and deformations can lead to dam failure (Chugh, 1990).

In addition to this differential settlement along conduits which penetrate the dam, and in extreme cases, transverse cracks that can lead to failure of the dam. Excessive settlement can cause misalignment of conduits, separation of joints, and possible conduit failure which results in leaking and possible soil piping (DNR, 2001).

There are two basic cause of settlement; settlement due to static loads of the structure and settlement due to secondary influences. The first type of settlement is directly caused by the weight of the structure and the thrust component of the impounded water in the reservoir. For example, the weight of a dam structure may cause compression of an underlying sand deposit or consolidation of an underlying clay layer.

The second basic type of settlement of dam is caused by secondary influence, which may develop after the completion of the structure. This type of settlement is not directly caused by the weight of the structure. For example, the foundation may settle as water infiltrates the ground and causes unstable soils to collapse. The foundation may also settle due to the collapse of limestone cavities or under- ground openings. Natural disasters such as earthquakes or undermining of the foundation from seepage would be other category of causes of settlement (Day, 2001).

In the light of above concept the present research aims to determine the causes for the partial settlement of Gidabo Dam project at its outlet conduit. An attempt is also made in the present research to predict the possible settlement potential of the dam and to evolve likely mitigation measures to minimize the risk of failure of the dam.

1.2. Study area

The present study was carried out at Gidabo Dam, which is proposed on Gidabo River in Oromia Regional State of Ethiopia. The proposed Gidabo dam is an earthfill dam with central clay core filling. The proposed dam height is 23.8 m and crest length is 335 m. A central outlet conduit is provided that will divert water towards right and left canals off take from dam. The reservoir capacity is 250 million m^3. The main purpose of the project is for irrigation and it is expected to cultivate 13000 hectare of farm land. Initially, the project was planned to irrigate 5193 hectare of land by Left bank main canal and 2181 hectare by Right bank main canal with total irrigation of 7374 hectare through its canal distribution network. However, due to additional fill of the reservoir it may irrigate up to 13000 hectare of farm land.

1.3. Location and Accessibility

The Gidabo dam is located in Oromia Regional State, 377 Km from capital city of Ethiopia. The study area is accessible by 360 Km asphalt road from Addis Ababa to Dilla town and the rest 17 km by gravel road. The dam is constructed on Gidabo River which originates in the highland area of Aleta Wondo Escarpment, joining numerous large streams, draining an extensive catchment and flowing into the Lake Abaya as the Eastern tributary. The Gidabo catchment is found in Borena zone in Oromia Region, Sidama Zone, and Gedeo Zone in SNNP Region (Birhanu Debisso, 2009).

The project area lies approximately between UTM co-ordinates 696000N to 726200N and 386000E and 422000E, a short distance east of Lake Abaya and just south of Gidabo river flood plain, at an average elevation of 1190 a.m.s.l (fig.1.1). Gidabo irrigation project is found in Abaya district, Borena zone of Oromia region and Dale district, Sidama zone of SNNPRS near Dilla town to east of Lake Abaya, located in Dibicha Laluncha Kebele of Gelana Abaya district, which is situated in Borena zone. The project area lies in the low land, very close to the Dure and Gola marsh. The command area is situated in the northern part of Lake Abaya. The northern Lake Abaya area, which is located in the southern part of the Main Ethiopian Rift (MER), encloses irrigable lands at different places.

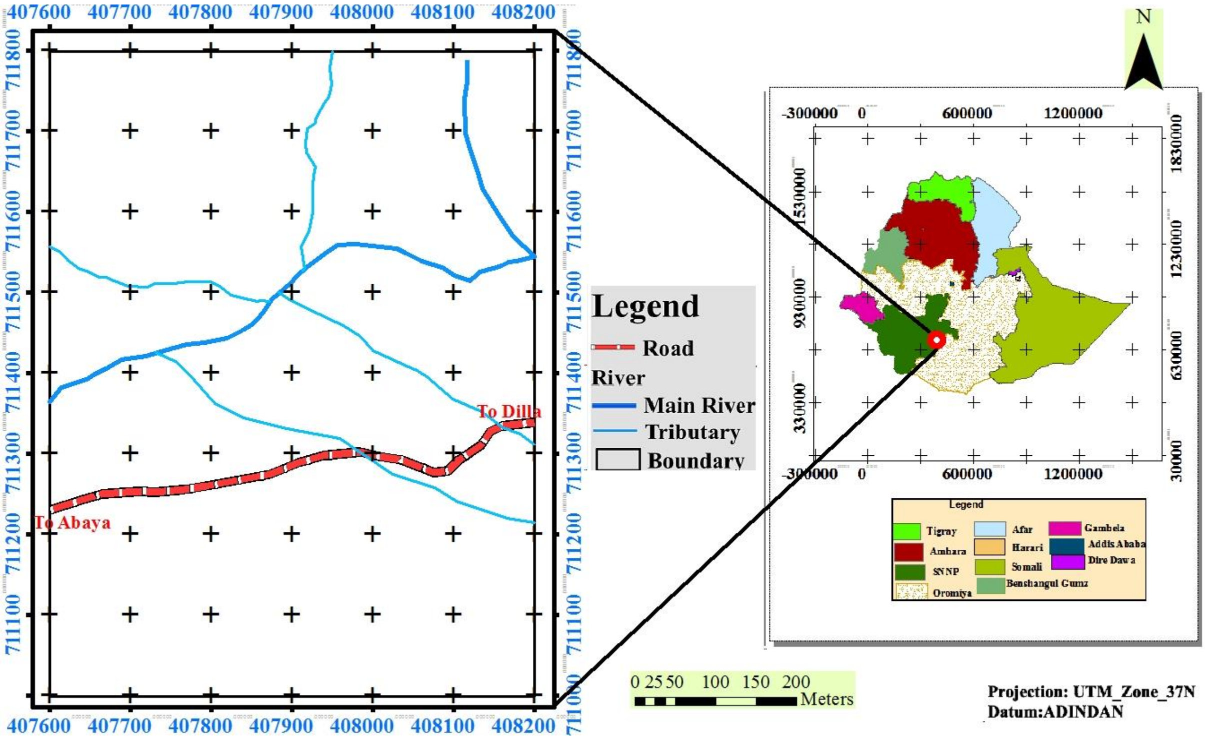

Fig. 1.1 Location map of the study area

1.4. Problem of statement

Failure of embankment dams, except for failures caused by unanticipated catastrophic events such as earthquakes or overtopping, is almost preceded by warning signals such as increased rate of deformation, strain discontinuities, cracking, leakage, and pore pressure buildup (Chugh, 1990). According to WWDSE (2016) Gidabo Dam has faced settlement at the conduit outlet foundation during the construction time. Due to this unexpected settlement it may initiate to differential settlement or losing of free board that may possibly cause for major failures. Therefore, the present research is intended to investigate the problem of settlement and possible causes responsible for this settlement at Gidabo Dam project. An attempt is also made to workout possible mitigation measures to overcome likely dam stability problems.

1.5. Objectives

1.5.1. General objective

The main objectives of this study are to assess the possible causes of partial settlement in the dam and to estimate the amount of settlement in the dam.

1.5.2. Specific objectives

- ✓ To determine the engineering geology properties of the foundation and the embankment material used in the dam
- ✓ To review the design of the dam
- ✓ To estimate the possible settlement potential on the dam foundation
- ✓ To determine the cause for the partial settlement of the dam by comparing the actual settlement happened in the dam with the estimated settlement
- ✓ To workout possible remedial measures for the safety and stability of the dam

1.6. General Methodology

The results of this study are based on the combination of the following fundamental works that are conducted sequentially. In order to achieve the objectives of the present study systematic methodology has been followed which includes;

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

- Collection of secondary data such as; in-situ and laboratory results, construction reports and report after partial settlement happened.

- Field investigation and Collection of soil samples from borrows areas for laboratory testing and analysis to determine various index properties with specific emphasis on consolidation test.

- Analysis of effective stress and pore water pressure conditions from the laboratory result and field data.

- Analysis of settlement in the foundation and within the embankment under all conditions by using different analysis methods and empirical relationships.
- Interpretation of the result for the determination of possible cause for the partial settlement on the dam during construction.

1.7. Importance of the study

The results and the findings of the present study are expected to be utilized by the Project Authorities or by any other individual or organization. The data generated through this study will also be utilized by the later researchers intending to work on the same subject or in the same study area. Since the present research study was intended to assess the causes for the partial settlement of the dam therefore, it may be possibly helpful for the mitigation of the problem through life time of the dam.

The present study will also be a guide line for geotechnical engineers and engineering geologist who are involved in foundation and construction material assessment for embankments. In addition it may also provide a good guideline for embankment dam designers and professionals involved in supervision of embankment construction especially on those areas which generally demonstrates settlement problems.

1.8. Limitation and the Scope of the study

The present research was focused on assessment of causes for settlement therefore, it demanded reliable data. During the field work it was difficult to collect undisturbed samples from the foundation as it is now buried under embankment fill. However, in order to have the representative foundation samples, the samples were collected from the nearby locations.

Besides, secondary data was also utilized to make necessary analysis. The present research was conducted under time, resources and the financial constraints.

1.9. Chapter Scheme

The present research study is compiled into seven chapters and a brief description of each chapter is presented hereunder;

Chapter 1: presents general introduction to the problem, the study area, location and accessibility, statement of the problem, objectives, methodology, importance of the study and limitation and scope of the study.

Chapter 2: this chapter presents literature review on the settlement problems in dams, review on conduit settlement, dam design review and theory on analyzing settlement.

Chapter 3: is on the study area, this chapter is focused on project background and salient feature, geology, hydrogeology and seismicity of the study area.

Chapter 4: presents the general methodology followed in the present study. It provides a description on type of data collected, processing and analysis followed.

Chapter 5: describes about data presentation, processing and analyzing.

Chapter 6: is about result and discussion. It presents analysis results on causes of settlement and the possible mitigation measurements.

Chapter 7: presents conclusion and recommendation

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CHAPTER- 2 LITERATURE REVIEW

2.1 Embankment Dams Settlement problem

The behavior of concrete dams is significantly different from that of embankment dams because of the differences in construction materials. In concrete dams, deformation is assumed to be elastic and any permanent deformation may be caused either by the adaptation of the foundation to the new load, aging of concrete, or foundation rock fatigue. In the case of embankment dams the deformation is usually permanent. Permanent vertical settlement of the fill material continues at a decreasing rate for decades after construction, while permanent horizontal deformation of the embankment is caused by the reservoir water pressure. The deformation values for concrete can be in millimeters or centimeters, however for embankment dams it can be in centimeters or decimeters (Saverio, 1993).

Earth embankments are massive structures that inherently have movements and seepage. Consolidation of the embankment and the foundation occurs most rapidly during construction and at a lesser rate for an extended period of time thereafter. The initial filling and its accompanying saturation may temporarily accelerate the consolidation of the upstream section of the embankment, and initial filling will also cause downstream seepage to develop. Consolidation of the embankment and the foundation is accompanied by transverse and longitudinal movements that may result in transverse and longitudinal cracks (Robert, 1988).

The predicted amounts of consolidation, movement and seepage should be determined by analyses during the design stage. These analyses should be reviewed at the end of construction, and modified if the as-constructed engineering characteristics are different from those assumed during design (Robert, 1988).

Load conditions during construction are induced by the progressive placement of compacted layers of material. The construction of an embankment dam is always associated with and followed by a differential settlement of its crest and slopes. Under unfavorable conditions they can be associated with the formation of open cracks across the impervious section of the dam. After the dam has been completed, the crest continues to settle at a decreasing rate. If the dam rests on sediments, the settlements of the crest and slopes is increased by the compression of the foundation materials produced by the weight of the dam and of the impounded water at a later stage (Terzaghi et al., 1993).

Foundations under conduits should have relatively uniform compressibility characteristics to prevent differential settlement and movement of conduit joints. Special precautions should be taken for joints where the conduit connects to a structure, such as an intake structure. This location may be in an area susceptible to differential settlement due to the differing weights of the two structures and the foundation beneath them.

An engineered fill to limit settlement may be needed under the intake structure, when the structure and conduit cannot be located on bedrock or a firm foundation. If the intake structure is constructed on a pile foundation, special precautions are also required for the first few joints of the conduit because high stresses can develop as a result of bending stresses caused by differential settlement. Extending the conduit and locating the intake structure beyond the limits affected by the embankment dam can reduce these stresses (FEMA, 2015).

2.2. Settlement Analysis

When a distributed load from a structure is applied to a soft soil stratum, the following three components of settlement are commonly distinguished (Das, 2008):

1. **Immediate settlement** (also called initial or undrained settlement), which takes place immediately upon load application and, if the soil is saturated, deformation is at constant volume caused by the shear strains beneath the loaded area. Little drainage takes place when the clay has a low permeability. Under the Centre-line of the load, the vertical compression is accompanied by lateral expansion (Arora, 2004).

2. **Consolidation settlement**, the increase in vertical pressure due to the weight of the structure constructed on top of saturated soft clays and organic soil will initially be carried by the pore water in the soil. This increase in pore water pressure is known as an excess pore water pressure (u). The excess pore water pressure will decrease with time as water slowly flows out of the cohesive soil. This flow of water from cohesive soil (which has a low permeability) as the excess pore water pressures slowly dissipate is known as **primary consolidation,** or simply **consolidation.** This is a time-dependent process and produces mainly volume change, but shear deformations are also involved, leading to further settlement (Arora, 2004; Das, 2008).

3. **Secondary compression settlement** (often also termed drained creep) the main part of which takes place after essentially complete dissipation of excess pore water pressures, i.e. at

practically constant effective stresses. In practical cases, it is often assumed that secondary compression does not start until after primary consolidation is completed (Arora, 2004).

Immediate Settlement in Cohesive Soils

According to Venkatramaiah (2006) if saturated clay is loaded rapidly, excess hydrostatic pore pressures are induced; the soil gets deformed with virtually no volume change and due to low permeability of the clay little water is squeezed out of the voids. The vertical deformation due to the change in shape is the immediate settlement.

The immediate settlement of a flexible foundation, According to Terzaghi (1943), is given by:

$$S_i = qB\left(\frac{1-v^2}{E_s}\right)I_t \quad \text{.....eq. 2.1}$$

Where;

S_i=immediate settlement at a corner of a rectangular flexible foundation of size $L \times B$,
B = Width of the foundation,
q = Uniform pressure on the foundation,
E_s= Modulus of elasticity of the soil beneath the foundation,
ν = Poisson’s ratio of the soil, and
I_t= Influence Value, which is dependent on L/B (Table 2.1),
L= length of the foundation

Table 2.1 The value of I_t after Terzaghi, 1943

L/B	**1**	**2**	**3**	**4**	**5**
Influence value I_t	0.56	0.76	0.88	0.96	1

An earth embankment may be taken as flexible and the above formula (eq.2.1) may be used to determine the immediate settlement of the soil below such a construction (Venkatramaiah, 2006). But for the conduit outlet foundation the above formula is not convenient since the foundation is rigid.

The following formula is appropriate:

$$S_{c(rigid)} \approx 0.93 S_{c(flexible,\ center)} \quad \text{.....eq. 2.2}$$

Elastic settlement from SPT value

Terzaghi and Peck (1948, 1967) proposed a correlation for the allowable bearing capacity, standard penetration number (N_{60}), and the width of the foundation (B) by the following relation.

$$S_e(mm) = C_w C_D \frac{3q}{N_{60}} \left(\frac{B}{B+0.3}\right)^2 \quad \text{.....eq. 2.3}$$

Where q=bearing pressure in kN/m^2, B = width of foundation (m), CW = ground water table correction,

C_D=correction for depth embedment= $1 - (\frac{D_f}{4B})$ and D_f = depth embedment.

The magnitude of Cw is equal to 1.0 if the depth of water table is greater than or equal to 2B below the foundation, and it is equal to 2.0 if the depth of water table is less than or equal to B below the foundation. The N_{60}value that is to be used in equation should be the average value of N up to a depth of about 3B to 4B measured from the bottom of the foundation.

Janbu approach

The Janbu approach was proposed by Professor Nilmar Janbu in the early 1960s. The main concept of this approach combines the basic principles of linear and non-linear stress-strain behavior. For linear stress-strain behavior Hook's low is the most recognized approach however Stress-strain behavior is non-linear for most soils. The non-linearity cannot be disregarded when analyzing compressible soils, such as silts and clays, that is, the linear elastic modulus approach is not appropriate for these soils. The method applies to all soils, clays as well as sand. By the Janbu method, the relation between stress and strain is simply a function of two non-dimensional parameters that are unique for any soil: a stress exponent, **J**, and a modulus number, **m** (Fellenuis, 2015).

The Janbu expressions for strain are derived into four categories according to the nature of the soil particle. They are expression for cohesionless, dens coarse grained soil, sandy or silty soil, and cohesive soils. In the present paper cohesive soils and sandy or silty soils expression were used.

For cohesive soils J=0 and normally consolidated clay;

$$\varepsilon = \frac{1}{m} \ln \frac{\sigma_1'}{\sigma_o'} \text{.....eq. 2.4}$$

For sandy or silty soil J=0.5

$\varepsilon = \frac{1}{5m}\left(\sqrt{\sigma_1'} - \sqrt{\sigma_o'}\right)$…..eq. 2.5

Where; ε= strain induced by increase of effective stress in kPa,
σ_o' = original effective stress
σ_1'= final effective stress and m = modulus number.

Modulus number is determined from empirical relationships or from laboratory and field tests.

For sand and silty soil in kPa:

$m = \frac{E}{10\sqrt{\sigma'}}$…..eq. 2.6

Where; E= Elastic Modulus and σ'= average change of effective stress (=$\sigma_1' - \sigma_o'$)

According to Schmertmann, 1970 as stated in Das (2008) the modulus E of elasticity of granular soils has been correlated to the field standard penetration number *N*:

$E(kN/m^2) = 766N$…..eq. 2.7

For cohesive soils by using conventional method or from odometer test:

$m = 2.3\frac{1+e_o}{C_c}$…..eq. 2.8

Where;e_o= initial void ratio and

C_c = compression index.

Finally, the deformation of a soil layer, *s*, is the strain, ε, times the thickness, *h*, of the layer. The settlement, *S*, of the foundation is the sum of the deformations of the soil layers below the foundation.

$S = \sum s = \sum \varepsilon h$…..eq. 2.9

One dimensional consolidation primary settlement

The phenomenon of consolidation occurs in clays because the initial excess pore water pressures cannot be dissipated immediately owing to the low permeability. The theory of one dimensional consolidation, advanced by Terzaghi (1925), can be applied to determine the total compression or settlement of a clay layer as well as the time-rate of dissipation of excess pore pressures and hence the time-rate of settlement. The settlement computed by this procedure is known as that due to primary compression since the process of consolidation as being the dissipation of excess pore pressures alone is considered (Venkatramaiah, 2006). Normally consolidated soils are usually found as recent alluvial deposits, and are mainly composed of silt and clay sized particles. It is extremely rare to find normally consolidated soils inland, away from the rivers or lakes in which they were deposited. Soils from the study area are recently river deposited. Therefore, the present investigation was done by considering soils to be normally consolidated soils.

For normally consolidated clay soils the following equation can be used;

$$S_c = H_o \frac{C_c}{1+e_o} \log \frac{\sigma_o' + \Delta\sigma'}{\sigma_o'} \text{.....eq. 2.10}$$

Where; S_c = primary settlement, H_o = initial height of the layer, C_c= compression index, e_o= initial void ratio, σ_o'= average original effective stress and $\Delta\sigma'$=average change of vertical stress.

Time Rate of Consolidation

Time-rate of settlement is dependent, in addition to other factors, upon the drainage conditions of the clay layer. If the clay layer is sandwiched between sand layers, pore water could be drained from the top as well as from the bottom and it is said to be a case of double drainage. If drainage is possible only from either the top or the bottom, it is said to be a case of single drainage. In the former case, the settlement proceeds much more rapidly than in the latter (Venkatramaiah, 2006).

Terzaghi (1925) advanced his theory of one dimensional consolidation based upon the following assumptions, the mathematical implications being given in parentheses:

(i) The soil is homogeneous (k_z is independent of z).

(ii) The soil grains and water are virtually incompressible (γ_w is constant and volume change of soil is only due to change in void ratio).

(iii) The behavior of infinitesimal masses in regard to expulsion of pore water and consequent consolidation is no different from that of larger representative masses (Principles of calculus may be applied).

(iv) The compression is one-dimensional (u varies with z only).

(v) The flow of water in the soil voids is one-dimensional, Darcy's law being valid.

(vi) Certain soil properties such as permeability and modulus of volume change are constant; these actually vary somewhat with pressure. (k and m_v are independent of pressure).

(vii) The pressure versus void ratio relationship is taken to be the idealised one (a_v is constant).

(vii) Hydrodynamic lag alone is considered and plastic lag is ignored, although it is known to exist. (The effect of k alone is considered on the rate of expulsion of pore water).

The theory of one-dimensional consolidation, advanced by Terzaghi, can be applied to determine the total compression or settlement of a clay layer as well as the time-rate of dissipation of excess pore pressures and hence the time-rate of settlement. The settlement computed by this procedure is known as that due to primary compression since the process of consolidation as being the dissipation of excess pore pressures alone is considered (Venkatramaiah, 2006).

The calculations are based upon the equation:

$T = \frac{C_v t}{H^2}$…..eq. 2.14

Where; T=non-dimensional time factor, C_v= coefficient of consolidation and H= thickness of the layer

The consolidation tests in the present studywere done using British Standard the coefficient of consolidation, C_v (in m^2/year), was determine using BS 1377, 1975 relation as following:

$C_v = 0.026\frac{(H_{Ave})^2}{t_{50}} = 0.026\frac{(\frac{H_1+H_2}{2})^2}{t_{50}}$……eq. 2.15

Where; H_1= is the height of the specimen at the start of the loading increment (in mm), H_2=is the height of the specimen at the end of the loading increment (in mm) and t_{50}= the time takes to reach 50% consolidation

Coefficient of consolidation for each sample was calculated for different load increment and an average value of C_v for the desired load range was determined.

2.3 Review on conduit settlement problem

According to WWDSE (2009) the allowance of 1 to 2% of the height of the dam should be provided for settlement in the foundation and the embankment. For Gidabo dam, a total settlement allowance of 3% of the dam height has been provided.

During the construction of Gidabo Dam the conduit facing settlement which was noticed when the contractor tried to put joint sealant on December 26/2015. Since then measurement and visual observation was taken. The result of surveying measurement showed that the settlement is continuing even after further construction was stopped. As a result of this the metal sheet, welded at the start of the conduit is showing cracks. However, starting from day 30 i.e. about 25 days after the construction of embankment was stopped, the settlement seems to be stopped and the minor differences are attributed to errors in surveying measurement. The maximum settlement recorded at a chainage 0+40 after 55 days was 42.1 cm. The Table 2.3 shows the measured settlement along the conduit. The settlement measurements were taken from 30/12/2015 to 2/22/2016 (WWDSE, 2016).

Table 2.2 Measured settlement along the conduit in meter (distance 0+00 refers to start of the conduit)

	Day	**1**	**5**	**10**	**15**	**20**	**25**	**30**	**35**	**35**	**40**	**50**	**55**
S.No	**Chainage**	**Total**	**total**	**total**	**total**	**Total**	**total**	**total**	**Total**	**Total**	**total**	**total**	**total**
1	0+00	0.231	0.295	0.312	0.317	0.333	0.336	0.343	0.350	0.354	-	-	-
2	0+9.16	0.224	0.262	0.301	0.315	0.329	0.331	0.340	0.343	0.349	0.355	0.364	0.367
3	0+19.15	0.279	0.301	0.304	0.352	0.368	0.371	0.377	0.382	0.386	0.391	0.398	0.403
4	0+29.15	0.294	0.313	0.349	0.364	0.379	0.381	0.389	0.393	0.396	0.400	0.408	0.413
5	0+39.19	0.296	0.325	0.364	0.372	0.389	0.387	0.399	0.401	0.403	0.410	0.416	0.421
6	0+49.37	0.252	0.274	0.302	0.319	0.334	0.335	0.340	0.345	0.349	0.351	0.358	0.364
7	0+59.23	0.173	0.190	0.211	0.225	0.237	0.237	0.241	0.243	0.246	0.248	0.254	0.262
8	0+69.14	0.083	0.091	0.102	0.113	0.121	0.120	0.122	0.124	0.126	0.126	0.133	0.142
9	0+79.20	0.004	0.009	0.014	0.021	0.026	0.024	0.024	0.021	0.025	0.024	0.030	0.131
10	0+89.11	0.006	0.005	0.001	0.006	0.008	0.004	0.002	0.004	0.003	0.003	0.009	0.012
11	0+98.71	0.034	0.35	0.029	0.037	0.039	0.039	0.036	0.035	0.037	0.034	0.038	0.041

After the settlement was noticed professional team was assigned to investigate and put possible remedial measures. This team predicted settlement during the construction (current height) and at post construction by using SIGMA/W Finite Element Model (FEM) software. Most of the parameters adopted for this model were from literature. The parameters used were Poission's Ratio and Modulus of Elasticity. During the current stage at the height of 13 m the maximum settlement at start of conduit 0+00 is about 24 cm compared to actual 35cm obtained from surveying. The maximum settlement that this model has estimated was found on chainage +40 is 50 cm compared to 42cm the actual measurement. The maximum settlement at the end of the construction (crest level) at start of the conduit is 42cm and the maximum possible deformation along the length of the conduit is estimated to be 67cm which is at the start of the conduit.

Settlement due to reservoir loading has been also made. The additional settlement due to reservoir loading is insignificant, as it increase only 3cm around conduit starting and vanishes after around 25m along the conduit compared to FEM done for end of construction (WWDSE, 2016).

2.4 Dam Design Review of Gidabo Dam

2.4.1 Original design of Gidabo dam

The original dam design was done in 2008 by Water Works Design and Supervision Enterprise (WWDSE) in association with consulting Engineering Service (India) (WWDSE, 2008).

Gidabo Irrigation project was proposed with construction of about 20m high rock-fill dam with central clay core at Gidabo dam site with spillway, two outlets for Left bank and Right bank main Canals off taking from the dam on river Gidabo. The project is planned to irrigate net area of 5193 hectare of land by Left bank main Canal and 2181 ha. of net area by Right bank main Canal with total irrigation of 7374 ha land through its canal distribution network. Further, the spillway is designed as a chute spillway. Due to topographic constraints, the overflow portion of spillway is made curved so as to get more length. The location of the spillway is at the left bank of the river. The main components of the spillway are approach channel, ogee type overflow spillway, discharge channel with sub critical slope and stilling basin as the terminal structure (WWDSE, 2008).

For river diversion during construction, a conduit (2 x 2m) will be laid on the left side of the main river channel. The length of the conduit will be approximately equal to the bottom width of the dam at the location of the conduit. The opening of the conduit is designed to pass the dry season flow during the construction. The diversion conduit will serve effectively only for dry season construction period and to be plugged after the construction of the dam and appurtenant structures are over. Irrigation outlet structures are closed conduits. There are two outlets, one at left bank and the other on the right. The irrigation and dry season diversion conduits will all be constructed on pile foundations (WWDE, 2008).

The impervious core of Gidabo Dam is proposed to be flanked by a 1V:2.5H upstream slope and 1V:2.0H to 1V:2.5H downstream slope free draining earth fills.

The original dam design project was finalized in June, 2009 that proposed the dam to be earth fill dam with central clay. Two rectangular conduits 2m wide in their bottom and 2.7 m height each and intake towers were proposed and outlet conduits and the intake towers were designed to rest on pile foundation (WWDSE, 2008).

2.4.2 Dam Design Revision

The dam design revision continued until November 2011 and it brought major changes within the drawings and the dam foundation treatment methods. After the finalization of this design in 2009, later there was a main revision in August 2010. In order to increase the irrigation command area by 6,000 ha (from 7,374 ha to 13,374 ha), the storage capacity of the dam was increased from 32.8 Mm^3 to 62.3 Mm^3 by increasing 3.8 m of the dam height. The design review was made by Ethiopian Water Works Design and Supervision Enterprise (WWDES) free-lance geotechnical engineer. In addition to this during this time it was recommended that the foundation of the two irrigation conduits to be a foundation created by excavating the dark brown soft silt clay with fine sand residual soil and backfill it with a new material referred to be compacted alluvium with relative density >70% and also foundation excavation was proposed (WWDSE, 2016).

The second major modification in dam design was made after visiting the site by freelance designer on July 2011 after one year of the construction began. The designer recommended that the weak and lose layer below both conduits shall be removed and replaced by about 2m thick, well compacted, selected gravelly material. In November 2011 new design was made for the conduits and diversion systems.

In this design both right and left conduit outlets were removed and were replaced by a single conduit with a new single intake tower. Intake Tower structure is to be constructed on the thick alluvial deposit with geotechnical method of stabilizing the foundation through piling work to improve the bearing capacity and stability of the foundation against settlement. The design modification made including arrangements for entrance; control dissipation in line with the design requirements. All such components were mentioned in the revised drawing, however nothing has been stated about the foundation condition in the drawing (WWDES, 2016).

2.4.3 Design Material Parameters Adopted for Gidabo Dam

The Gidabo Dam axis is found at around 80 m downstream from the convergence of Gidabo and Ameleke Rivers. For foundation investigation of dam and reservoir sites, core drilling and test pit excavation have been proposed. Eight boreholes were drilled around Gidabo dam site and Quarry site (GIBH-1 to 7 and GIQH-1). A total depth of 279.52 m was drilled. In addition to this one borehole was drilled with depth 39.3 m in order to determine the depth to bed rock for purpose of Intake Tower pile foundation (WWDSE, 2008).

Dam foundation

For the characterization of the subsurface of the dam foundation there were four boreholes GIBH-2, GIBH-4,GITPBH-1 and GIBH-3 which is outside of the dam axis located downstream but it was helpful by interpolating to the dam axis. The alluvial deposit extends up to 40m depth below the N.G.L. at center of the dam foundation. The top 4-5m thick part of the subsurface geological materials on GIBH-2 and 4 is characterized by light brown to reddish brown, soft to firm, clayey silt or silty clay with sand and it extended up to 9 m in GITPBH-1. Annexure VI shows location of the boreholes and cross section.

Then up to a depth ranges of 25 to 37m, layers of loose to moderately dense sand to gravelly sand and stiff, moderately plastic clayey to silty loam. Towards the bottom most part gravelly material becomes dominant. There is inorganic clay between 11- 17m on GIBH-4 (WWDES, 2008).

Below the gravelly soil layer in GIBH-2 and GIBH-4 around the depth ranges of 34 to 45.25m and 39.5 to 45.06m, respectively, there is weakly weathered, moderately strong, crystal containing rhyolitic ignimbrite (ignimbrite). From 38- 39 m depth on GITPBH-1 is dark gray dolorite -gabbro rock (WWDES, 2014).

For the upper silty clay deposit the Standard penetration test (N) result is in the range of 2 to 9 and for the coarse gravelly sand sillty deposit is 11 to refusal. The patch of organic sillty clay deposit which was identified in GIBH-2 has N value of 8. The weathered rock unit has N value of >50 indicating Refusal to be penetrated. From the standard penetration test (N) the foundation strength are the fine alluvium overburden including organic clay has soft to medium consistency while the coarse alluvium and weathered rock is very stiff to hard(WWDES, 2010).

Field permeability of in situ formation was investigated by using falling head, constant head and Lugeon test methods. Most of the values of K were in the range of 10^{-7} to 10^{-6} cm/s except for borehole GIBH-4 at 10-15 m depth at a coarse – fine alluvium contact was 10^{-5} cm/s. The permeability measured in the rock units varies between 8-172 Lugeon units with the lowest Lugeon values 8 was measured in lithic Tuff unit. Generally the permeability value indicates the overburden deposit and the weathered rock units are semi pervious to impervious. The bed rock (tuff, ignimbrite and the basaltic flow) is pervious (WWDES, 2010).

Construction material of the dam

Some dam sites require considerable excavation to reach a competent foundation. In many cases, the excavated material is satisfactory for use in portions of the embankment. Excavations for a spillway or outlet works also may produce usable materials for filters, for an impervious core, or for other zones in the embankment. However, designated borrow areas will be required in most cases for embankment materials (USBR, 1987).

Table 2.3 Some soil properties of the dam foundation (WWDES, 2008)

Sample Identification	Depth (m)	Grain size distribution (%)				Atterberg'sLimits			Free swell (%)	Shrinkage limit (%)	Specific gravity	Bulk density (gms/cc)
		Gravel	Sand	Silt	clay	LL	PL	PI				
GIBH2-2-CBS2	7.10-7.40		4.16	53.34	44.5	72.65	30.4	42.25	65	17.21	2.65	1.78
GIBH2-CBS1	17.45-17.75		27.93	56.82	15.25	54.38	33.97	20.41	37.5	3.21	2.35	1.54
GIBH2-CBS4	20.00-20.3		31.90	45.60	22.50	46.10	31.05	15.05	42.5	6.57	2.46	1.64
GIBH3-CBS2	6.47-7.00		13.01	12.12	74.87	33.30	19.42	13.88	37.5	3.00	2.51	1.73
GIBH3-CBS3	17.85-18.08	9.6	71.60	16.82	1.98	30.12	-	-	10	2.14	2.42	
GIBH3-CBS1	36.53-36.83		3.42	76.58	20.00	65.39	42.10	23.29	57.5	7.64	2.51	1.45
GIBH4-CBS2	21.00-21.30		23.6	50.15	26.25	65.45	34.85	30.60	52.5	12.25	2.64	
GIBH4-CBS3	23.58-24.12		59.42	27.33	13.25	47.5	27.38	20.12	55.0	4.92	2.64	
GIBH5-CBS1	7.12-7.52		53.65	24.10	22.25	72.30	20.66	51.64	120	19.46	2.52	
GIBH5-CBS2	17.02-17.41					72.92	34.67	38.25	20	2.78	2.53	
GIBH5-CBS3	21.48-21.88		44.73	35.17	20.10	72.92	34.67	38.25	122.	16.86	2.50	

During the feasibility study of Gidabo dam and irrigation project, assessment for the availability of suitable construction materials at close proximity to the proposed site were investigated. This investigation continued until during the construction and supervision stage. The availability of machineries has allowed further investigation and only one source which is the reddish brown silty clay material from Hadama area, 6 – 7 km from the dam site towards to Dilla town, has been found satisfying the quality as well as quantity requirement for core material. As a result, detailed Investigation has been made through digging trial pits, taking samples and further assessing the volume of clay core material from Hadama borrow site. For this purpose a total of 8 pits were made, varying in depth from 1.2 to 2.3 m, within the previously delineated borrow area. Moreover, during production stage a number of pits and trenches have been assessed. The core of the embankment was constructed from the soil from this borrow area (WWDE, 2014).

Alluvium material has been used for dam general foundation, compacted back fill material after excavation to the required design level. For this purpose, the first materials excavated out from the dam and spillway foundation has been used and later a source area from d/s to spillway outside the dam area has been excavated out and used. Furthermore, additional source has been required and material in excess of volume has been handed over to the contractor from the upstream, reservoir, area at a maximum of 5 kms from the dam axis flanking Gidabo River to be used as foundation backfill material for the dam (WWDSE, 2014).

According to WWDSE (2016) the material used as a backfill beneath the conduit is alluvium material with a fine normally consolidated soil and according to Unified Soil Classification System (USCS) -Plasticity chart, the soil is silt of high compressibility (MH) containing organic material.

The requirement of shell materials is semi pervious materials silty sands or gravels (SM or GM). Sands with dual symbol classification such as SW-SM, SW-SC, SP-SM, or SP-SC [i.e. sands having as high as 12 percent passing the 75-µm (No.200) sieved]. Three quarries were assessed and investigated and bounded reservoir area. Shell quarry-1 and 2 composed of weathered ignimbrite while shell quarry-3 is weathered and fractured basalt. These shell material contain 19-82% gravel, 6-18%, sand and 10-55% silt. Shell quarry-1and -3 have relatively small amount of fines and are semi free draining and less weathered. So it is more appropriate to be used as shall material (WWSSE, 2010).

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CHAPTER- 3 THE STUDY AREA

3.1 General

The Gidabo dam is constructed on Gidabo River which is located in Oromia Regional State, 377 Km from capital city of Ethiopia. The study area is accessible by 360 Km asphalt road from Addis Ababa to Dilla town and the rest 17 km by gravel road. The Gidabo catchment is located in Borena zone in Oromia Region, Sidama Zone, and Gedeo Zone in SNNP Region (BirhanuDebisso, 2009).

3.1.1. Project background

According to WWDSE (2009) the Gidabo River basin study was initiated in 1975 to assess the resource and development potentials of Rift Valley Lake Zone. In 1990 WAPCOS (India) Limited conducted master plan study of water resources of the country and identified Gidabo irrigation project as one of the potential projects in Rift Valley Lakes Basin. Later on in1992 Sir William Halcrow in association with ULG Consultants made reconnaissance master plan study for development of natural resources of the Rift Valley Lakes Basin. The study identified some potential irrigable areas in Lake Abaya-Chamo basin. During this study Gelana, Gidabo and Bilate irrigation projects were proposed, aimed to irrigate a total net irrigation command area of 31, 900 ha. In 1998 TAHAL Consulting Engineers Ltd. in association with Metaferia Consulting Engineers PLC (MCE) made feasibility study (preliminary assessment) in the proposed project area and submitted interim report covering their findings regarding physical resources of Gidabo Irrigation Project Area. Approximately 7,260 ha was identified suitable for providing irrigation by TAHAL and MCE on the basis of topographical and soil surveys (WWDSE, 2009).

From the preliminary hydrological study it was observed that in general, there is considerable inflow of Gidabo River at the dam site and therefore a detailed study was made to assess if a diversion structure without storage, instead of a dam will fulfill the requirement of the project. Low flow analysis at Gidabo dam site was made which indicates that in December, January and February 80% dependable river flow is less than the downstream irrigation requirement, as a result of which it is necessary to store some water during these three months. Therefore, a dam was proposed, instead of a barrage or a weir, for the Gidabo Project. Moreover, considering the topography, it was observed that the irrigable area that can be fed by gravity by a dam will be more than that by a barrage or weir since the operating level of the dam will be much higher than that of a barrage or a weir. Consequently, with the same head of lift the total area that can be covered in area of a dam will be much more than in case of a barrage/weir. It is also pertinent to mention here that flood spills from Gidabo River periodically submerge about 6000 ha areas and convert them into temporary swamps. These areas have very high agronomical potentials. If a dam is constructed, it will also moderate the flood and a considerable area can be reclaimed making it fit for agriculture which can be put to use for sugarcane or rice farming in future. Due to this Gidabo irrigation project expected to farm 1300 ha (WWDSE, 2010).

The dam is on construction by Ethiopian water works construction Enterprise (EWWCE) under Water Works Design and Supervision Enterprise consultant. The owner of the project is Ministry of Water, Irrigation and Electricity and the water will sell to the investors which are currently active in that area (WWDSE, 2016).

3.1.2. Salient features

The dam is Earthfill dam with central clay core and the maximum height of the dam over the deepest river bed is about 23.8 m. The dam is 335 m in crest length and 8 m top crest width with side spillway and one central outlet conduit is provided which will divert water in to the right and lift canals off taking from the dam on river Gidabo (Fig. 3.1).

The sediment distribution calculation based on area-reduction method showed that the 50year dead storage levels for Gidabo reservoir is at 1209 m a.m. s. l. for Gidabo reservoir. The remaining live storage, above 1209 m elevation will be 12.83 MCM for FRL of 1215.5 m. below the dead storage levels indicated above, all the storage is occupied by sediment (3.64MCM). The remaining sediment (18.97 – 3.64 = 15.33 MCM) is distributed all over the reservoir above the dead storage level (WWDSE, 2009).

Gidabo irrigation conduit is horse shoe type concrete structure with internal head room of 3.6 m, conduit thickness of 0.6 m, foundation pad thickness of 1.2 m and with width of 7 m. for water tightness additional 2.2 m diameter internal steel pipe lining within the conduit with thickness of 4 mm. The steel pipe is reinforced by bar anchorage from inside for the bottom and sides part and filled with 1.5 m thickness of second stage concrete for the crown part. The foundation along the conduit is 10 m thickness of alluvium backfill at shell part and 13.158 m thickness of clay at the center beneath the clay core (Fig. 3.2) (WWDSE, 2016).

Plate 3.1 View of the dam from left side down stream

Plat 3.2 View of the outlet conduit during the construction

3.2. Physiography

The physiographic features of the MER are mainly the results of faulting and volcanism associated with rifting processes. The landscape of the Gidabo River Catchment can be broadly categorized into four large groups: The edge of Eastern high plateau, the large eastern escarpment of the Rift Valley, the Structural Basaltic reliefs and the floor of the Rift Valley. The major tectonic scarp connects the rift floor with the uplifted plateau; the plateau rises to elevations of 3200 m a.s.l., whereas the rift floor descends regularly into the Lake

Abaya, where it lies at 1175 m a.s.l.. Local increases in the elevation of the rift floor are generally due to volcanic edifices and step faulting (Raunet, 1977 as cited in Birhanu Debisso, 2009; WWDSE, 2007, and Abraham Mechal et.al., 2015).

Gidabo dam is located within the rift floor lowest elevation 1205 m a.s.l. at the valley plane and it is decreasing towards the command area. On the upper part of the hill it rises up to 1290 m a. l..

3.3. Climate

The main rainy season in the Gidabo River Basin is between March and June with a second peak in September to October. These two peaks are separated by a relatively small rainy season in July to August. The main dry season is between November and February (Raunet, 1977 as cited in Birhanu Debisso, 2009). The mean annual rain fall in the north tip of Lake of Abaya is 818 mm and 745.1 mm at Mirab Abaya (WWDSE, 2008).

The areas adjacent to the Abaya Lake are characterized by arid Kolla climatic zones (Habtamu Eshetu, 2014). The annual monthly temperature at Gidabo dam is in the range 15^o to 30^o C. The command area is relatively hotter as it is the lowest part of the catchment, daily temperature may reach 36 to 40 ^{o} C. Maximum average temperature is attained during the month of February and March, in July the minimum temperature is observed (WWDSE, 2007; 2008).

3.4. Hydrogeology of the study area

Gidabo dam is located in Gidabo River Catchment which is part of Abaya sub-Basin and found within Rift Valley Basin. Major river system is Gidabo River it collects small streams in the direction of NS and Eastern direction near the Lake Abaya. Gidabo River Catchment covers the whole of the hydrographical system of the Gidabo River which rises in the highland area of the Aleta Wondo Escarpment, joining numerous large streams, draining an extensive catchment and flowing into the Lake Abaya as the Eastern tributary (Seleshi, 2000).

The Region drained by the Gidabo River is bordered by the catchment of the: Lake Awassa to the North, River Bilate to the West, River Galana to the South and Genale-Dawa Rivers to the East. The absolute geographical location of the area is between $6^{\circ}9^{'}$ to $6^{\circ}57'$ N latitude and 38° to $38^{\circ}38^{'}$ E longitude with an area and perimeter of 3342.37 km^2 and 305.25 km, respectively (Birhanu Debisso, 2009).

According to Berhanu Debisso (2009) the main recharge area for the Gidabo River Catchment is the eastern high plateau lineament including mountain Geremba, town Hagereselam and town Bule areas. The escarpment areas (Aleta Wondo, Teferi Kella, Kebado and Wonago) are relatively semi-humid region possessing slightly to highly weathered and fractured volcanic rocks, silty clay to clay loam soil regolith, good vegetal cover and high human population density. Thus, the rift escarpment areas are very good groundwater recharge regions of the Gidabo River Basin. Few part of the rift floor town areas such as; Leku, Dilla, Chuko and Morocho are relatively semi-humid to semi- arid region possessing slightly to highly weathered and fractured volcanic rocks, pyroclastic fall deposits, sandy-silt-clay loam to silty clay loam soil regolith, an important vegetal cover. As a result, the mentioned parts of the rift floor areas are good groundwater recharge zones in the study area. Discharge area of Gidabo River Catchment starts at rift floor from partially Dilla and YirgaAlem towns towards Lake Abaya. The Gidabo irrigation project is found within the discharge area of the Catchment.

3.4.1. Ground water depth

Ground water in Gidabo irrigation project is expected to be shallow depth since this is found within Gidabo delta (discharge) area of the catchment and it is near to marshy area which indicate shallow ground water (WWDSE, 2007).

During the site investigation installation of two piezometers were done on borehole GIBH-3 and GIBH-5. A simple stand pipe consisting of a PVC tube with perforated 1 m at the depth of 32.5 m and below the perforated around 14.6 m depth of back filled with gravel without piezometer, surrounding by a granular filter in the expected aquifer zone was placed in GIBH-3 at depth of 47.26 m. The other piezometer was installed in GIBH-5 with borehole bottom 25 m and the 1.5 m perforated of the total height 24 m piezometer the rest 1 m is gravel fill without PVC. In addition to this during the excavation of the spill way foundation ground water table level was 1200 m.

Table 3.1 Observed ground water depth on the dam axis (Source: WWDSE, 2007)

Borehole name	Location	Depth of ground water below surface
GIBH-3	Left bank, near upstream	4.5 m
GIBH-5	Right abutment, foot of hill	5.5 m
Foundation excavation	Spill way	5 m

3.4.2. Surface water

The surface water on the catchment is dominated by Lake Abaya and by the other rivers. The marshy area that is found on the delta, which is between the project and the Lake Abaya, is also a major part of the Gidabo River Catchment surface water. The cold and thermal springs occur within the catchment (WWDSE, 2007)).

According to WWDSE (2007)Gidabo River is gauged at Aposto (646 km^2), Kolla (206 km^2) and Bedssa at Dilla (80 Km^2).This three stations contribute about 75% of the flows of the catchment. In addition to this short record length (1997-2005) data, was done from Gidabo River at Meissa close to the project area. This station effectively used to transfer the long period flow condition from upstream stations to dam site using regression analysis. Generally, the discharge is found to be 211.8 MCM per annum at Aposto (1977-2005), 60.52 MCM per annum at Bedssa on Dilla (1982-2005), 81.6 MCM per annum at Kolla near AlataWondo (1975-2005) and 507 MCM per annum at Meissa (1997-2005).

3.5. Geology of the study area

3.5.1. Regional Geology

The Main Ethiopian Rift (MER) is a NNE–SSW to N–S trending trough 80 km wide in its central portion and 1,000 km long. It separates the southern Ethiopian plateau to the west from the Somali plateau to the east. Northward, the MER progressively widens out into the complex Afar triple junction, while at its southern end, a 200–300-km tectonically disturbed area (Baker et al. 1972) marks the transition to the Kenyan Gregory Rift in the Turkana depression.

According to Corti (2009)the MER volcanic stratigraphy characterized as a lower basalt unit with trachy basalts and subordinate silicic flows from 11 to 8 Ma old followed by a widespread ignimbrite cover (e.g., Nazaret Group) ranging in age from 7 Ma in the northern sector to 2 Ma to the south and up to 700 m thick. Most of the ignimbrite layers are believed to have formed by catastrophic eruptions related to the collapse of large calderas, such as the 3.5-Ma old Munesa caldera now buried beneath the Ziway–Shala lakes.From a morphological and geological point of view, the MER has been subdivided into three main segments: the northern, central, and southern (Mohr 1983; Woldegabriel Gebremedhin et.al. 1990; Hayward and Ebinger 1996; Bonini et al. 2005). The northern MER funnels from the Afar depression, where it is about 100 km wide, to the 80-kmlong Dubeta Col sill (north of Ziway Lake). The central MER, which is 80 km wide, includes most of the lake region and extends southward up to the W/E Goba–Bonga line. This portion of the MER has an average elevation of 1,600 m, and the lowest altitude is at Lake Abiyata (1,580 m).

At the Bonga–Goba line, the southern MER narrows up to 60 km, shifts to a N–S trend, and reaches an elevation of 2,000 m, decreasing southward to 1,000 m. From its middle portion to the south, the southern MER bifurcates into two branches (the Lake Chamo and Galana river rifts) separated by the 3,000-m-high Amaro horst (Mohr, 1967). The southern MER keeps its morphological identity until the Sagan line.

The MER is continuously bordered on the two sides by crest lines, with abrupt transitory scarp faces overlooking the rift valley floor. The eastern crest line elevations vary between 2500 m and 3000 m. On the western side, the elevations of rift shoulders are comprised between 1800 and 3500 m. The rift is segmented into graben basins, each one asymmetric, the graben fill dipping east, and the major fault lying along the eastern side. A narrow fault zone, the Wonji Fault Belt (Mohr, 1962) obliquely crosses the MER between northwestern border in the north and southeastern border in the south. The MER ends near Lake Awasa.

According to Abraham Mechal et. al (2016)the rocks covering the Gidabo River Catchment can be categorized into three major groups: pre-rift volcanic rocks, rift volcanic rocks and post rift sediments.

The Pre-rift rocks (Oligocene-Middle Miocene) occur mainly in the escarpment and highland and to a lesser extent in the rift floor. This group mainly comprises basalt and ignimbrite and represents the oldest rocks in the area, likely separated from the underlying basement by the residual sandstone to the south of the catchment (Abraham Mechalet. al, 2016).

Rift volcanic rocks (Upper Miocene Pleistocene) are mainly exposed in the rift floor and dominated by silicic volcanic rocks. A thick succession of stratoid silicics comprising predominantly ignimbrites with subordinate un-welded tuffs, ash flows, rhyolites and trachytes, which is commonly known as the Nazreth group form parts of the rift floor and also outcrops in the escarpment and highland. In the rift floor, the Nazreth group is unconformably overlain by younger volcanic rocks called Dino formation which comprises coarse un-welded pumiceous pyroclastics and a complex mixture of different pyroclastic materials such as ash, tuff and ignimbrite (Abraham Mechalet. al, 2016).

Rhyolitic lava flows, composed of stratified ash, pumice and rhyolite flows mainly occur to the north of Lake Abaya along the axial zone of the rift but similar prominent volcanoes also have erupted pumice and unwelded tuffs forming volcanic mountains in the highland. Post-rift sediments (Holocene) such as alluvial and lacustrine sediments mainly occur along the lower reaches of the Gidabo River and as patchy deposit along the axial zone of the rift, respectively (Abraham Mechalet. al, 2016).

The volcanic sequences and sediments in the area are densely dissected by extensional fault systems resulting from the rifting process. The major fault types are normal faults having generally similar strike but some dip to the east and others to the west (Abraham Mechalet. al, 2016).

3.2.2. Local Geology

The following lithological units have been identified at the dam site, reservoir and the command area. The lithological units are pyroclastic rocks (volcanic breccias, tuff and Ignimbrite) and rhyolite rocks (Fig.3.1). Bedrocks exposed on the hills and low-lying topography near the dam axis and reservoir area consists of a sequence of inter-bedded pyroclastic fall deposits and rare Tertiary basic lava (WWDSE, 2008).

Pyroclastic rock

The pyroclastic sequence consists of poorly welded ignimbrite, graded fall deposits that are moderately to strongly weathered and volcanic ash. The deposits are from a central volcano. Besides the poorly welded ignimbrite the pyroclastic fall deposit contains various lithic tuff layers through the reaches of the reservoir.

The ignimbrite unit is poorly welded and locally shows jointing. Occasionally pumice fragments are flattened. Cavity filling of secondary minerals is also common. The ignimbrite is overlain by white to light yellow, fine grained, well-bedded tuff possibly reworked.

The volcanic breccias form the top most part of the left abutment ridge. It is blocky in nature and very hard. It is light colored, moderately weathered, brecciate and rhyolitic in composition. It contains angular fragmental rocks set in fine-grained to glassy groundmass.

This type of rock is found on the right side of the canal at the distance of around 1 km downstream.

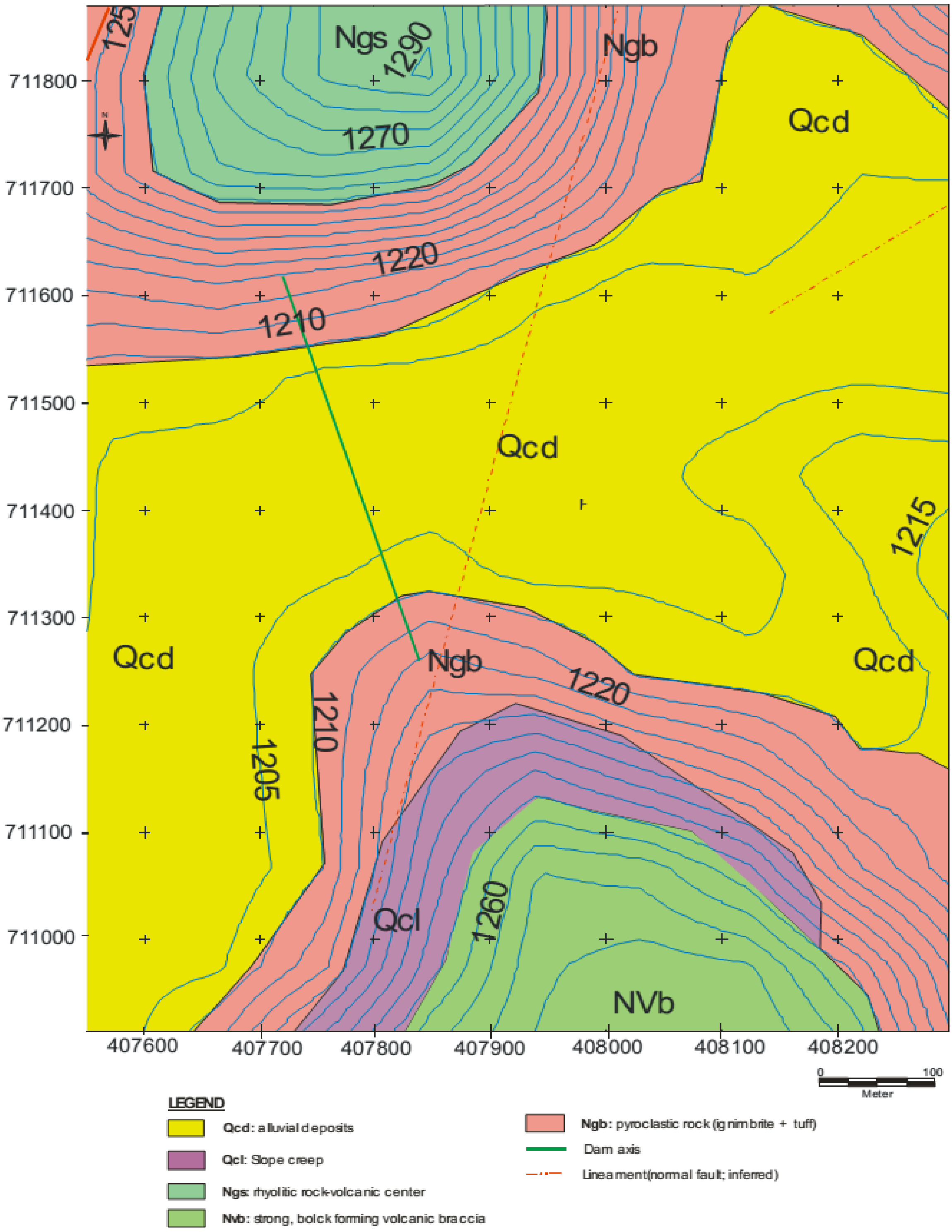

Fig. 3.1 Geological of Gidabo Dam site (WWDSE, 2008)

Below the volcanic breccias the friable lithic tuff contains undulated clasts of pumice set in sandy sized volcanic groundmass. It is generally reddish brown to light gray, moderately friable with weakly strong. This type of unit also found on lift abutment beneath ignimbrite.

The ignimbrite that forms the lower part of the above unit around the left abutment is light gray to greenish gray, weakly weathered to fresh, moderately strong (strongly welded) porphyritic in nature with three set of joints. The tuff (or ignimbritic tuff) below ignimbritic unit is weak rock (lateral variation in strength can be found) containing crystal and lithic fragments. It is light gray to brownish gray in color.

Rhyolitic Rock

The rhyolite forms a very step part of the right abutment or a plug. The unit is fine grained,light in color and fresh on the outcrop. The Rhyolitic flow covers the surrounding area of thetop of the right abutment (Fig.3.2).

Alluvial Soil

Alluvial deposits are transported by running water and settled when the speed of the flowing water is no longer sufficient to carry them. This deposit is restricted to low-lying area closeto river course and foot of ridges and hills (Fig.3.2).These are sand, silt and clay with gravel that have been deposited in the channels and aroundmargins of Gidabo River and its tributaries.Fine silt and clay are deposited on thin horizontal layers during floods. This is mainly observed on both the banks, right and left bank of the river. Alluvial deposits in the flat area of the dam site which have been deposited in the channels and flood plains of the rivers.

The foundation of the outlet was constructed on this type of soil including the backfilled material. Most of the upper part of the soil was excavated and filled with other type of soil but it is from the soft alluvial soil from the river bed.

3.6. Seismicity of the Area

According to the seismic risk map of Ethiopia 100 years return period, 0.99 probabilities by Laike Mariam Asfaw, (1986) the country is divided into zones of approximately equal seismic risks based on the known distribution of the past earthquakes. According to Johnson and Degraff (1988) as stated in Negatu Fikadu, 2006, these seismic intensity zones are related to the ground acceleration as follows;

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

The Gidabo Dam Project area falls in the intensity scale 8, thus accordingly the estimated ground acceleration as per Johnson and Degraff (1988) will be 0.2g.

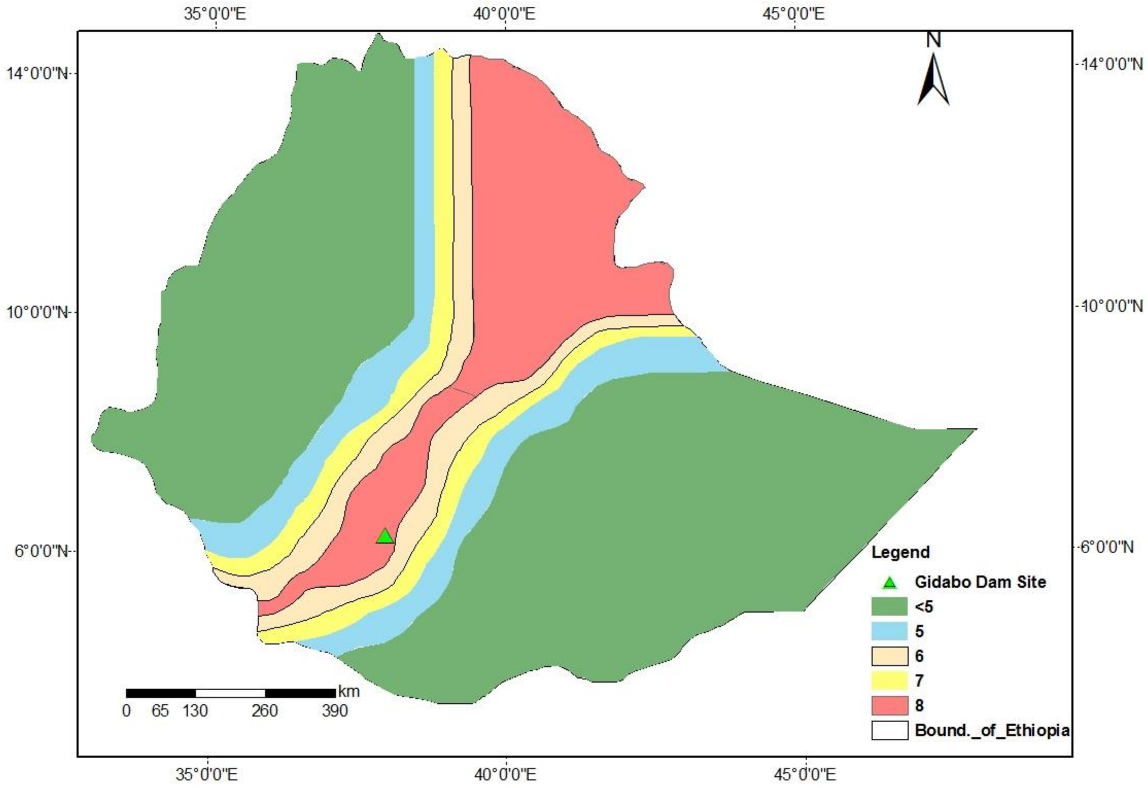

Fig. 3.2 Seismic map of Ethiopia modified after Laike Mariam Asfaw, (1986)

The intensity scale 8 indicatesthat the project area lies in the high seismic risk zone. In addition to this the dam is constructed on 40 m deep soil foundation which can amplify the amplitude of seismic waves several times comparing to bed rock foundation (WWDSE, 2008).

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CHAPTER- 4 Methodologies

4.1 Data collection

During the present study the data from secondary sources was obtained from different reports. Besides, raw data from Gidabo Dam site was also utilized. In addition to these representatives samples were also taken from the borrow site of the fill material, used as foundation material for the conduit structure.

The specific data collected in this regards is summarized as follows;

(i) Geological information about the dam site both Regional and Local geology of the area.
(ii) Design reports and geometry of the dam, excavation level readings especially on the main section of the dam.
(iii) Geotechnical properties of the dam foundation (Atterberg limits, shear strength characteristics, consolidation parameters and SPT test values).
(iv) Observed settlement in the foundation of the dam.

4.1.1. Primary data

To collect the primary data field investigation was done with field description and laboratory test which includes identification of rock units and their structure at the dam section and its surrounding area, soil samples were collected from the borrow area. Later, soils were classified and identified based on the index tests. Disturbed samples were collected from original borrow site which used for the backfill material as foundation.

The main index properties of coarse-grained soils that were used are particle size and relative densities whereas; for fine-grained soils the main index properties that were used are Atterberg limits and consistency (Arora, 2003).

The soil that was used as aback fill at conduit site was fine grained with more than 90% fine therefore the basic index properties (Atterberg limit) were determined at the site laboratory and in the soil mechanics laboratory at School of Earth science, Addis Ababa University. Further, for the determination of silt and clay content in the soils, and test for Atterberg limit were done in the soil mechanics laboratory at School of Earth science, Addis Ababa University. In addition to tests compaction test, Atterberg limit and sieve analysis was done at the project site laboratory. The unconsolidated undrained tri-axial test (UU) was done in laboratory of Ethiopian Water Works Design and Supervision Enterprise, Addis Ababa.

4.1.2. Secondary data

During the present study various data were collected from the project consultant, Ethiopian Water Works Design and Supervision Enterprise, Addis Ababa. In addition to this, during the field work additional secondary data has been collected from the consultant and contractor's site office. The secondary data procured were hydro-geological, geotechnical, geological and design reports (dam and appurtenant design). Besides, raw data on soil laboratory test, in-situ test, and excavation level and water pressure tests were also procured. The main data that was used during the analysis of settlement of the foundation was compressibility properties of the backfill material, data on SPT – N value and the index properties for the foundation material beneath the backfill material.

The settlement problem at Gidabo Dam foundation is basically associated with the immediate consolidation, as it occurred during the construction period within a very short period of time. After the settlement was observed efforts were made by the project authorities to determine the possible causes and to workout feasible mitigation measures. For this, representative samples were collected from the foundation soils of the conduit. Four samples were taken from the foundation near to the conduit and two samples were collected from the representative borrow site, from where construction material was used as a backfill for conduit outlet foundation. Later, the collected samples were tested for classification test, Atterberg limit test and compaction tests at the site laboratory and odometers consolidation test and direct share test were conducted at Water Works Design and Supervision Enterprise geotechnical laboratory, Addis Ababa. In addition to this, the central part of the conduit outlet foundation which is constructed on the clay, similar to the core material, samples were also collected and tested.

4.2. Data Evolution and Analysis

4.2.1. Data Evolution

Different methods were employed to analyze settlement of conduit structure within Gidabo dam. The methods used for this purpose are settlement analysis methods and empirical. A procedure for the computation of anticipated settlements is called Settlement analysis. This analysis may be divided into three parts. The first part consists of obtaining the soil profile, which gives an idea of the depths of various characteristic zones of soil at the site of the structure, as also the relevant properties of soil such as initial void ratio, grain specific gravity, water content, and the consolidation and compressibility characteristics. The second part consists of the analysis of the transmission of stresses to the subsurface strata. The final part consists of the final settlement predictions based on concepts from the theory of consolidation and data from the first and second parts (Venkatramaiah, 2006).

During the analysis most of the lab results were taken as an average value for the alluvium fill type from different samples which were obtained from the four locations (chainge) of the foundation and two samples from original borrow site for alluvium fill. For the core portion that was used as cutoffs beneath the outlet conduit, average values of consolidation tests from four samples from borrow area were used.

Further, physical tests, strength tests and compaction test results from the laboratory and in situ test results during the construction were also evaluated during the present study. All these data were used to determine the possible qualitatively settlement. The properties of the soil and SPT - N value were used to determine the settlement potential of the foundation below the alluvium fill and clay cutoff of the dam.

The settlement analysis of Gidabo Dam was done by dividing the embankment section in to four parts along the dam axis. More emphasis was given to the portion along the outlet conduit. The first two parts were from Chaing 0+045 to 0+215 which is filled by the alluvium soil and the outlet conduit is within this range. The third part is from chaing 0+215 to0+245 which are filled by the clay material. The last part is from Chainge 0+245 up to the right side which is filled by compacted granular soil similar to the shall material.

4.2.2. Settlement Analysis

During the present study the immediate and consolidation settlement were determined. The secondary compression settlement was not determined because the settlement of the conduit section happened within a short period of the time. As a matter of fact the secondary consolidation of mineral soils is usually negligible however it may be considerable in the case of organic soils due to their colloidal nature. This may constitute a substantial part of total compression in the case of organic soils, micaceous soils, loosely deposited clays, etc. (Venkatramaiah, 2006).

To predict the settlement amount on the foundation of the dam which is used for comparing with actual settlement different methods were used during the investigation. Settlement analysis methods and empirical relationships were used to determine immediate and primary consolidation. For some parameters of foundation units especially below the backfill (excavation level) there were gaps of data so to fill the gaps of data empirical relationships and taken some values were taken from published literatures.

Most of the analysis focused on the backfill materials of the foundation including the alluvium fill and the cutoff and compressible clay layer below the excavation level because of those layers goes to the considerable depth and there is fine grain material which is susceptible to settlement. Even though influence of change of stress due to load of the outlet conduit and embankment fill decrease with depth efforts were made to determine the settlement amount of the sand to gravelly sand part of the foundation by using empirical relations from SPT values for elastic settlement.

Immediate settlement

The immediate settlement of the cohesive soils which is the Alluvium backfill, clay cutoff and compressible layer (below the excavation level) were predicted by using Terzaghi' s equation eq.2.1 and eq.2.2. An earth embankment may be taken as flexible and eq.2.1 was used to determine the immediate settlement of the soil below such a construction. For the conduit outlet foundation eq.2.2 was convenient since the foundation is rigid.

The most important parameters of Terzaghi's equation are uniform pressure on the foundation, Modulus of elasticity of the soil beneath the foundation, Poisson's ratio of the soil, width and length of the foundation. Poisson's ratio of the soils was taken from literature and Modulus of elasticity of soils is determined by using empirical relation from coefficient of compressibility m_v (Bowles, 1997).

$$E_s = \frac{1}{m_v} \text{.................eq. 4.1}$$

The width of the outlet conduit is 7 m and the total length more than 140 m from the entrance up to the diversion to the canal. However, for this analysis purpose the total length of the outlet conduit was dividing in to three parts which is the upstream, downstream and central part of the dam because of the backfill material as foundation is different. The width for the sections considered as 10 m. the length of the foundation on up/down stream shall is 55 m and 30 at the center of the dam portion.

In addition to the Terzaghi's equation elastic settlement was determined from SPT-N value by using Terzaghi and Peck eq.2.3 for the sand and gravelly sand part of the foundation. First

N values was corrected and converted to N_{60} then it was taken the average value of N_{60} from different part of foundation with the same type of soil. Ground water table correction was taken as 2 since the ground water on the foundation is less than the foundation width.

Janbu approach

The Janbu method is widely used internationally and by several North American engineering companies and engineers. However, many others are yet reluctant to use the Janbu approach, despite its obvious advantages over the conventional Cc/e_o method (Fellenuis, 2015). In Ethiopia case the Janbus approach not commonly used. During the present study it used for analysis purpose and to compare with one dimensional method. This approach was discussed in chapter 2 and the main equation for the cohesive soils eq. 2.4. The main parameters are modulus number, original effective stress and final effective stress. The effective stress was calculated as below stress determination part. The modulus number was calculated from compression index and initial void ratio by using eq. 2.8 and eq.2.9.

Primary settlement

One dimensional consolidation

Normally consolidated soils are usually found as recent alluvial deposits, and are mainly composed of silt and clay sized particles. The present investigation was done by considering soils to be normally consolidated soils.

One dimensional consolidation was done by using eq. 2.10. The parameters used during the analysis are compression index, void ratio, original effective stress and change of vertical stress due to the load. The first two parameters were found from oedemeter test of the soils for the alluvium backfill and clay cutoff part. Compression index of the compressible silt clay below the excavation level was determined from liquid limit, LL by using Sekmpton and others, Tarzaghi and Peck (1948) equation it's applicable to normal consolidated clays of low to moderate sensitivity as follow.

$$C_c = 0.009 * (LL - 10) \text{.................eq. 4.3}$$

During the feasibility study undisturbed sample were collected and grain size analysis, Atterberg Limit, specific gravity and bulk density test were done. Initial void ratio of compressible silt clay was calculated by the derived equation eq.4.4 from eq.4.3 relationships from between specific gravity and saturated unit weight of the soil. Since the sample was taken from 19-20 m i.e. below ground water table level and the sample assumed to be fully saturated so the saturated unit weight assumed to be equal to the bulk unit weight.

$$\gamma_{sat} = \frac{G+e_o}{1+e_o}\gamma_w \text{..................eq. 4.4}$$

$$e_o = \frac{G*\gamma_w-\gamma_{sat}}{\gamma_{sat}-\gamma_w} \text{............eq. 4.5}$$

Where;e_o =initial void ratio,γ_{sat}= saturated unit weight, γ_w=unit weight of water (9.81kN/m^3) and G=specific gravity.

Time Rate of Consolidation

Determination of settlement rate for the foundation of the dam used to determine the main cause of the settlement on the conduit by comparing the predicted settlement in time with actual one. Consolidation rate of the dam was done in time during the construction (half year) after 5 years, 10 years and 20 years. The main input parameters are coefficient of consolidation C_v, dimensionless time factor T_v and the predicted primary consolidation of the dam at different section by using one dimensional consolidation. In the present study the time rate of consolidation was done only for the backfill material of the foundation (Alluvium backfill and clay cutoff) because of the consolidation tests data available this part.

The time factor T_v is depend on the coefficient of consolidation and the drainage path. The drainage path is the maximum distance that water travels to reach the free drainage boundary. The drainage path of Gidabo dam is varying at different section of the dam. The layers of the sections at chainage 0+115 and 0+135 are half-closed layers (one way drainage) due to the existence of the clay layer below the backfill layer and the drainage path is the thickness of the layer. For sections at chainge 0+235 and 0+250 also half thickness layers (one way drainage) was considered.

Stress Determination

In the present study the change in the vertical stress due to the embankment load on the conduit was considered to be the load due to shell material and the load due to central core clay. Long earth embankments with sloping sides represent trapezoidal loads. The basic problem is to determine stresses due to a linearly increasing vertical loading on the surface. The vertical stress due to the embankment loading, σ_z is determined from methods of superposition as following (Das, 2008).

$$\sigma_z = q * I_z \text{………………….eq.4.2}$$

Whereσ_z= vertical of stress due to the embankment load and I_z= the values of the influence factor for various a/z and b/z are given in fig 4.1

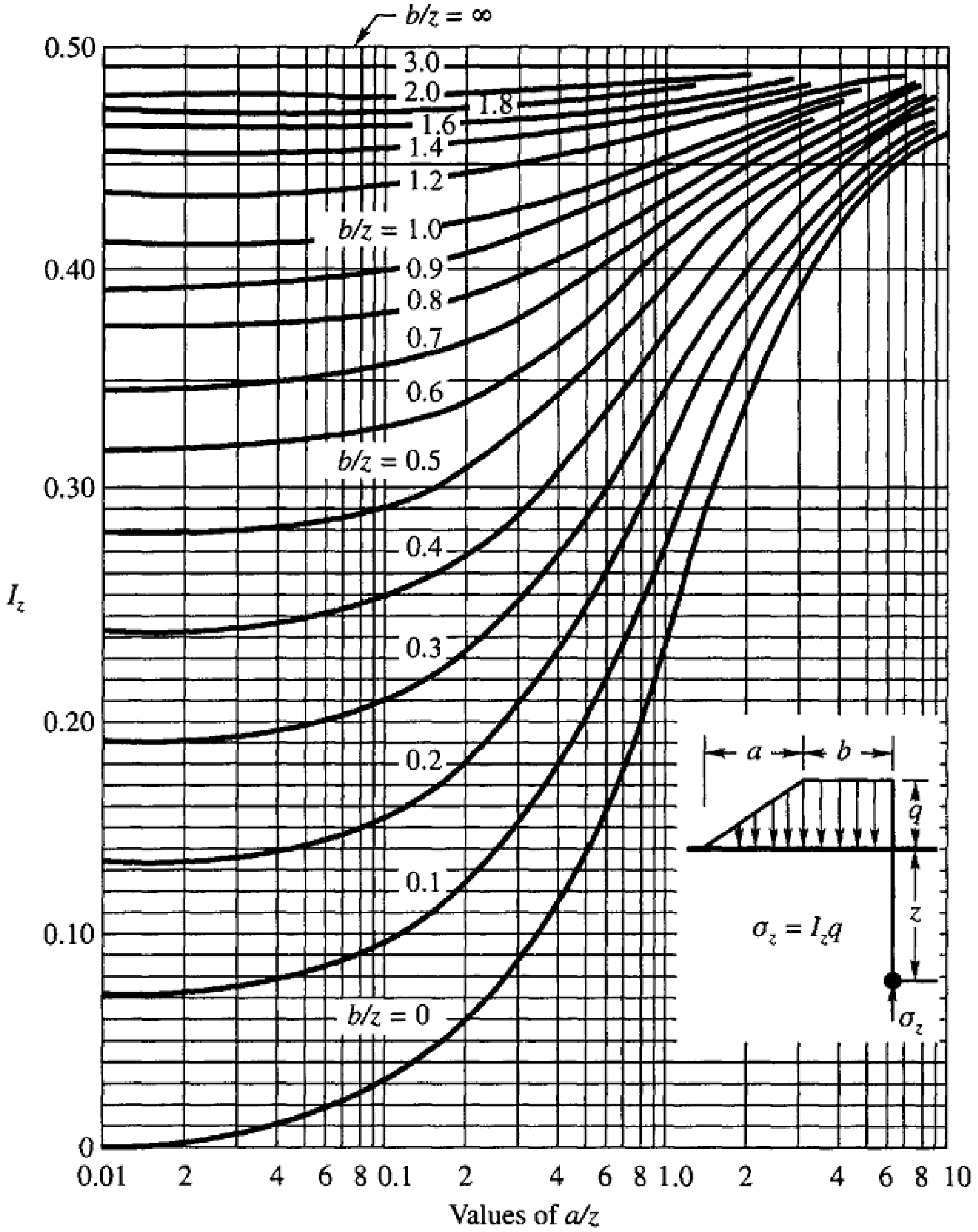

Fig. 4.1 Influence factors for embankment load (after Osterberg, 1957)

For the foundation below the shell part along the selected part eq4.51 was used to determine the vertical stress due to the embankment loading, σ_z ;

$\sigma_z = I_z * q$ …….eq. 4.51

When $q = \gamma_{emb} * h_{emb}$

h_{emb}= the height of the embankment at the center of the dam

For the foundation below the central of the dam portion of the foundation eq.4.52 was used;

$\sigma_z = 2 * q * I_z$ …….eq. 4.52

For the outlet conduit part change in pressure has been determined by eq.4.53.

$q = \gamma_{emb} * h_{emb} + \gamma_{con} * h_{con}$ …….eq. 4.53

Where; $\gamma_s, \gamma_c and\ \gamma_{con}$ = the unit weight of shell fill, clay fill and concrete, respectively. h_s,h_c,h_{con} and h_{emb}= the height of the shall fill, clay, concrete fill and embankment (shell or clay) above the conduit

The average initial (original) effective stress σ_o' is the total stress minus the pore pressure (the water pressure in the voids). Total stress at a certain depth below a level ground surface is the easiest of all values to determine as it is the summation of the total unit weight (total density times gravity constant) and depth.

Pore pressures are usually assumed to be zero in the zone above the groundwater table.
Original effective stress, σ_o' above ground water table is given as following relation:

$\sigma_o' = \gamma * H$ …….eq. 4.54

Original effective stress, σ_o' below ground water table

$\sigma_o' = (\gamma_{sat} - \gamma_w) * H_2$…….eq. 4.55

Total original effective stress;

$\sigma_o' = \gamma * H + (\gamma_{sat} - \gamma_w) * H_2$ …….eq. 4.56

Where; γ, γ_{sat}= bulk unit weight and saturated unit weight respectively, H= thickness of layer above ground water and H_2=thinness of layer below ground water

Fig. 4.1 shows the flow chart for general methodology followed during the present study.

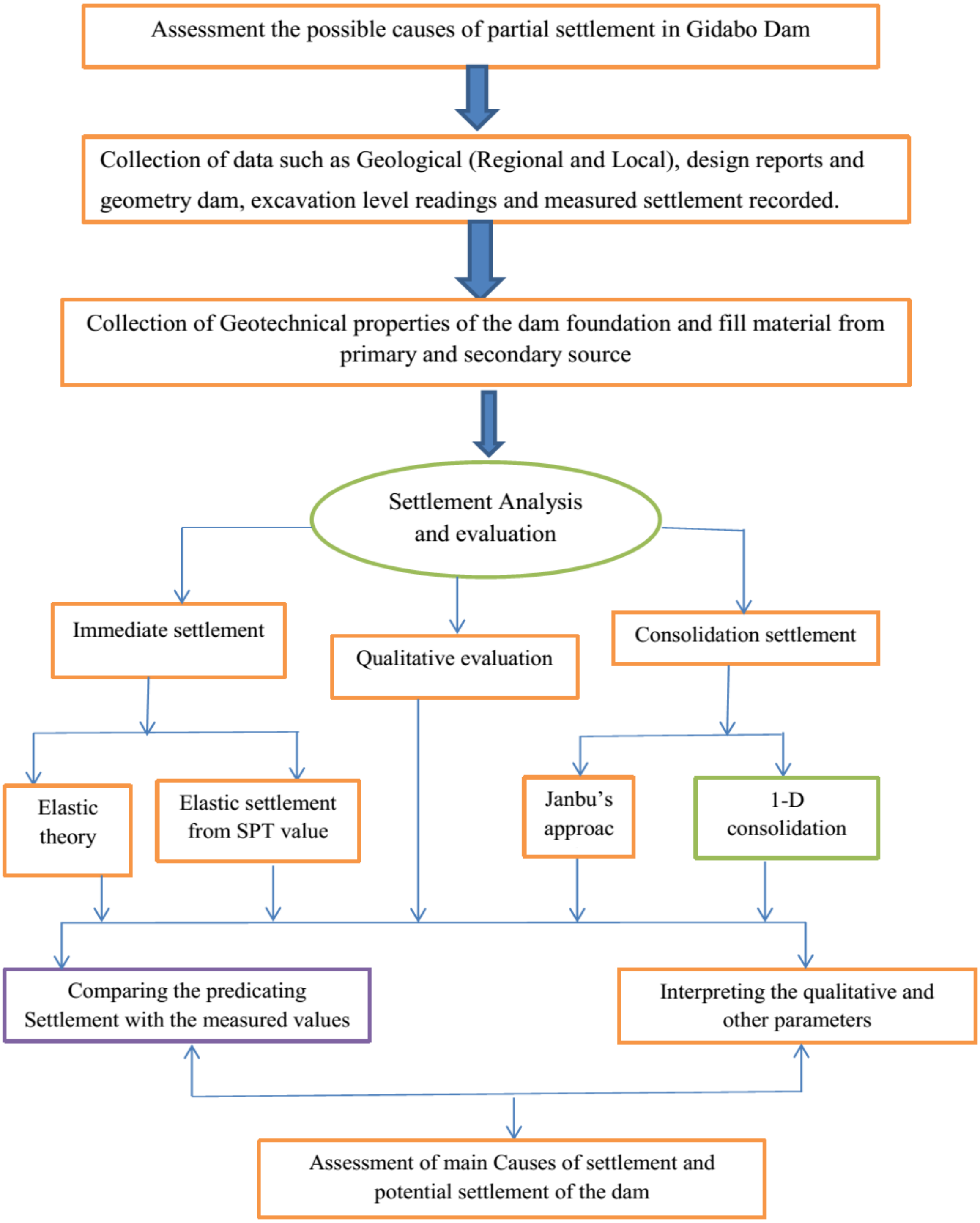

Fig. 4.2 Flow chart of methodology that was used during the present study

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CHAPTER- 5 DATA PREPARATION, PROCESSING AND ANALYSIS

5.1. Data preparation and processing

The data that was utilized for the present study is systematically prepared and presented in the form of tabular. The most important data which was used for the analysis during the present study is summarized below:

- ✓ Geological cross section and geometry of the dam at the main cross sections – chainages (Chainge 0+115, 0+135, 0+235 and 0+255) (Plat 5.1).

- ✓ Geotechnical properties of the dam foundation and backfill for the foundation (Atterberg limits, compaction test and shear strength characteristics, consolidation parameters, and SPT value, N). These data were procured from the feasibility study stage of the dam (2008) up to the primary data generation for the purpose of the present investigation.

Plat 5.1 view of dam and selected chainge location

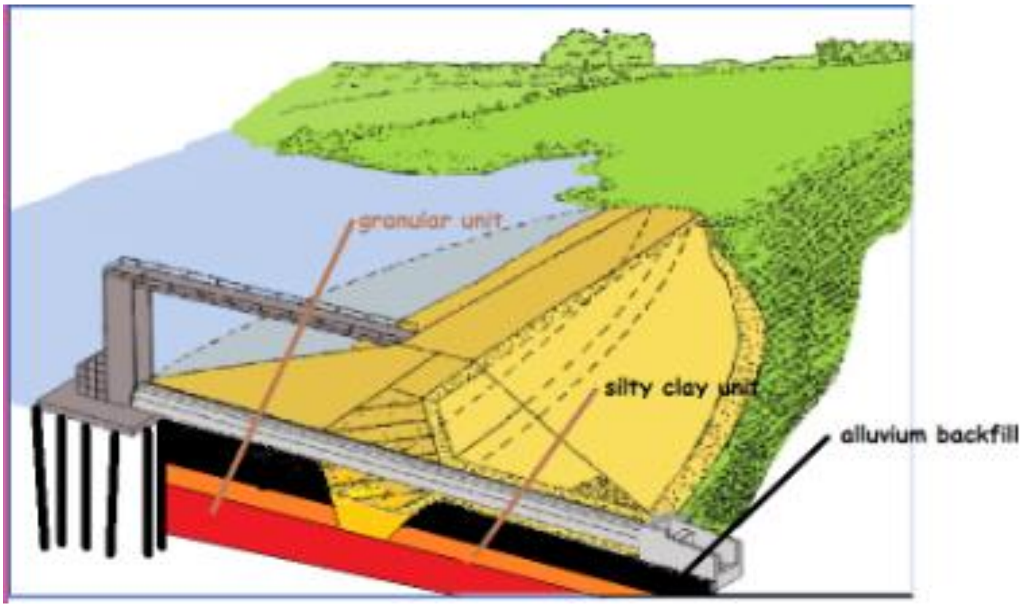

Fig. 5.2 systematic diagram of conduit outlet and the foundation material

5.1.1 Cross section and foundation units of Gidabo Dam

Backfills of the dam foundation was placed towards downstream and upstream of the dam and the central part of the foundation which was filled by the clay to serve as a cutoff. Soil units beneath the backfill were determined from geological cross section prepared during the feasibility study and from the available borehole log reports (Annexure VI). Results from the secondary sources have showed that below the backfill materials sand to gravelly sand or compressible silty clay layer is present. However, they are not clean sandy gravelly; it contains silty clay as pocket layers within this unit. Foundation units that were used during the present analysis are mentioned in Tables 5.1 to 5.4.

Table 5.1 Cross section and foundation units of Gidabo Dam at Chainge 0+115

Units	Thickness (m)	Elevation (m)
Alluvium backfill	9.4	1204.81-1195.45
Clay cutoff	13.4	1204.81-1191.42
Compressible Clay silt with sand	10.45	1195.45-1185
Sand to Gravelly sand	8	1185-1177
Weathered rhyolite and ignimbrite rock	10	1177-1166

This chanage is placed at the same position as that of GIBH-2 borehole and the alluvium backfill is at the same level to the foundation of the outlet conduit (chainge 0+135).

Table 5.2 cross section and foundation units of Gidabo Dam at Chainge 0+135

Foundation units	Thickness (m)	Elevation (m)
Alluvium backfill	9.75	1204.55-1194.8
Clay cutoff	13.15	1204.55-1191.4
Compressible Silt clay with sand	3.2	1194.8-1191.6
Corse to gravel sand with intermittent clay and silt layer	18	1191.6-1173.6

Outlet conduit is found in this chainge which is already settled by more than 40 cm.

Table 5.3 cross section and foundation units of Gidabo Dam at Chainge 0+235

Foundation units	Thickness (m)	Elevation (m)
Alluvium backfill near to camp	6.1	1204.8-1198.7
Clay cutoff	10.83	1204.8-1193.97
Sand to Gravelly sand	8.7	1198.7-1190

From chainge 0+215 to 0+245 filled by another type of backfill which is different from the alluvium backfill which used first section of the dam (0+46 to 0+215).

Table 5.4 cross section and foundation units of Gidabo Dam at Chainge 0+250

Foundation units	Thickness (m)	Elevation (m)
Granular soil backfill	6.5	1204.36-1197.9
Clay cutoff	11.2	1204.36-1193.2
Sand to Gravelly sand	7.9	1197.9-1990

Compacted Granular soil was used as a backfill from chainge 0+245-0+250 (the construction of the dam stopped at chainge 0+255). This part was constructed after the settlement of the outlet conduit was recognized. Due to this reason the backfill material was changed into from fine material to granular soil type which is similar to shall material.

5.1.2. Geotechnical properties of foundation backfill material

Grain size analysis and Atterbreg limit

For the present study grain size and Atterbreg test data was collect from the secondary source which was prepared during feasibility study, construction and primary test was conduct during this investigation for alluvium backfill to cross check the existing data (test PA-1 and PA-2). Data was grouped in to two: samples from foundation and samples from the borrow site. The first four samples were taken from the foundation with chainge 0+055, 0+115, 0+085 and 0+145, respectively. The next four samples were taken from the original borrow site. The second alluvium foundation fills which is used as backfill material from Chainge 0+215 to 0+245. At the end the table granular soil also presented which is used as backfill material from Chainge 0+245 to the right Abutment. It is summarize as follows:

Table 5.5 Grain size analyses and Atterberg limit test for backfill as foundation

Sample no.	**Grain size analyses**			**Atterberg Limit**		
	Sand %	**Silt %**	**Clay %**	**LL**	**PL**	**PI**
TP-1	0.64	63.04	36.32	60.2	40.08	20.12
TP-2	7.7	92.3		54.2	24.8	29.4
TP-3	5.9	94.1		60.2	26.6	33.6
TP-4	3.8	96.2		62.2	32	30.2
Pit-1	0.69	60.66	38.65			
Pit-2	0.59	59.42	39.99			
PA-1	2.2	97.8		54.2	36.9	17.3
PA-2	2	98		62.2	36.5	25.7
From chainge0+215-0+245 alluvium backfill 2						
Pit-1	16.82	56.56	26.62	36.80	18.2	18.6
Pit-2	12.22	55.78	32	38.33	20.2	18.13
From Chainge 0+245-0+255						
	Gravel %	Sand %	Silt %	Clay %		
TP-1	75.42	13.25	6.97	4.36		
TP-2	79.23	10.34	6.58	3.85		

According to U.S. Department of Agriculture System (USDA) soil textural classification system the soil of alluvium backfill from chainge 0+46-215 is classified as silty clay soil and for the second section is silty clay loam type of soil. For farther classification the unified soil classification system was used. The Alluvium backfill on the first section is grouped under CH or OH type of soil and for the second section is CL or OL type of soil. The soil group of the granular type of soil on section-3 in general it is gravelly soil.

The grain size analysis and Atterberg Limit of clay used for core and cutoff the dam foundation central part along the axis of the Gidabo Dam were determined and it is listed on the following table (table 5.6).

Table 5.6 Grain size analyses and Atterberg limit test for clay cutoff as foundation

Sample no.	**Grain size analyses**			**Atterberg Limit**		
	Sand %	**Silt %**	**Clay %**	**LL**	**PL**	**PI**
TRN No-1,CB-3	11.61	26.89	61.50	55.15	28.59	26.56
TRN No-1,CBA-1	6.33	33.67	60	75.45	30.90	44.55
TRN No-1,CBA-2	9.76	52.74	37.50	61	27.08	33.92

Clay used as cutoff and core material grouped under clay soil in terms of texture and the soil distinguished as CH or OH type soil.

Shear strength and compaction test data

To determine the shear strength parameters of the foundation units, direct shear test was widely used during design and construction of the dam. Tri-axial UU test was also done for the alluvium from the borrow site (table 5.8). The shear strength from the secondary data from the project Authority is presented in the table 5.7.

Table 5.7 shear strength parameter from direct shear test

Sample no.	**C**	**Ø**
0+045-0+215		
TP-1	50.55	11.70
TP-2	89.69	7.69
TP-3	89.69	12.19
TP-4	79.64	15.6
0+215-0+245 Pit -1	26.86	8.78
Clay cutoff		
TRN No-1,CB-1	70.04	36.02
TRN No-1,CB-3	114.20	39.49
TRN No-1,CBA-1	90.97	35.72
TRN No-1,CBA-2	147.20	38.19

Table 5.8 undrain shear strength from Tri-axial UU test for alluvium backfill

Sample	**C**	**Ø**
PA-1	44	13.98

Compaction tests were conducted in the site laboratory to determine the optimum moisture content (OMC) and maximum dry density (MDD). In addition to this in situ density tests were conducted during the construction. Most of the in situ tests agree 100+ with laboratory for those tests below 95%. It is recommended to compact again until it achieves the lab results. Some lab results of compaction test are presented as follow.

Table 5.9 Compaction test of foundation fill materials

Sample no.	**Maximum dry density (MDD)**	**Optimum moisture content (OMC)**
0+045-0+215		
Pit-1	1.57	28.5
Pit-2	1.5	26.5
Pit-3	1.56	27.9
0+215-0+245		
Pit-1	1.66	23.8
Pit-2	1.65	23.6
0+245-0+255		
TP-1	1.77	14.25
TP-2	1.73	18
Clay cutoff		
TRN No-1,CB-1	1.55	21
TRN No-1,CB-3	1.5	27.8
TRN No-1,CBA-1	1.4	32.5
TRN No-1,CBA-2	1.5	24

Consolidation tests data

Consolidation tests were conducted for the project during the design, especially for the clay that was used as cutoff fill and for the core material. After the settlement of the outlet, samples were taken from the foundation for the alluvium fill and the borrow area. So for these samples consolidation test using oedometer were conducted. The main consolidation input parameters used in the analysis are initial void ratio e_o,compression index C_c, coefficient of compressibility m_v, coefficient of consolidation C_v and specific gravity. The following table shows the main consolidation input parameters.

Table 5.10 the main consolidation input parameters

Sample no.	e_o	C_c	$m_v(m^2/MN)$	$C_v(m^2/year)$	**G**
0+45-0+215					
TP-1	1.36	0.249	0.2574	1.9904	2.54
TP-2	0.97	0.139	0.1674	2.3368	2.5
TP-3	0.97	0.113	0.2418	4.087	2.59
TP-4	1.16	0.163	0.1232	3.6744	2.54
Pit-1	1.26	0.203	0.228	2.4739	2.73
Pit-2	1.02	0.22	0.1105	2.408	2.66
Average	**1.12**	**0.181**	**0.188**	**2.828**	**2.6**
0+215-0+245 pit-1	**0.75**	**0.217**	**0.134**	**5.77**	**2.7**
0+245-0+255					
TP-1	0.60	0.1	0.2364	3.8372	2.55
TP-2	0.64	0.071	0.2254	2.0452	2.56
Average	**0.62**	**0.086**	**0.231**	**2.94**	**2.56**
Clay cutoff fill					
TRN No-1,CB-1	0.586	0.116	0.1524	3.0356	2.54
TRN No-1,CB-3	0.754	0.134	0.2021	4.7615	2.67
TRN No-1,CBA-1	0.847	0.234	0.186	2.4496	2.53
TRN No-1,CBA-2	0.879	0.154	0.1544	2.8308	2.53
Average	**0.77**	**0.16**	**0.174**	**3.269**	**2.57**

5.1.3. Additional properties of backfill foundation units

There are additional properties of backfill foundation units that were used during the settlement and stress distribution analysis. Those additional parameters are bulk unit weight γ, saturated unit weight γ_{sat}, poission's ratio v, modulus of elasticity Es and modulus number m. the bulk unit weight was calculated from maximum dry density times by gravity. Values poisson's ratios are obtained from the following table 5.10. Modulus number was determined by eq.2.8 and the Elastic modulus also determined by using eq. 4.1.

Table 5.11 Typical range of Values for Poisson's Ratio (Bowles, 1996)

Type of soil	**Poisson's ratios V**
Clay	0.4-0.5
Sandy clay	0.3-0.35
Silty	0.2-0.35
Sand (dense)	0.2-0.4
Coarse (Void ratio 0.4 to 0.7)	0.15

Table 5.12 additional properties of backfill foundation units

Material type	$\gamma\ (kN/m^3)$	$\gamma_{sat}\ (kN/m^3)$	V	**Es (kPa)**	M
Shell material	17.2		0.3		
Cutoff and clay core	14.6	18.5	0.5	5747	25.44
Alluvium backfill	15.2	17.2	0.4	5318	27
Alluvium backfill 2	16.2	19.3	0.4	7485	18.55
Granular backfill	17.5	19.2	0.3	4329	45.6

5.1.4. Geotechnical properties data below excavation level

The main units that are found below the excavation level are silty clay with sand below and course to gravelly sand with intermittent silty and clayey soil. These units could have effects on the settlement of the dam. The following SPT - N values were used to calculate elastic settlement and settlement due to earthquake loading. In addition to this it was also used to determine elastic modulus of the soil.

Table 5.13 Standard Penetration Value of foundation Units

BH-ID	**Test depth (m)**	**SPT value-N**	**Soil type**	N_{cor}
GIBH-2	12.02-12.047	12	Clay silt with sand	9
	19.21-19.66	8	Silt clay	5
GIBH-3	11.50-11.65	17	Gravelly sand	13
	14.5-14.95	15	Sandy loam with gravel	10
	17.0-17.45	8	Clayey silt sand	5
	20.0-20.45	13	Gravelly silt sand	8
	22.54-22.25	23	Silty sand with gravel	13
GIBH-4	8.30-8.75	17	Gravelly sand	14
	11.90-12.35	11	Gravel associated with clayey sand	8
	22.67-23.12	34	Sandy gravel	19

From the above table the corrected N values shows that the soil with clayey materials are below 10 but for the sandy to gravelly sand the values of N_{cor} are greater than 10. For the sandy to gravely sand part of the foundation the average N_{cor} values of granular type of soils was used and it is 14 and N_{60}=16.

Some properties of silty clay with sand unit are summarized in chapter two in table 2.2 and for the sand to gravelly sand (coarse alluvium) the properties were taken from SPT Value by using empirical relations (eq.2.7) and Elastic modulus of the silty clay layer were taken assumption. The following table shows the average values of parameter which were used during the analysis.

Table 5.14 Summary of values of parameters of the foundation below the excavation level

Soil type	e_o	C_c	$\gamma_{sat}\ {kN}/{m^3}$	E_s	M
Silt clay with sand				5500	
GIBH2-CBS1	1.5	0.399	15.11		16.03
GIBH2-CBS4	1.3	0.33	16.1		15.6
Sand to gravely sand			18	12256	

5.2. Effective Stress distribution within the foundation

The foundation units were divided into layers of 3 m thickness to increase the effectiveness of the settlement analysis especially for consolidation analysis. The following table (table 5.14) shows the average initial effective stress at the middle of foundation layers which calculated as discussed in chapter four.

Table 5.15 the average initial effective stress at the middle of the layer in ${kN}/{m^2}$ at chainge 0+115 and 0+135

	Chainge 0+115				Chainge 0+135			
Layers	Below Shell		Below core clay		Below Shell		Below core clay	
	Depth m	σ_o	Depth m	σ_o	Depth	σ_o	Depth	
1	0-3	22.8	0-3	21.9	0-3	22.8	0-3	21.9
2	3-6	68.4	3-6	65.7	3-6	68.4	3-6	65.7
3	6-9.4	96	6-9	94.73	6-9.8	97.4	6-9	94.73
4	9.4-12.4	117.2	9-13.4	127	9.8-13	121	9-13.2	126
5	12.4-15.4	134.6	13.4-16.4	154.7				
6	15.4-19.85	156.72	16.4-19.85	173.4				

Table 5.16 the average initial effective stress at the middle of the layer in ${kN}/{m^2}$ at chainge 0+235 and 0+250

	Chainge +235				Chainge +250			
Layers	Below Shell		Below core clay		Below Shell		Below core clay	
	Depth m	σ_o	Depth m	σ_o	Depth	σ_o	Depth	
1	0-3	24.3	0-3	21.9	0-3	26.25	0-3	21.9
2	3-6.5	77	3-6	65.7	3-6.5	83	3-6	65.7
3	6.5-9.5		6-9	94.73	6.5-10		6-9	94.73
4	9.5-12.5		9-11	116.5	10-13.4		9-11.2	115

The analysis was carried out in two stages by considering two fill level for the outlet conduit portion. For first stage the fill was considered at level of 13 m during the construction which is the settlement of the outlet conduit detected. The second level was considered at the dam crest level and it was used in the analysis other sections (Chainge 0+115, 0+235 and 0+250). To determine the vertical stress due to the embankment loading eq. 4.51 and eq.4.52 were used. So the average change of vertical stress due to the embankment fill is presented in the following table 5.17 and table 5.18 at different section.

Table 5.17 The average change of vertical stress of the dam foundation at chainge of 0+115and 0+135

	Chainge 0+115				Chainge 0+135					
Layers	Below Shell		Below core clay		Below Shell			Below core clay		
	Depth m	σ_z	Depth m	σ_z	Depth	σ_z	σ_{z1}	Depth	σ_z	σ_{z1}
1	0-3	181	0-3	334	0-3	170	95	0-3	340	189
2	3-6	181	3-6	327	3-6	170	95	3-6	333	185
3	6-9.4	177	6-9	321	6-9.8	167	93	6-9	326	185
4	9.4-12.4	173	9-13.4	314	9.8-13	163	91	9-13.2	320	182
5	12.4-15.4	173	13.4-16.4	307	13-31	160	89	13.2-31	286	159
6	15.4-19.9	169	16.4-19.9	294						
7	19.9-27.9	168	19.9-27.9	281						

σ_z= the stress at full height the dam and σ_{z1}= the stress during the construction at 13 m height level.

Table 5.17 the average change of vertical stress of the dam foundation at chainge of 0+235 and 0+255

	Chainge +235				Chainge +250			
Layers	Below Shell		Below core clay		Below Shell		Below core clay	
	Depth m	σ_z	Depth m	σ_z	Depth	σ_z	Depth	
1	0-3	181	0-3	334	0-3	185	0-3	341
2	3-6.5	181	3-6	327	3-6.5	185	3-6	334.2
3	6.5- 15.5	177	6-9	321	6.5-14.4	182	6-9	327.4
4			9-11	314			9-11.2	321
5			11-15.5	307			11.2-14.4	314

The change of vertical stress due to the conduit and embankment fill at chainge 0+135 was determined by averaging the vertical stress at the center of the conduit crown and the two sides of the conduit. The unit weight of concrete is 23 KN/m^3.

5.3. Elastic settlement analysis

The elastic settlement of the backfill materials of the dam foundation and the compressible layer at the selected four sections are calculated from eq. 2.1 and as discussed in chapter 4. The results are mentioned in the following table.

Table 5.19 Predicted immediate settlement of the backfill materials of the foundation in meter

Material	**Z1 in m**	**q (KN/m^2)**	**I_t**	**S_i**
0+115				
Below the Clay core	15	307	0.88	0.35
Below the shall	15	173	1	0.27
0+135				
Below the Clay core	6	333, 185	0.88	0.36, 0.21
Below the shall	6	170, 95	1	0.25,0.15
0+235				
Clay cutoff	5.5	327	0.88	0.38
Alluvium backfill	3.25	181	1	0.2
0+250				
Clay cutoff	5.6	334	0.88	0.38
Alluvium backfill	3.25	185	1	0.39

The immediate settlement was estimated at the middle of the backfill layer thickness (Z=2Z1). The net pressure at corner in the middle layer was determined by eq. 4.2 and change of the stress due to the embankment load used from table 5.17 and 5.18.

There are two q and S_i at chainge 0+135 and they represents the embankment at full length and at construction stage (13 m of height) which the level where settlement is happened. respectively. The estimated value of the immediate settlement for the granular fill at chainge 0+250 is over estimated because of the elastic modulus was estimated from the M_v.

5.4. Elastic settlement from SPT value

Efforts were made to determined elastic settlement of the coarse grained alluvium deposit which calls it gravelly sand in this study. SPT test was found the best way to determine the elastic settlement for this part. The net pressure at the center of in the middle layer was determined on table 5.17 and table 5.8. The width of the foundation was considered as 10 m for each of foundation below the clay cutoff and alluvium backfill since the fill of the dam was made stage by stage.

Table 5.20 Elastic settlement of the gravelly sand part of the foundation from SPT value-N

Chainge	Z in m	q (KN/m^2)	Se(mm)
0+115			
Below alluvium	24	168	59.4
Below cutoff	24	281	99.5
0+135			
Below alluvium	22	168, 89	59.4, 31.5
Below cutoff	22	281, 159	99.5, 56.3
0+235			
Below alluvium	13	177	62.6
Below cutoff	13	307	108.7
0+255			
Below alluvium	13	182	64.4
Below cutoff	13	314	111.2

The elastic settlement of the sand to gravelly sand from SPT value-N_{60} shows the small when its compare its thickness due to the depth from the foundation.

5.5 Analysis by Janbu approach

Janbu approach is one of the settlement analysis methods which were used during this analysis. As discussed on chapter four this analysis used initial average effective vertical stress (table 5.15 and 5.16), average change vertical stress (table 5.17 and table 5.18) and the modulus number (table 5.12 and table 5.14). The values of predicted settlement from this approach are listed in the following tables (table 5.21 to table 5.24 for each section of the dam.

Table 5.21 Predicted settlement of the foundation by using Janbu's approach at the chainge 0+115

Layer s	At the center of the dam (below clay core)			Below the shall		
	H_o(m)	ε	$S=\varepsilon*H_o$	H_o(m)	ε	$S=\varepsilon*H_o$(m)
1	3	0.11	0.33	3	0.08	0.24
2	3	0.07	0.21	3	0.05	0.15
3	3	0.06	0.18	3.4	0.04	0.13
4	4.4	0.05	0.22	3	0.06	0.18
5	3	0.07	0.21	3	0.052	0.16
6	3.45	0.06	0.21	4.45	0.046	0.21
Total	19.85		1.15	19.85		1.07

The predicted settlement amount by using Junbu approach for the clay cutoff is 0.94 m and for the compressible silty clay layer below the cutoff is 0.42 m. The expected settlement in the alluvium backfill and compressible layer below the alluvium backfill are 0.52 m and 0.55 m, respectively.

Table 5.22 Predicted settlement of the foundation by using Janbu's approach at the chainge 0+135

Layer s	**At the center of the dam (below clay core)**			**Below the shall**		
	H_o(m)	**ε**	**$S=\varepsilon*H_o$**	**H_o(m)**	**ε**	**$S=\varepsilon*H_o$(m)**
1	3	0.11	0.33	3	0.08	0.24
2	3	0.07	0.21	3	0.05	0.15
3	3	0.06	0.18	3.8	0.04	0.15
4	4.2	0.05	0.21	3.2	0.054	0.17
Total	**13.2**		**0.93**	**13**		**0.71**
	At the level of 13 m					
1	3	0.09	0.27	3	0.06	0.18
2	3	0.053	0.16	3	0.033	0.1
3	3	0.042	0.13	3.8	0.025	0.1
4	4.2	0.035	0.15	3.2	0.035	0.11
Total	**13.2**		**0.71**	**13**		**0.49**

The expected settlement within the clay cutoff fill material at the two fill stage is 0.9 m for full height of the dam and 0.71 m at the level of the 13 m height of the dam. The predicted settlements for the alluvium fill are 0.54 m and 0.38 m for full height of the dam and at the level of the 13 m height of the dam respectively. The compressible layer below the alluvium fill is expected to experience 0.17 m and 0.11 m of settlement within the two fill level.

Table 5.23 Predicted settlement of the foundation by using Janbu's approach at the chainge 0+235

Layer s	**At the center of the dam (below clay core)**			**Below the shall**		
	H_o(m)	**ε**	**$S=\varepsilon*H_o$**	**H_o(m)**	**ε**	**$S=\varepsilon*H_o$(m)**
1	3	0.11	0.33	3	0.12	0.36
2	3	0.07	0.21	3.5	0.065	0.23
3	3	0.06	0.18			
4	2	0.05	0.1			
Total	**11**		**0.82**	**6.5**		**0.59**

The predicted settlement at chainge 0+235 is only for the clay cutoff material and the alluvium backfill from borrow near to the camp. Below those materials there is no compressible silty clay and gravelly sand is available. The estimated settlements in this section for the clay cutoff and alluvium backfill materials are 0.82 m and 0.59 m respectively.

Table 5.24 Predicted settlement of the foundation by using Janbu's approach at the chainge 0+250

Layers	At the center of the dam (below clay core)			Below the shall		
	H_o(m)	ε	$S=\varepsilon*H_o$	H_o(m)	ε	$S=\varepsilon*H_o$(m)
1	3	0.107	0.321	3	0.058	0.174
2	3	0.069	0.207	3.5	0.036	0.125
3	3	0.057	0.171			
4	2.2	0.052	0.115			
Total	**11.2**		**0.814**	**6.5**		**0.299**

The predicted settlement amount of the granular soil backfill is 0.3 m and for the clay cutoff is 0.81 m.

5.6. Conventional settlement analysis (one dimensional Method)

The primary settlement of the foundation units mainly the alluvium backfill, clay cutoff fill and the compressible clay were determined by using eq. 2 and the processes was discussed in chapter four. The units were divided in to 3m thickness layers similar to initial average effective stress as presented in table 5.15 and 5.16. The main input parameters are presented on table 5.10. The following tables show the results of consolidation from conventional settlement analysis.

Table 5.25 Primary settlement of the foundation in meter at the chainge 0+115

Layers	At the center of the dam (below clay core)		Below the shell	
	H_o(m)	1-D-method	H_o(m)	1-D-method
1	3	0.32	3	0.24
2	3	0.21	3	0.15
3	3	0.18	3.4	0.13
4	4.4	0.22	3	0.18
5	3	0.22	3	0.17
6	3.45	0.23	4.45	0.22
Total	**19.85**	**1.38**	**19.85**	**1.09**

The depth of clay cutoff is up to depth of 13.4 m and the predicted total settlement amount is 0.93 m. For the alluvium backfill material the predicted total settlement is 0.52 m. The predicted settlement of the compressible silty clay layer below the excavation level is 0.45 m at the center of the dam with thickness of 6.45 m and 0.57 m below the shall with thickness of 10.45 m.

Table 5.26 Primary settlement of the foundation in meter at the chainge 0+135

Layers	**At the center of the dam (below clay core)**		**Below the shall**	
	H_o(m)	**1-D-method**	**H_o(m)**	**1-D-method**
1	3	0.33	3	0.24
2	3	0.21	3	0.14
3	3	0.18	3.8	0.14
4	4.2	0.21	3.2	0.18
Total	**13.2**	**0.93**	**13**	**0.7**
	At the level of 13 m			
1	3	0.27	3	0.18
2	3	0.16	3	0.1
3	3	0.13	3.8	0.1
4	4.2	0.15	3.2	0.12
Total	**13.2**	**0.71**	**13**	**0.5**

The clay cutoff at the center was filled to the depth of gravelly sand i.e. the compressible silty clay layer was excavated and removed. However, in the sides of the dam axis toward the upstream and downstream the compressible soil was not fully removed as per the recommendation. The settlement was predicated in two stages: at full height of the dam fill (crest level) and at the level during the settlement was detected (13 m height of the dam).

The predicated settlement at the center of the conduit below the clay core fill is 0.93 m at crest level and 0.71 m at 13 m height of the fill. For the alluvium back fill part the analysis was carried out to below shall material and the results are 0.52 and 0.38 at full height of the embankment and at 13 m height of the dam respectively. The thickness of compressible silty clay layer beneath the alluvium backfill is 3.2 m and the predicted settlement amounts are 0.18 m and 0.12 m at full height of the embankment and at height of 13 m respectively (table 5.26).

Table 5.27 Primary settlement of the foundation in meter at the chainge 0+235

Layers	**At the center of the dam (below clay core)**		**Below the shall**	
	H_o(m)	**1-D-method**	**H_o(m)**	**1-D-method**
1	3	0.33	3	0.35
2	3	0.21	3.5	0.23
3	3	0.18		
4	2	0.11		
Total	**11**	**0.83**	**6.5**	**0.58**

The alluvium backfill of this section (at chainge 0+235) is not similar as that of above section. It is filled from another borrow area which is located near to the camp and its thickness is relatively less. The predicted settlement for this section of the dam is 0.83 for the clay cutoff; for the alluvium-2 backfill is 0.58 m expected (table 5.27).

Table 5.28 Primary settlement of the foundation in meter at the chainge 0+235

Layers	**At the center of the dam (below clay core)**		**Below the shall**	
	H_o(m)	**1-D-method**	**H_o(m)**	**1-D-method**
1	3	0.33	3	0.15
2	3	0.21	3.5	0.11
3	3	0.18		
4	2.2	0.12		
Total	**11.2**	**0.84**	**6.5**	**0.26**

The predicted primary settlement of the granular backfill material is 0.25 m it is small by half from the alluvium backfills. However the settlement on the clay cutoff is similar to other part of the dam.

5.7 Time Rate of Consolidation

This analysis was done for the primary settlement from the one dimensional method to determine how much settlement amount expected at different period of time and how much time will take to reach 90 % of the settlement. The degree of consolidation was taken from chart (annexure 1) by related to calculated Tv value. By using eq. 2.14 and the about its detail was discussed on chapter 4. The following table shows time rate of consolidation of the dam.

Table 5.29 time rate of consolidation of the dam foundation at different sections

Time	**0+115**		**0+135**		**0+235**		**0+250**	
	Alluvium	**Clay cutoff**	**Alluvium**	**Clay cutoff**	**Alluvium -2**	**Clay cutoff**	**Granular backfill**	**Clay cutoff**
During construction (after 0.5 year)	0.1	0.09	0.11	0.1	0.22	0.1	0.07	0.1
After 5 years	0.32	0.27	0.28	0.294		0.29	0.21	0.292
After 10 years	0.411	0.36	0.413	0.392		0.44		0.43
After 20 years		0.53		0.570		0.63		0.64
Time for 90 %	7.2 years	46.6 years	10 years	45.2 years	3 years	31.4 years	1 years	32.5 years

As it is tried to explain at the table (table 5.29) clay cutoff due to the thickness of the layer about 13 m it takes longer time (46.6 years at chainge of 0+115) whereas, Alluvium backfill at chainge 0+250 expected to be faster rate time of consolidation relative to others. It's believed because of the soil is granular soil type.

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CHAPTER -6 RESULT, INTERPRETATION AND DISCUSSION

6.1. Potential Settlement of the Dam

Gidabo dam was constructed on soil with 40 m thickness therefore it is expected that settlement may surely take place during the construction and life time of the dam. During the present study, the dam foundation was characterized in to three layers according to the physical properties of the soil. These are; backfill material (Alluvium backfill and clay cutoff fill), compressible silty clay layer and the granular unit (sand to gravelly sandy) with silty clay layer.

The backfill material along the dam axis was filled from three different types of soil. The first section from chainge 0+046 to 0+215 was filled with CH or OH type of soil, the second section from chainge 0+215 to 0+245 was filled with CL or OL type of soil and the third section was filled with gravelly soil type. Further, clay soil, which was used as a cutoff and the clay used in the core material can be grouped as CH or OH type soil.

The compressible silty clay or clayey silt with sand was found below the excavation level at chainge 0+115 and 0+135. Based on the results of the sieve analysis this soil can be grouped as CH or OH and CL or OL type of soil (table 2.3). The foundation of the dam below the compressible soil on the first section and below the backfill foundation at the other sections was granular type of soil dominates sand type (silt sand, clayey sand and gravelly types of soil) with intermittent layers of silty clay and clayey soil.

Table 6.1 General properties of soils (Arora, 2004)

Soil Group	Permeability	Compressibility	Shear strength
CL	Pervious	Medium	Fair
OL	Semi-pervious	Medium	Fair
CH	Impervious	High	Poor
OH	Impervious	High	Poor
Coarse-Grained (GP-GM,SW-SC)		Very low to low	Excellent to good

According to Arora (2004) the soils of the foundation units possess medium to high compressibility behavior. However, the coarse-grained soils may possess very low to low compressibility.

In the present study efforts were made to predict the potential settlement on the dam at four selected sections of the dam. The potential settlement of the dam at the outlet conduit section was made at two levels of fill. These were taken at 13 m height of the embankment fill and at full height of the crest. The total predicted potential settlement of the dam at four sections is presents in the following table 6.2.

Table 6.2 The total predicted potential settlement of the dam along the sections

Section/chainge	Immediate settlement	Primary settlement		Total settlement
		1-D	Janbu's	
0+115				
Up/downstream	0.33	1.09	1.07	1.42
Central part	0.45	1.38	1.15	1.83
0+135				
Up/downstream	0.31, 0.18	0.7, 0.5	0.71, 0.49	1.01, 0.68
Central part	0.46, 0.266	0.93, 0.71	0.93, 0.71	1.39, 0.98
0+235				
Up/downstream	0.26	0.58	0.82	0.84
Central part	0.48	0.83	0.59	1.31
0+250				
Up/downstream	0.49	0.26	0.299	0.75
Central part	0.45	0.84	0.82	1.29

The estimated values of immediate settlement were summation of elastic settlement of the upper cohesive backfill materials, elastic settlement of compressible silty clay layer and the elastic settlement of granular alluvium (sand to gravelly sand) unit below the excavation level which was calculated from SPT-N value. The estimated total settlement is the summation of the primary settlement from one dimensional (conventional) method and the immediate settlement. Janbu's approach results were found to be in quite agreement with the conventional one.

The results showed that maximum potential settlement is on the first section (chainge +115) at the up/downstream part of the dam. This includes the alluvium backfill, compressible layer and the granular soil below the excavation level. In this section about 1.42 m of settlement is expected and about 0.43 m of the estimated settlement is believed, which is already happened during the construction of the dam. Out of total estimated settlement, 54% of the estimated settlement contributed by the alluvium backfill, 42 % is from compressible layer and 4 % is expected from the granular soil type below the excavation level. However, the settlement contributions of each layer for the central part of the embankment section are different from the above. The central part of the foundation contains clay cutoff, compressible silt clay and granular soil below the excavation level and the percentage of settlement is 60 %, 35 % and 5 %, respectively. Due to the excavation for the cutoff of the dam in the foundation area some part of the compressible silty clay layer was excavated. Therefore, it is expected much more settlement on the clay cutoff part of the foundation.

The potential settlement of the dam at chainge 0+135, which is along the outlet conduit of the dam, was determined for two stress conditions. First, for the full height of the embankment this was used for the future potential settlement estimation of this section. The second, used to determine the cause of partial settlement along the conduit. It was used for the determined of the causes of the partial settlement of the section. The estimated potential settlement along this section (chainge 0+235) is less than the estimated settlement on first section (chainge 0+115). This is possibly because of the fact that most of the compressible silty clay layer was excavated except about 3.2 m below the alluvium fill and about 0.25 m below the clay cutoff fill which is considered as clay cutoff during the analysis. However, significant amount of settlement was estimated because of the considerable thickness of the fine material backfilled. About 0.5 m settlement already happened during the construction and about 0.6 m and 0.83 m amount of settlement is expected in the future at up/downstream and central part of the outlet conduit, respectively.

The alluvium backfill material which was used for dam foundation from chainge 0+215 up to 0+245 is different from the backfill material that was used initially. The estimated potential settlement of this type of alluvium backfill was done at chainge 0+235 and the expected settlement amount of the up/down stream and central part of the dam are almost comparable except the time and the rate of consolidation (Table 5.29). In this regard the settlement in alluvium is estimated to be faster as compared to the clay cutoff section.

The last section was from the chainge 0+245 up to +0+255 and at end of this section the construction was stopped during the course of the present study as the diversion canal was in the next section and preparation of the outlet conduit was not finished. The foundation of this section was filled by granular type of backfill. The settlement analysis at this section showed small amount of primary settlement and large amount of elastic settlement as compared to other type of backfills. The estimated elastic settlement could be over due to under estimated elastic modulus. Even if the predicted elastic settlement amount is large, it does not affect the performance of the dam. This is because, it is believed to happen during the construction and such amount of fill was filled to attain the full height of the dam.

The time and the rate of consolidation of the backfilled foundation units were analyzed and the results are presented in Table 5.27. The time and the rate of the consolidation for alluvium backfill for the first two sections (0+115 and 0+135) and the clay cutoff portion throughout the dam axis during the construction, (which is taken as 6 months), estimated to be about 0.1 m, which has already occurred. The clay cutoff at chainge 0+115 takes the longest time about 46.6 years because of its thickness. The alluvium backfill at chainge +235 is fastest about 1 year to attain the 90 % of estimated primary settlement.

In the present study, the estimated rate of the consolidation did not include the rate of the compressible silty clay layer which is present below the backfill units at chainge 0+115 and 0+135. Also, the secondary consolidation settlement was not included for all units in the present study. Thus, the settlement amount as per the time is more than the calculated amount. In addition to this the consolidation time rate at upper 3-6 m is believed to have faster rate than that of the sections lying below it. Further, all assumptions of the theory of time rate consolidations must be kept in mind while considering variation in the estimated and the actual settlement.

According to WWDSE, (2009) an allowance of 1 to 2% of the dam height should be provided for settlement in the foundation and the embankment. Besides, an allowance of about 1% of the dam height should be provided for settlement due to strong ground motion. For Gidabo dam, a total settlement allowance of 3% (0.75 m) of the dam height has been provided. Total settlement allowance of dam during the design was smaller by half with estimated settlement for foundation at chainge 0+115 and 0+135. It's less than the other sections estimated values.

Differential settlement is relative movement between neighboring points or sections with in an embankment dam, or in the foundation zones. Differential settlement that exceeds about 1 foot per 100 feet (measured longitudinally along the embankment dam) is critical. Settlement that exceeds this limit of acceptable strain can lead to concern for hydraulic fracture (FEMA, 2005). Differential settlement among the sections especially for those along alluvium backfill has changed in between chainge 0+235 and 0+250. Differential settlement between the central part (clay cutoff) and the up/downstream (backfill) section is expected in section from chainge 0+245 to the right abutment.

According to Sherard (1973) the differential settlement around the conduit may cause cracks in the soil, leading to the piping failure. It is recommended in cases where the soil foundation is thick and compressible and it is not desirable to excavate trench under conduit and fill it with compacted earth fill. However, in Gidabo dam along conduit sections not only excavation was made but around 3.2 m compressible silty clay layer at up/downstream on the foundation was also left unexcavated, which by all means is the worst scenario, as far as settlement is concerned. Differential settlement is also expected in between the fine backfill material and the clay cutoff. Also it is expected within the materials itself, even if it is well compacted.

The expected differential settlement is not only within the outlet conduit itself, but also along the joints of the Intake tower and the outlet conduit. Because of the Intake tower of Gidabo dam was constructed on pile foundation. This means less or minor settlement is expected and the outlet conduit which is constructed directly on the fine backfill material on which more settlement is expected.

6.3. Comparison between the predicted and observed settlement

Settlement measurement was taken along the conduit after it has been recognized during the construction time and the measured results are presented in table 2.3. The comparison of the actual settlement and the estimated results are useful in identifying the possible causes of the settlement and also to validate the results of the present study.

The settlement was measured at different section, along the outlet conduit every ten meters and for the purpose the comparison a chainge 0+19.15 to 0+49.37 (central part of the conduit) was selected. This section was selected because it represents up/downstream and the central part of the dam, the cross section, the excavation level and the height of the embankment. The actual measured settlements on these sections are about 0.40m.

Analysis were made to estimate the settlement amount of the outlet conduit foundation at the embankment fill level, which the settlement recognized (13 m of fill height), and the expected settlements due to the stress at upstream and central part of the dam foundation are 0.68 m and 0.9 respectively. The actual measured settlement was found within range of estimated value.

From elastic settlement of the foundation at that level expected 45 % of the actual settlement at the up/down stream and the rest about 55 % it’s expected from primary consolidation settlement. Predicted primary consolidation of the alluvium backfill is 0.11 m which is about 28 % of the actual settlement during the construction time (half year) but it is expected more due to the dewatering process on the right side of the conduit after the whole conduit work is complete. The time rate of consolidation for the compressible silty clay was not determined because of the absence of data coefficient of compressibility. Thus dewatering processes effect and the primary settlement of compressible clay the rest of roughly 25 % of the actual settlement.

The settlement analysis of central part of the dam shows nearly 65 % of the actual settlement was from the elastic and the rest of 35 % is from primary settlement of the clay cutoff. However the primary consolidation settlement during the construction time is expected about 25 % of the observed settlement and due to the dewatering processes at the right side of the conduit. In addition to that about 0.20 m to 1 m of compressible silty clay was below the cutoff which was excluded from the analysis and the effect of this layer could play some roles.

6.4. Causes of the settlement

The main causes of the settlement of outlet conduit were determined from the above discussions those are:

- ✓ The backfill material including the alluvium backfill and the clay cutoff
- ✓ Compressible silty clay layer (organic) below the excavation
- ✓ The excavation method (dewatering process)
- ✓ Granular soil foundation below the excavation level

The alluvium backfill material on the up/down stream is compressible type of soils and the estimated settlement result shows its play major roles on the settlement. The construction of conduits on the clay cutoff is not the worst idea and its preferable than on piles. According to FEMA (2005) use of piles is not recommended, because of the conduit may become undermine, allowing uncontrolled seepage to occur under it. However, due to fine material used in the back fill on the up/downstream it can carry much more loads which is to mean no stress distribution to the alluvium backfill through the conduit. Consequently, excessive settlement within the clay cutoff is obvious. Further, the result showed contribution of the backfill materials to the settlement, which is expected to be about 50 % at normal condition.

Compressible silty clay soil is with some organic material which is expected to involve in settlement. The result showed 0.26 m settlement from the elastic and primary settlement but during the construction not more than 0.1 m probable. It covers nearly 22 % of the settlement happened is within this soil. The settlement from granular soil foundation below this layer was estimated from SPT value-N about 10 % of the settlement happened during the construction.

Out of the total, 18 % of the settlement is believed to be due to the dewatering process on the right side of the conduit work. The dam construction was done in different sections along the dam axis. The excavation level of the foundation near to the dam conduit was shown it is below the ground water so excessive dewatering is expected and dewatering process was done during the construction. The process of dewatering facilitate the settlement by increasing effective stress on the foundation and it allow as free drainage for the clay cutoff and backfill material which could fastest the estimated consolidation settlement time rate. In addition to this, during the excavation of next section there was not any sheet pile or other protective measurement except slope cut and erosion of fine sediments could be part of this settlement.

6.4. Validation of the Result

The result of the present study is for the anticipated condition rather than the conservative estimates because most of the data that were used are from different sources; secondary data from reports, average values considered for the parameters used, theoretical assumptions and empirical relationships. However, it does not make results to be overestimated as of the actual settlement and the estimated settlement is in agreement (above 90 %). In addition to this, the present study does not include the settlement from secondary consolidation, due to dynamic conditions and further settlement due to reservoir fill.

6.5. Possible Remedial Measurements

The result of the present of study showed excessive differential settlement within the outlet conduit and between the outlet conduit and the intake tower. Three options were identified to minimize the risk of conduit failure. The first two options are more focuses on the stress reduction.

Option-1 under pile, inserting piles below the outlet conduit at the entrance and exit portion of the conduit could transfer the some stress due to the load to the weathered rock or on the foundation below the excavation. However, the driving of piles could disturb the foundation soil and it needs detail design work on the installation process. To minimalize the disturbance of the foundation due to pile deriving micropiles or minipiles could be helpful.

Option-2, tying the conduit with the intake tower to transfer the stress to hard rock part through the piles on the intake tower foundation towards the entrance part of the conduit and insert piles on the exit portion. This option could work in theory but it is difficult to find tying instrument without movement especially during the water passes through the conduit. In addition to this, it could lead to seepage if the foundation soils settle and the conduit hang on the intake tower.

Option-3 the first two options seems a risky work beside their disadvantage no one knows how much they could improve. Option-3 is constructing of alternative conduit at the right side of the dam with proper investigation and improving technic. Third option seems more expensive and may possibly delay construction period time. However, this option is the only option which could be the permanent solution.

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CHAPTER -7 CONCLUSIONS AND RECOMMENDATION

7.1. Conclusion

The Gidabo dam is constructed on Gidabo River, which is located in Oromia Regional State, 377 Km from capital city of Ethiopia. The dam is an Earth fill dam with central clay core and the maximum height of the dam over the deepest river bed level is 23.8 m. The dam has a 335 m crest length and an 8 m top crest width with side spillway. It also has a central outlet conduit which will divert water towards the right into the lift canals off taking from the dam. Gidabo Dam has faced foundation settlement during the construction, along the outlet conduit which is about 0.4 m (table, 2.2). The present research study was aimed to investigate the causes of partial settlement of Gidabo Dam along the Conduit section and to investigate any further potential settlement of the dam foundation. In the present study, primary and secondary data were collected to identify the causes and to predict potential settlement of the dam foundation. During the investigation immediate and primary settlement of the dam foundation material were analyzed.

The analysis of the settlement was done in three steps. First, the dam was divided into four sections along the dam axis at chainge 0+115, 0+135, 0+235 and 0+250. The foundation units were identified from the dam cross section, borehole logs and from excavation level readings from the reports. The relevant properties of foundation units such as initial void ratio, grain size distribution, specific gravity, water content, and the consolidation and compressibility characteristics were processed and calculated from empirical relationships. In the second step, stress distribution of the soil profile was determined, for which soils were divided into sub layers. The settlement amount of the layers was determined from the soil parameters and the stress distribution by using Janbu’s elastic settlement theory and one dimensional settlement analysis. Besides, for coarse grained soil in the foundation settlement was assessed from the semi-empirical approach by utilizing static penetration test (SPT) data.

Three types of soil units were identified the first two dam sections (chainge 0+115, 0+135). These were the backfill materials at the top, the compressible silty clay at the middle and the granular alluvium material at the bottom of the foundation. For the rest of two sections the backfill materials at the top and granular alluvium unit at the bottom were identified. The alluvium backfill for the first section was classified as CH or OH type of soils as per Unified

Soil Classification System (USCS). The backfill material for the third section was OL or OH type of soil (USCS). Further, for the fourth section Gravelly type of backfill soil was considered.

The present study, showed potential settlement to be 1.42 m in upstream and downstream embankment section and 1.83 m in the center part of the foundation at section-1 (chainge 0+115). Predicted settlement at section-2 (chainge 0+135 along the conduit) is 1.1 m and 1.39 m in upstream and downstream embankment section and central part, respectively. Further, at section-4, 0.84 m and 1.31 m settlement is predicted for upstream and downstream embankment section and central part of the embankment, respectively. However, some of the settlement has already occurred during the construction. The present study results showed excessive settlement is expected. For Gidabo dam, a total settlement allowance of 3% (0.75 m) of the dam height has been provided. Total settlement allowance of dam during the design was smaller by half with estimated settlement for foundation at chainge 0+115 and 0+135. It’s less than the estimated values for the other sections.

The main causes of the settlement of outlet conduit, as per the present study are due to; (i) incompetent backfill material including the alluvium backfill and the clay cutoff, (ii) Compressible silty clay layer (organic) below the excavation and (iii) inappropriate excavation method (dewatering process). Further, the result showed contribution of the backfill materials to the settlement, which is expected to be about 50 % at normal condition. Nearly, 22 % of the settlement happened due to compressible silty clay layer (organic) below the excavation. Out of the total, 18 % of the settlement is due to the dewatering process on the right side of the conduit work. The dam construction was done in different sections along the dam axis. The settlement from granular soil foundation below this layer was estimated from SPT value-N about 10 % of the settlement happened during the construction.

The result of the present study is for the anticipated condition rather than the conservative estimates because most of the data that were used are from different sources; secondary data from reports, average values considered for the parameters used, theoretical assumptions and empirical relationships. However, it does not make results to be overestimated as of the actual settlement and the estimated settlement is in agreement (above 90 %).

7.2 Recommendation

From the review of the previous investigation reports and the investigations made during the present study it has been found that the Gidabo dam foundation has serious settlement problems especially along the outlet conduit. Thus, through the present study following recommendations are made;

The present study was done for the dam foundation under static conditions only. Therefore, it is recommended to conduct farther studies for dynamic settlement analysis.

In the present study, three options were identified as possible remedial measurement. However, further detailed investigations in terms of technical and financial feasibility are recommended.

From the present study further potential for differential settlement is predicted. Monitoring of such settlement through instrumentation may further help in managing the settlement problem, particularly for conduit section.

The present study has been conducted under the constraints of time, resources and financial support, therefore the results and the recommendations made through this study must be considered as indicative only. More elaborate systematic studies would be required before coming to any final decisions.

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Annexure

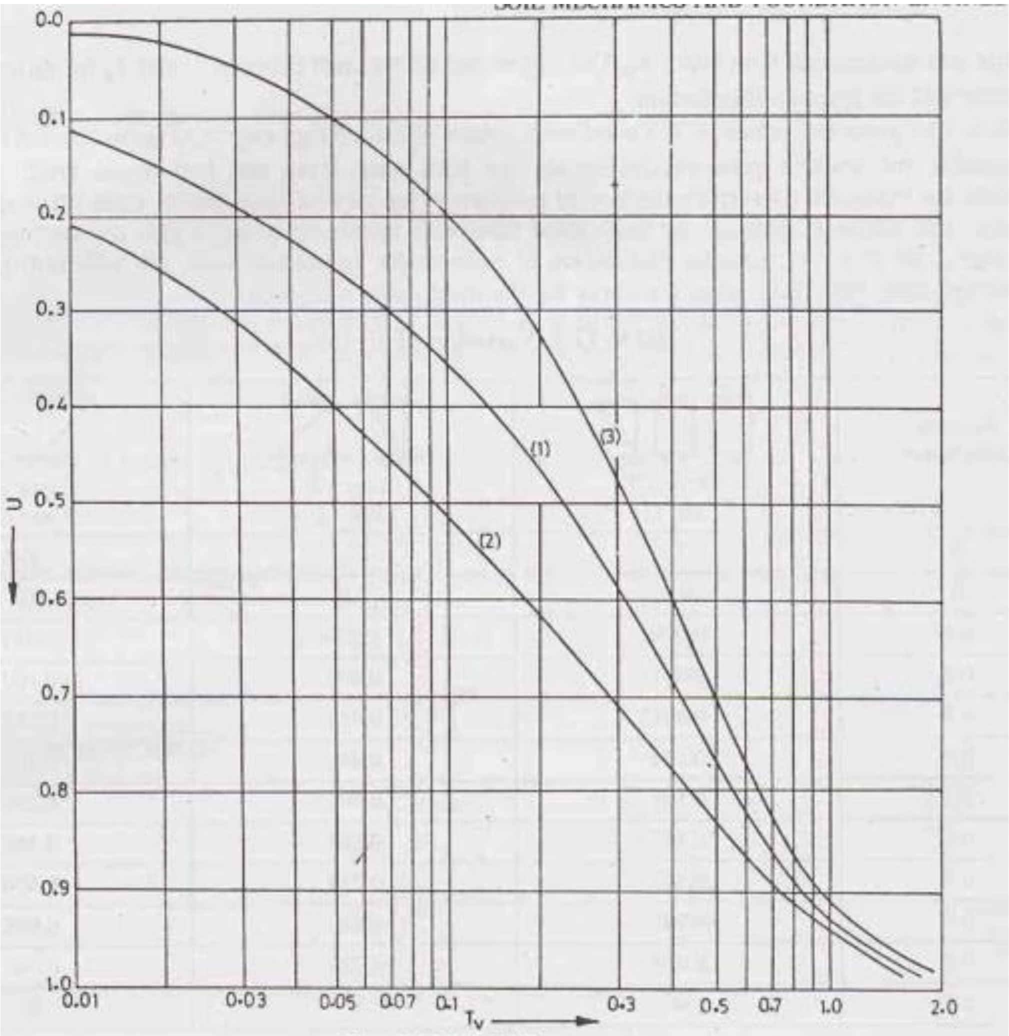
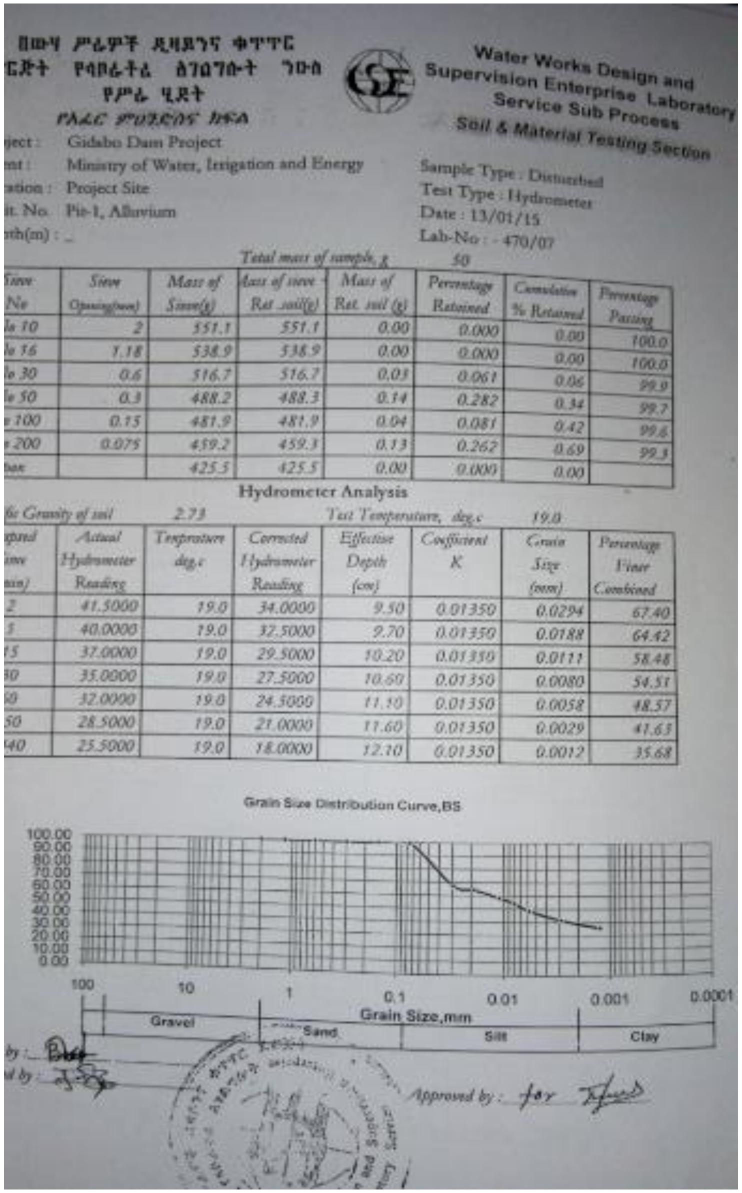


Fig. 12.13. *U—T_v* Curves.

Annexure I U-T_v curves



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የአፈር ምህንድስና ክፍል

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

ject : Gidabo Dam Project
nt : Ministry of Water, Irrigation and Energy
ation : Project Site
it. No. Pit-1, Alluvium
pth(m) : _

Sample Type : Disturbed
Test Type : Hydrometer
Date : 13/01/15
Lab-No : - 470/07

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
lo 10	2	551.1	551.1	0.00	0.000	0.00	100.0
lo 16	1.18	538.9	538.9	0.00	0.000	0.00	100.0
lo 30	0.6	516.7	516.7	0.03	0.061	0.06	99.9
lo 50	0.3	488.2	488.3	0.14	0.282	0.34	99.7
100	0.15	481.9	481.9	0.04	0.081	0.42	99.6
200	0.075	459.2	459.3	0.13	0.262	0.69	99.3
an		425.5	425.5	0.00	0.000	0.00	

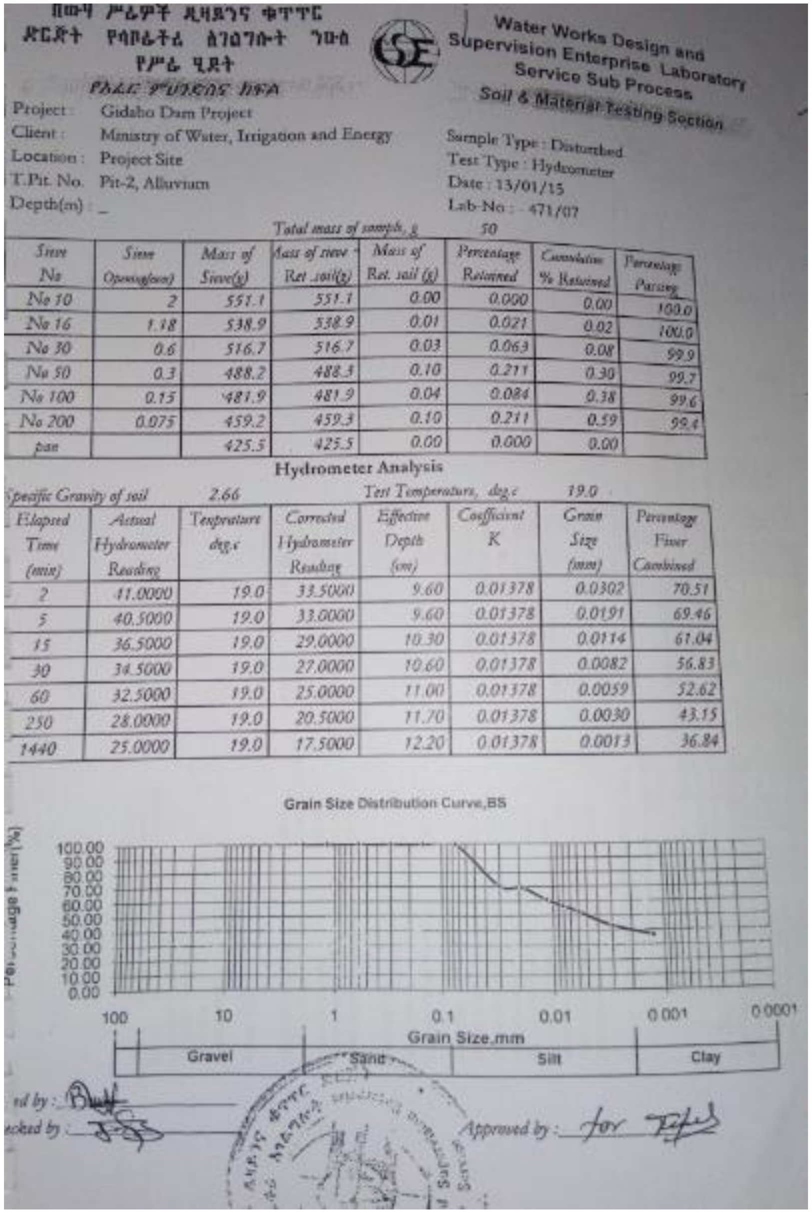
Hydrometer Analysis

fic Gravity of soil 2.73 Test Temperature, deg.c 19.0

psed ime nin)	Actual Hydrometer Reading	Temprature deg.c	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Percentage Finer Combined
2	41.5000	19.0	34.0000	9.50	0.01350	0.0294	67.40
5	40.0000	19.0	32.5000	9.70	0.01350	0.0188	64.42
15	37.0000	19.0	29.5000	10.20	0.01350	0.0111	58.48
30	35.0000	19.0	27.5000	10.60	0.01350	0.0080	54.51
0	32.0000	19.0	24.5000	11.10	0.01350	0.0058	48.57
50	28.5000	19.0	21.0000	11.60	0.01350	0.0029	41.63
40	25.5000	19.0	18.0000	12.10	0.01350	0.0012	35.68

Grain Size Distribution Curve,BS

by :
d by :
Approved by : for



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ድርጅት የላቦራቶሪ አገልግሎት ንዑስ
የሥራ ሂደት
የአፈር ምህንድስና ክፍል

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

Project : Gidabo Dam Project
Client : Ministry of Water, Irrigation and Energy
Location : Project Site
T.Pit. No. Pit-2, Alluvium
Depth(m) : _

Sample Type : Disturbed
Test Type : Hydrometer
Date : 13/01/15
Lab-No : - 471/07

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	538.9	0.01	0.021	0.02	100.0
No 30	0.6	516.7	516.7	0.03	0.063	0.08	99.9
No 50	0.3	488.2	488.3	0.10	0.211	0.30	99.7
No 100	0.15	481.9	481.9	0.04	0.084	0.38	99.6
No 200	0.075	459.2	459.3	0.10	0.211	0.59	99.4
pan		425.5	425.5	0.00	0.000	0.00	

Hydrometer Analysis

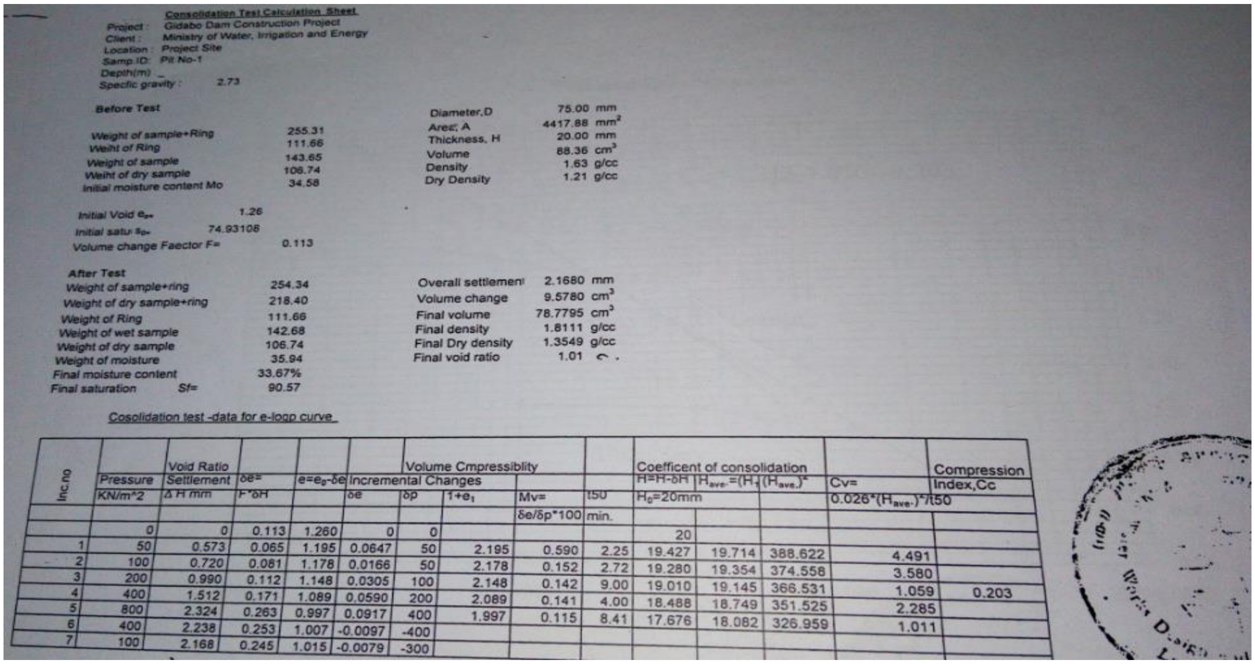
Specific Gravity of soil 2.66 Test Temperature, deg.c 19.0

Elapsed Time (min)	Actual Hydrometer Reading	Temprature deg.c	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Percentage Finer Combined
2	41.0000	19.0	33.5000	9.60	0.01378	0.0302	70.51
5	40.5000	19.0	33.0000	9.60	0.01378	0.0191	69.46
15	36.5000	19.0	29.0000	10.30	0.01378	0.0114	61.04
30	34.5000	19.0	27.0000	10.60	0.01378	0.0082	56.83
60	32.5000	19.0	25.0000	11.00	0.01378	0.0059	52.62
250	28.0000	19.0	20.5000	11.70	0.01378	0.0030	43.15
1440	25.0000	19.0	17.5000	12.20	0.01378	0.0013	36.84

Grain Size Distribution Curve,BS

ed by :
ecked by :
Approved by : for

Annexure II Hydrometer Analysis results from Alluvium Backfill for the conduit foundation



Consolidation Test Calculation Sheet

Project : Gidabo Dam Construction Project
Client : Ministry of Water, Irrigation and Energy
Location : Project Site
Samp ID: Pit No-1
Depth(m) _
Specfic gravity : 2.73

Before Test

Weight of sample+Ring	255.31
Weiht of Ring	111.66
Weight of sample	143.65
Weiht of dry sample	106.74
Initial moisture content Mo	34.58
Initial Void e_o=	1.26
Initial satu S_o=	74.93108
Volume change Faector F=	0.113

Diameter,D	75.00	mm
Area, A	4417.88	mm^2
Thickness, H	20.00	mm
Volume	88.36	cm^3
Density	1.63	g/cc
Dry Density	1.21	g/cc

After Test

Weight of sample+ring	254.34
Weight of dry sample+ring	218.40
Weight of Ring	111.66
Weight of wet sample	142.68
Weight of dry sample	106.74
Weight of moisture	35.94
Final moisture content	33.67%
Final saturation Sf=	90.57

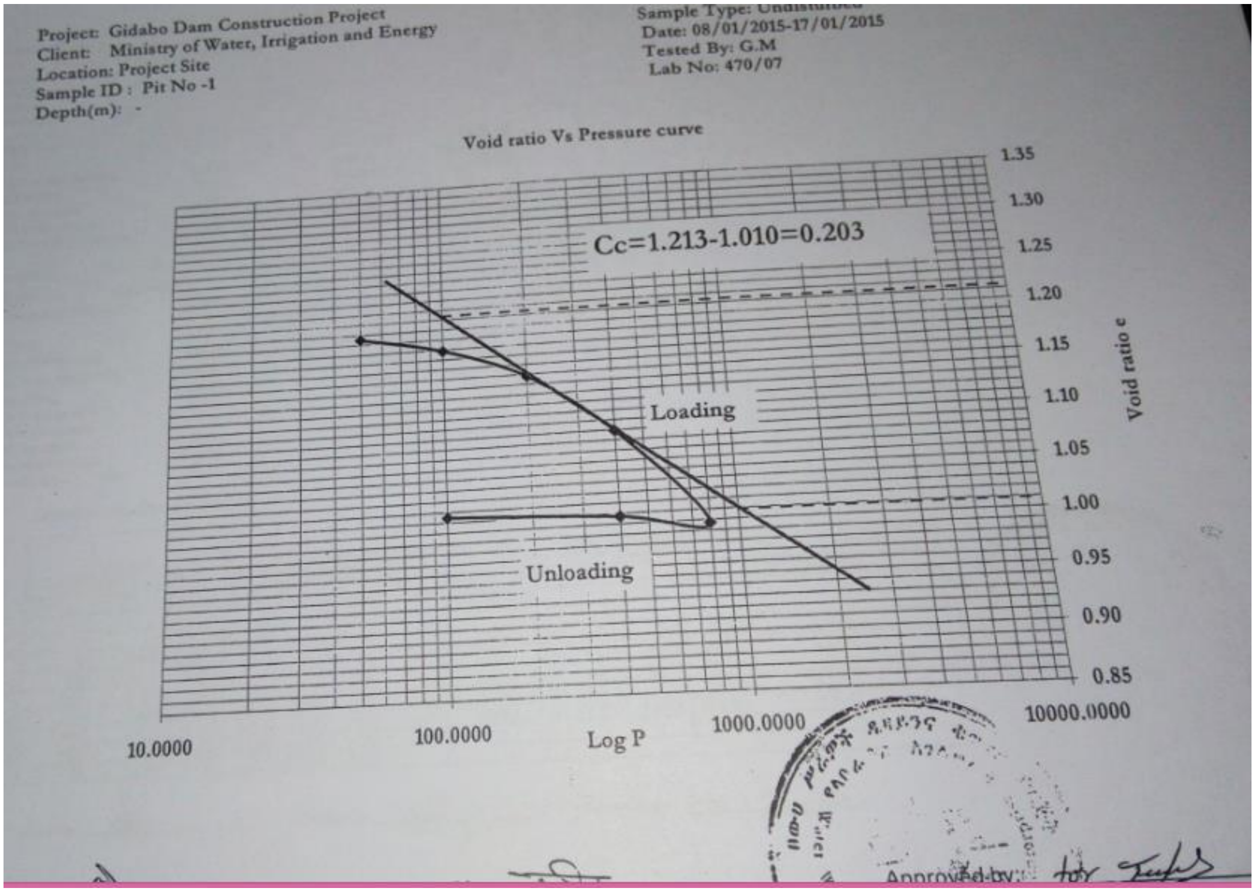
Overall settlement	2.1680	mm
Volume change	9.5780	cm^3
Final volume	78.7795	cm^3
Final density	1.8111	g/cc
Final Dry density	1.3549	g/cc
Final void ratio	1.01	

Cosolidation test -data for e-logp curve

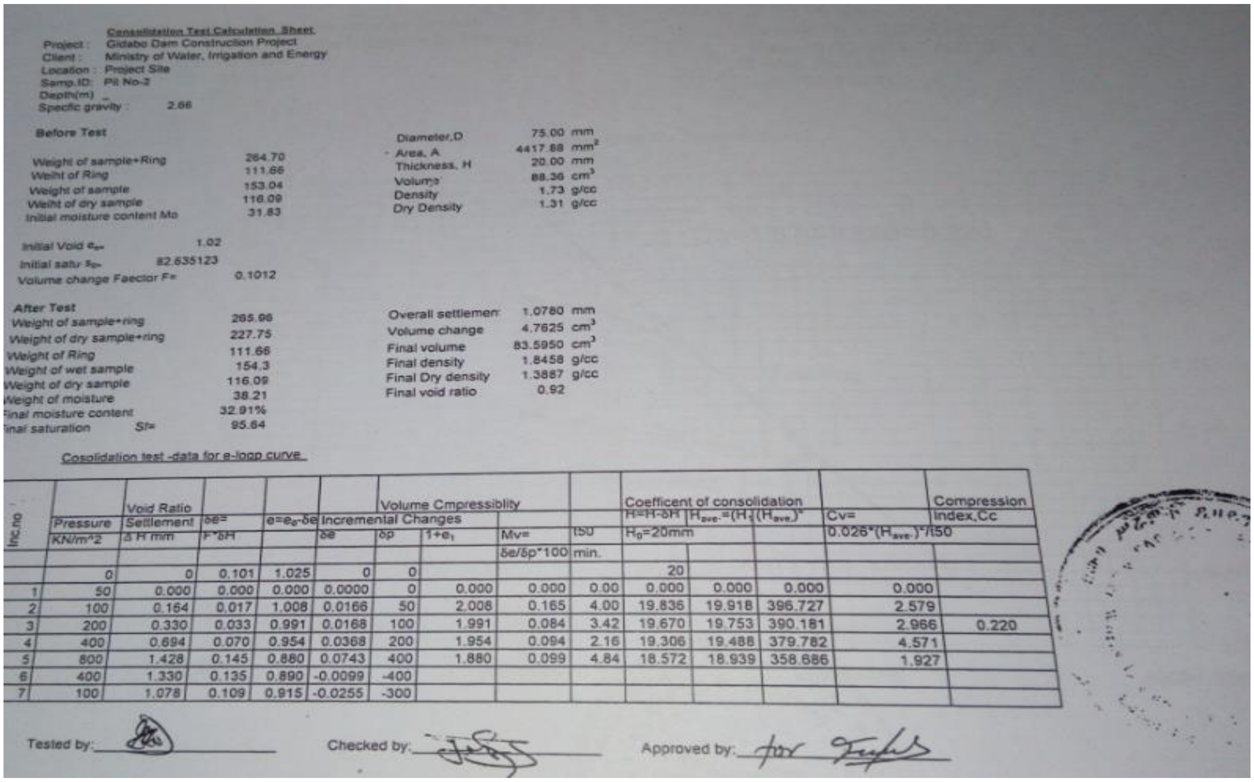
Inc.no	Pressure KN/m^2	Void Ratio Settlement ΔH mm	δe= F*δH	e=e₀-δe	Volume Cmpressiblity Incremental Changes δe	δp	1+e₁	Mv= δe/δp*100	t50 min.	Coefficent of consolidation H=H-δH H₀=20mm	H_{ave}=(H+	(H_{ave})²	Cv= 0.026*(H_{ave})²/t50	Compression Index,Cc
	0	0	0.113	1.260	0	0				20				
1	50	0.573	0.065	1.195	0.0647	50	2.195	0.590	2.25	19.427	19.714	388.622	4.491	
2	100	0.720	0.081	1.178	0.0166	50	2.178	0.152	2.72	19.280	19.354	374.558	3.580	
3	200	0.990	0.112	1.148	0.0305	100	2.148	0.142	9.00	19.010	19.145	366.531	1.059	0.203
4	400	1.512	0.171	1.089	0.0590	200	2.089	0.141	4.00	18.488	18.749	351.525	2.285	
5	800	2.324	0.263	0.997	0.0917	400	1.997	0.115	8.41	17.676	18.082	326.959	1.011	
6	400	2.238	0.253	1.007	-0.0097	-400								
7	100	2.168	0.245	1.015	-0.0079	-300								

Project: Gidabo Dam Construction Project
Client: Ministry of Water, Irrigation and Energy
Location: Project Site
Sample ID : Pit No -1
Depth(m): -

Sample Type: Undisturbed
Date: 08/01/2015-17/01/2015
Tested By: G.M
Lab No: 470/07


Void ratio Vs Pressure curve

Approved by: for



Consolidation Test Calculation Sheet

Project : Gidabo Dam Construction Project
Client : Ministry of Water, Irrigation and Energy
Location : Project Site
Samp.ID: Pit No-2
Depth(m) _
Specific gravity : 2.66

Before Test

Weight of sample+Ring	264.70	Diameter,D	75.00	mm
Weiht of Ring	111.66	Area, A	4417.88	mm^2
Weight of sample	153.04	Thickness, H	20.00	mm
Weiht of dry sample	116.09	Volume	88.36	cm^3
Initial moisture content Mo	31.83	Density	1.73	g/cc
		Dry Density	1.31	g/cc

Initial Void e_o= 1.02
Initial satu S_o= 82.635123
Volume change Faector F= 0.1012

After Test

Weight of sample+ring	265.96	Overall settlemen	1.0780	mm
Weight of dry sample+ring	227.75	Volume change	4.7625	cm^3
Weight of Ring	111.66	Final volume	83.5950	cm^3
Weight of wet sample	154.3	Final density	1.8458	g/cc
Weight of dry sample	116.09	Final Dry density	1.3887	g/cc
Weight of moisture	38.21	Final void ratio	0.92	
Final moisture content	32.91%			
Final saturation Sf=	95.64			

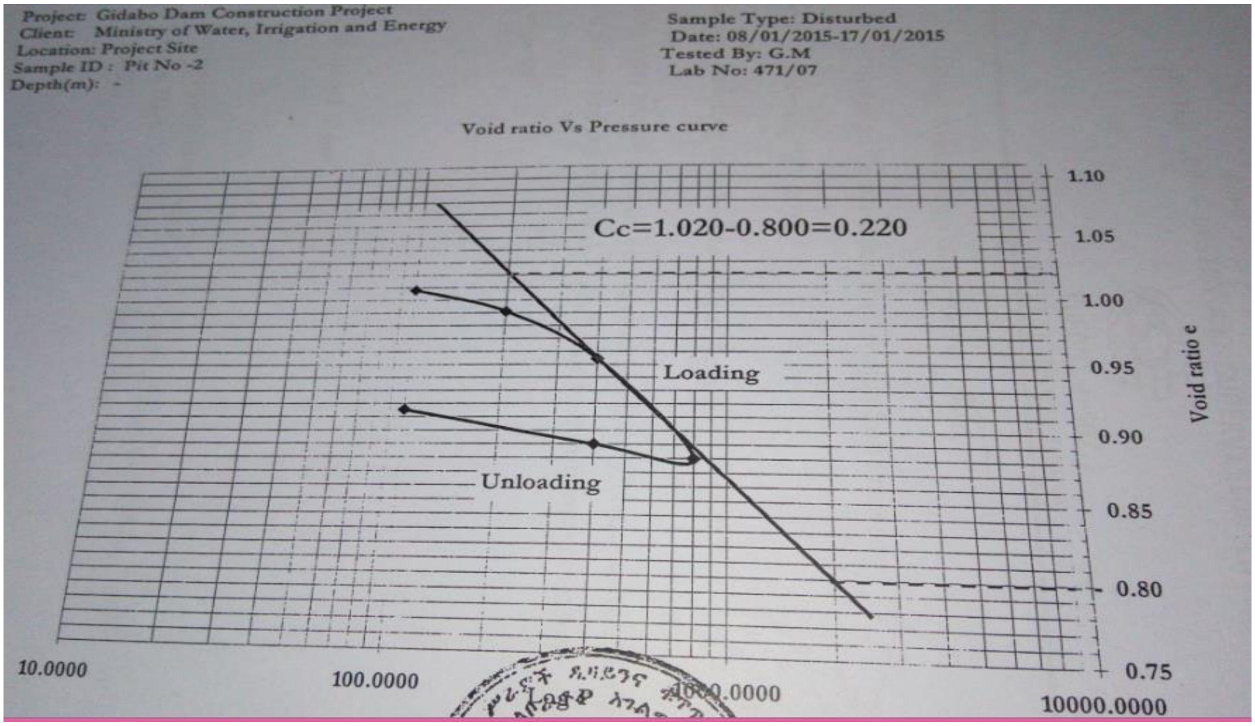
Cosolidation test -data for e-logp curve.

Inc.no	Pressure KN/m^2	Void Ratio Settlement ΔH mm	δe= F*δH	$e=e_o-\delta e$	Volume Cmpressiblity Incremental Changes δe	δp	$1+e_1$	Mv= δe/δp*100	t50 min.	Coefficent of consolidation H=H-δH H_0=20mm	$H_{ave.}=(H_{…}$	$(H_{ave.})^2$	Cv= $0.026*(H_{ave.})^2/t50$	Compression Index,Cc
	0	0	0.101	1.025	0	0				20				
1	50	0.000	0.000	0.000	0.0000	0	0.000	0.000	0.00	0.000	0.000	0.000	0.000	
2	100	0.164	0.017	1.008	0.0166	50	2.008	0.165	4.00	19.836	19.918	396.727	2.579	
3	200	0.330	0.033	0.991	0.0168	100	1.991	0.084	3.42	19.670	19.753	390.181	2.966	0.220
4	400	0.694	0.070	0.954	0.0368	200	1.954	0.094	2.16	19.306	19.488	379.782	4.571	
5	800	1.428	0.145	0.880	0.0743	400	1.880	0.099	4.84	18.572	18.939	358.686	1.927	
6	400	1.330	0.135	0.890	-0.0099	-400								
7	100	1.078	0.109	0.915	-0.0255	-300								

Tested by:

Checked by:

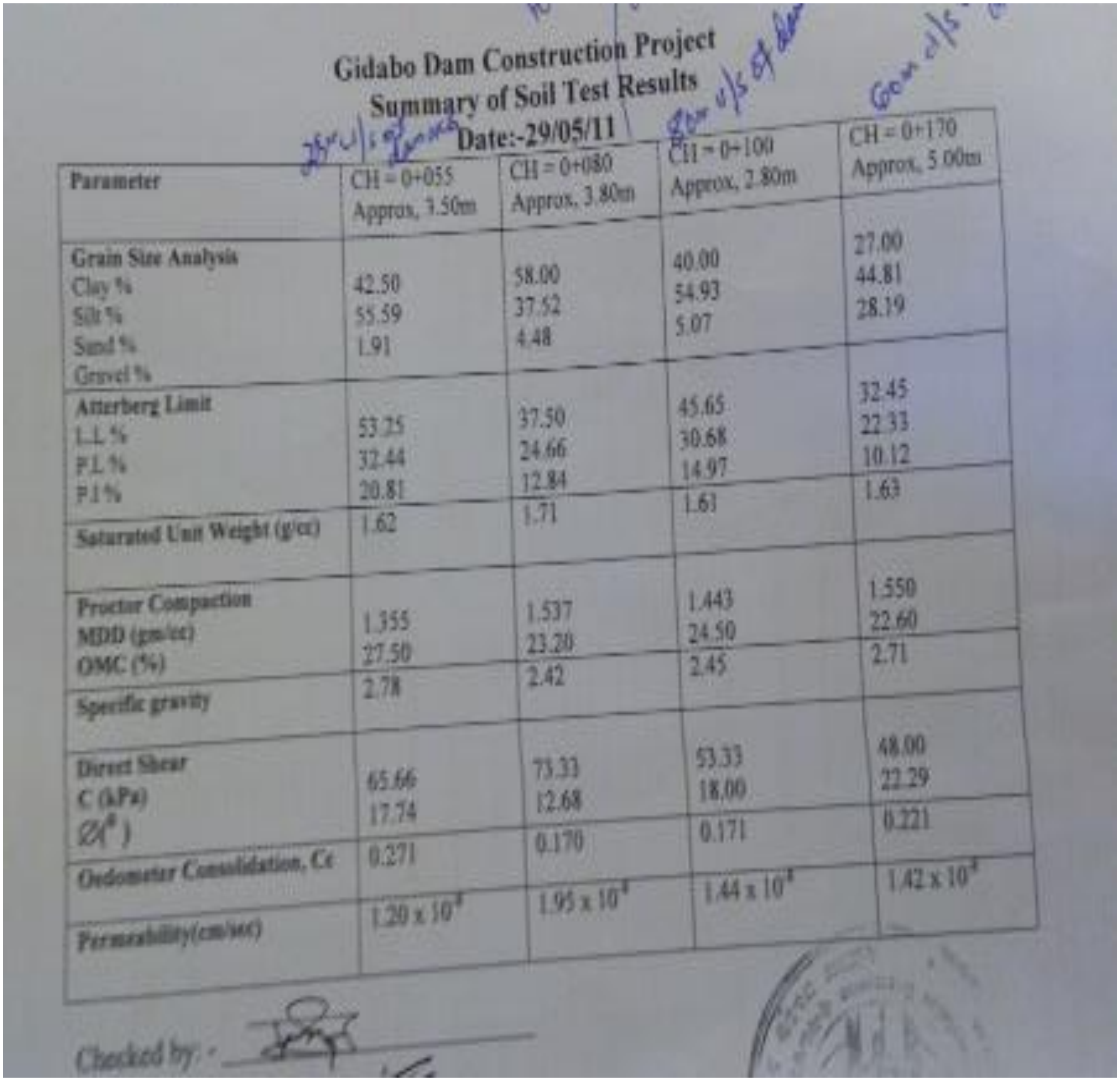
Approved by: for



Project: Gidabo Dam Construction Project
Client: Ministry of Water, Irrigation and Energy
Location: Project Site
Sample ID : Pit No -2
Depth(m): -

Sample Type: Disturbed
Date: 08/01/2015-17/01/2015
Tested By: G.M
Lab No: 471/07

Annexure III consolidation test results from Alluvium Backfill for the conduit foundation



25m u/s of

80m u/s of

60m u/s

Gidabo Dam Construction Project

Summary of Soil Test Results

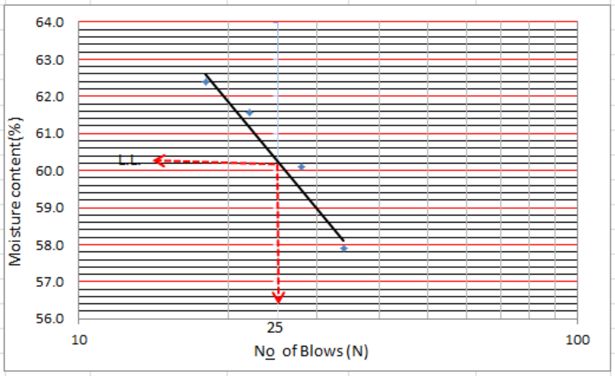
Date:-29/05/11

Parameter	CH = 0+055 Approx, 3.50m	CH = 0+080 Approx, 3.80m	CH = 0+100 Approx, 2.80m	CH = 0+170 Approx, 5.00m
Grain Size Analysis				
Clay %	42.50	58.00	40.00	27.00
Silt %	55.59	37.52	54.93	44.81
Sand %	1.91	4.48	5.07	28.19
Gravel %				
Atterberg Limit				
L.L %	53.25	37.50	45.65	32.45
P.L %	32.44	24.66	30.68	22.33
P.I %	20.81	12.84	14.97	10.12
Saturated Unit Weight (g/cc)	1.62	1.71	1.61	1.63
Proctor Compaction				
MDD (gm/cc)	1.355	1.537	1.443	1.550
OMC (%)	27.50	23.20	24.50	22.60
Specific gravity	2.78	2.42	2.45	2.71
Direct Shear				
C (kPa)	65.66	73.33	53.33	48.00
Ø(°)	17.74	12.68	18.00	22.29
Oedometer Consolidation, Cc	0.271	0.170	0.171	0.221
Permeability(cm/sec)	1.20×10^{4}	1.95×10^{4}	1.44×10^{4}	1.42×10^{4}

Checked by: -

Annexure III summery of soil tests for the alluvium Backfill from the foundation

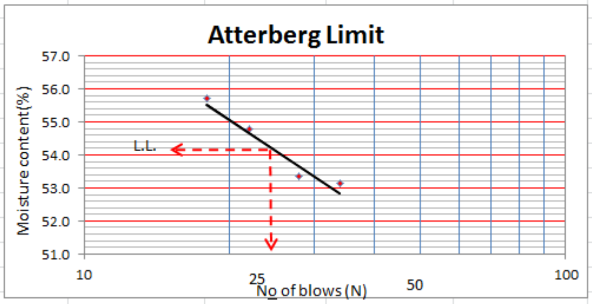
Tin No.	A	4	3	9		11	13
Wet soil + tin (g)	22	22	22.6	24.5		17.0	17.0
Dry soil + tin (g)	15.6	15.5	15.8	16.9		13.7	13.8
Wt of tin (g)	5	5	4.9	4.8		4.8	4.9
Wt of Water (g)	6.4	6.5	6.8	7.6		3.3	3.2
Wt of Dry soil (g)	10.6	10.5	10.9	12.1		8.9	8.9
No. of blows, N	35	28	21	17			
Moisture content,w %	60.4	61.9	62.4	62.8		37.1	36.0



Description	Value %
LL	62.2
PL	36.5
PI	25.7

Tin No.	4	8	9	70		A9	A10
Wet soil + tin (g)	31.71	44.58	34.58	37.3		23.3	22.5
Dry soil + tin (g)	31.05	34.6	28.1	29.3		21.3	20.7
Wt of tin (g)	15.97	16.33	16.2	15.9		15.92	15.8
Wt of Water (g)	0.66	10.03	6.52	7.98		2.0	1.8
Wt of Dry soil (g)	15.08	18.22	11.86	13.4		5.38	4.9
No. of blows, N	23	28	32	20			
Moisture content,w %	4.4	55.0	55.0	59.6		37.2	36.5

Description	Value %
LL	54.2
PL	36.9
PI	17.3



Annexure IV Atterberge Limit results from Alluvium Backfill for the conduit foundation Primary data

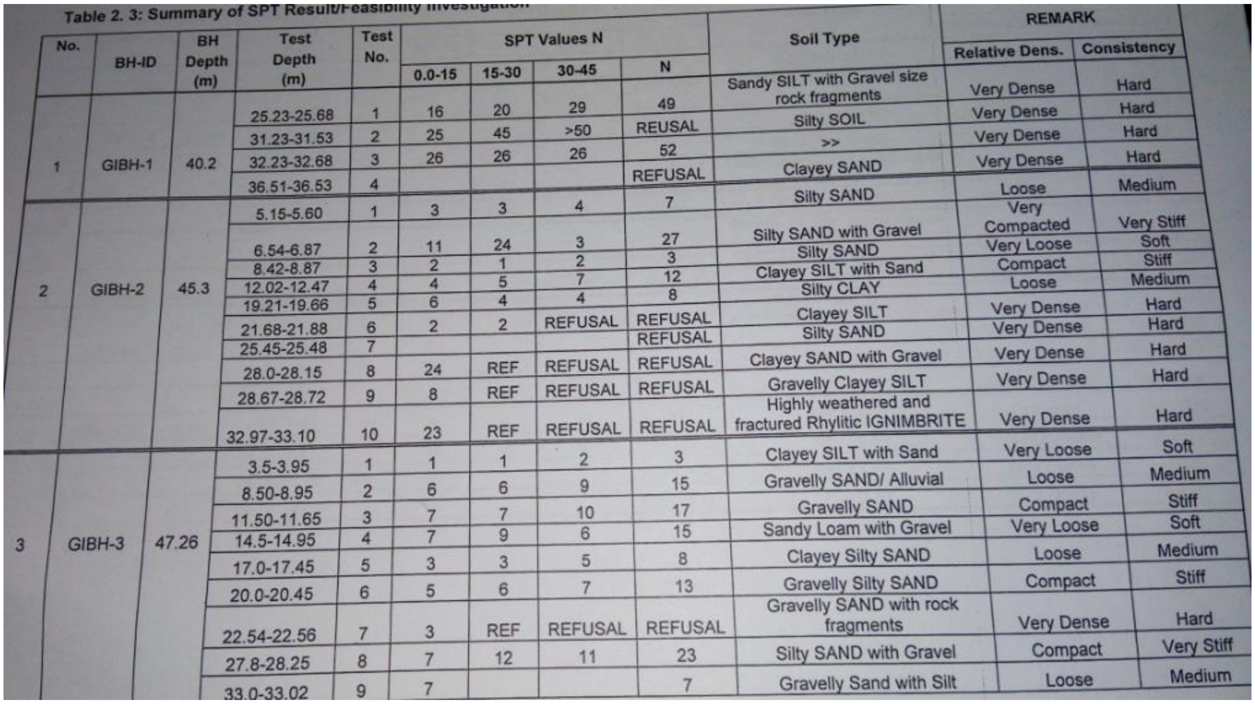
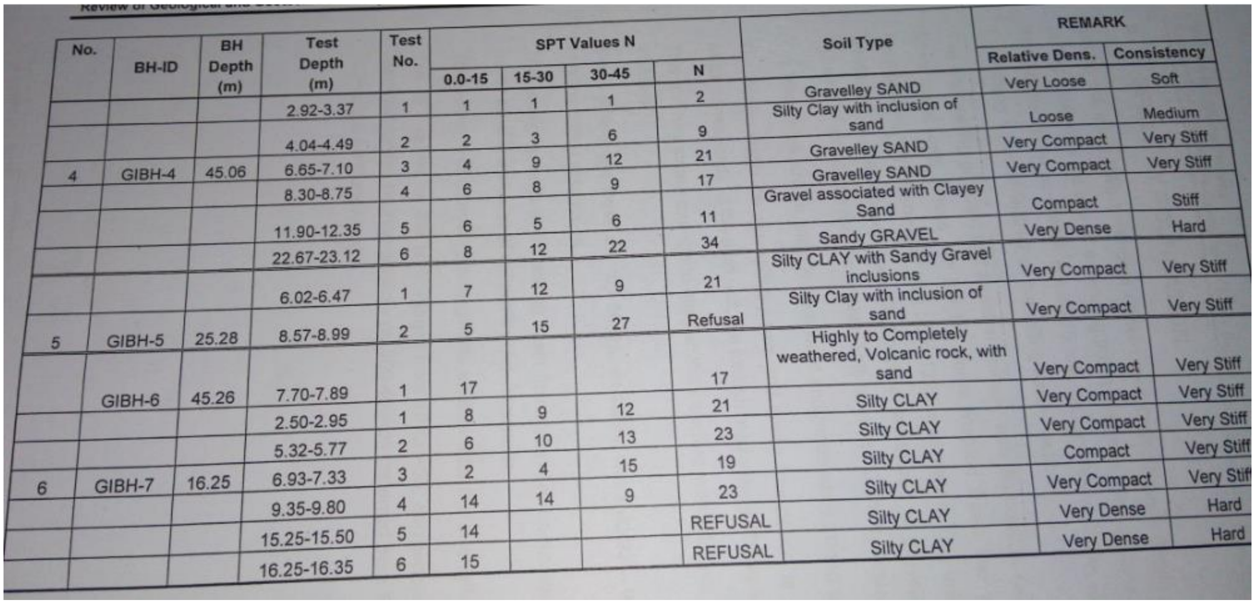


Table 2. 3: Summary of SPT Result/Feasibility Investigation

No.	BH-ID	BH Depth (m)	Test Depth (m)	Test No.	SPT Values N				Soil Type	REMARK	
					0.0-15	15-30	30-45	N		Relative Dens.	Consistency
1	GIBH-1	40.2	25.23-25.68	1	16	20	29	49	Sandy SILT with Gravel size rock fragments	Very Dense	Hard
			31.23-31.53	2	25	45	>50	REUSAL	Silty SOIL	Very Dense	Hard
			32.23-32.68	3	26	26	26	52	>>	Very Dense	Hard
			36.51-36.53	4				REFUSAL	Clayey SAND	Very Dense	Hard
2	GIBH-2	45.3	5.15-5.60	1	3	3	4	7	Silty SAND	Loose	Medium
			6.54-6.87	2	11	24	3	27	Silty SAND with Gravel	Very Compacted	Very Stiff
			8.42-8.87	3	2	1	2	3	Silty SAND	Very Loose	Soft
			12.02-12.47	4	4	5	7	12	Clayey SILT with Sand	Compact	Stiff
			19.21-19.66	5	6	4	4	8	Silty CLAY	Loose	Medium
			21.68-21.88	6	2	2	REFUSAL	REFUSAL	Clayey SILT	Very Dense	Hard
			25.45-25.48	7				REFUSAL	Silty SAND	Very Dense	Hard
			28.0-28.15	8	24	REF	REFUSAL	REFUSAL	Clayey SAND with Gravel	Very Dense	Hard
			28.67-28.72	9	8	REF	REFUSAL	REFUSAL	Gravelly Clayey SILT	Very Dense	Hard
			32.97-33.10	10	23	REF	REFUSAL	REFUSAL	Highly weathered and fractured Rhylitic IGNIMBRITE	Very Dense	Hard
3	GIBH-3	47.26	3.5-3.95	1	1	1	2	3	Clayey SILT with Sand	Very Loose	Soft
			8.50-8.95	2	6	6	9	15	Gravelly SAND/ Alluvial	Loose	Medium
			11.50-11.65	3	7	7	10	17	Gravelly SAND	Compact	Stiff
			14.5-14.95	4	7	9	6	15	Sandy Loam with Gravel	Very Loose	Soft
			17.0-17.45	5	3	3	5	8	Clayey Silty SAND	Loose	Medium
			20.0-20.45	6	5	6	7	13	Gravelly Silty SAND	Compact	Stiff
			22.54-22.56	7	3	REF	REFUSAL	REFUSAL	Gravelly SAND with rock fragments	Very Dense	Hard
			27.8-28.25	8	7	12	11	23	Silty SAND with Gravel	Compact	Very Stiff
			33.0-33.02	9	7			7	Gravelly Sand with Silt	Loose	Medium



Review of Geological and

No.	BH-ID	BH Depth (m)	Test Depth (m)	Test No.	SPT Values N				Soil Type	REMARK	
					0.0-15	15-30	30-45	N		Relative Dens.	Consistency
4	GIBH-4	45.06	2.92-3.37	1	1	1	1	2	Gravelley SAND	Very Loose	Soft
			4.04-4.49	2	2	3	6	9	Silty Clay with inclusion of sand	Loose	Medium
			6.65-7.10	3	4	9	12	21	Gravelley SAND	Very Compact	Very Stiff
			8.30-8.75	4	6	8	9	17	Gravelley SAND	Very Compact	Very Stiff
			11.90-12.35	5	6	5	6	11	Gravel associated with Clayey Sand	Compact	Stiff
			22.67-23.12	6	8	12	22	34	Sandy GRAVEL	Very Dense	Hard
5	GIBH-5	25.28	6.02-6.47	1	7	12	9	21	Silty CLAY with Sandy Gravel inclusions	Very Compact	Very Stiff
			8.57-8.99	2	5	15	27	Refusal	Silty Clay with inclusion of sand	Very Compact	Very Stiff
	GIBH-6	45.26	7.70-7.89	1	17			17	Highly to Completely weathered, Volcanic rock, with sand	Very Compact	Very Stiff
6	GIBH-7	16.25	2.50-2.95	1	8	9	12	21	Silty CLAY	Very Compact	Very Stiff
			5.32-5.77	2	6	10	13	23	Silty CLAY	Very Compact	Very Stiff
			6.93-7.33	3	2	4	15	19	Silty CLAY	Compact	Very Stiff
			9.35-9.80	4	14	14	9	23	Silty CLAY	Very Compact	Very Stiff
			15.25-15.50	5	14			REFUSAL	Silty CLAY	Very Dense	Hard
			16.25-16.35	6	15			REFUSAL	Silty CLAY	Very Dense	Hard

Annexure IV summary of SPT Results

STARTED:19/01/2008 COMPLETED: 29/01/2008 Sheet: 1 of 5

DRILLING METHOD: Rotary Core Drilling	Elevation: (m) 1207	CO-ORDINATES (m) X:407803 Y:711389	BORE HOLE:GIBH-2
Drill Machine XY-1	Inclination 90°	Project : Gidabo dam site, Gidabo Dam and Irrigation project Location : Left bank of the river, flood plain	

01 02 03 04 05 06 07 08 09 10

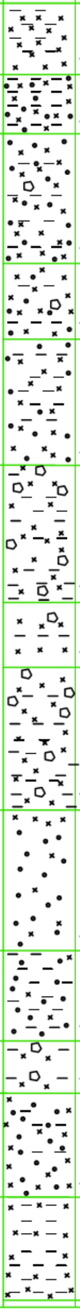
DESCRIPTION OF CORE SAMPLE	SYMBOLICLOG	Depth (m)	Core Run Information						Notes/remark
			Core run	Longth of core: (m)	Core recovery %	SCR %	RQD %	F	
Brown, loose clayey sand and /or silty sand contains recent plant root.		0.26	1	0.25	96				
		0.80	2	0.52	96				
		1.68	3	0.85	97				
		2.58	4	0.88	98				
Soft, light brown, silty clay		3.07	5	0.48	98				
Brown, soft, moderately plastic clayey silt with sand		3.37							
		4.10	6	1.00	97				
		5.15	7	1.00	95				
Dark brown, loose silty sand.		6.14	8	0.93	96				
Layers of dark brown, loose silty sand with gravel and soft clayey loam.		6.54	9	0.40	100				
		6.94							
		7.55	10	0.98	97				
Dark brownish gray, soft clayey silt containing gravelly		8.42	11	0.85	98				
Yellowish brown, loose to weakly dense silty quartz sand		9.52	12	1.00	91				
Brown, loose gravelly gravelly quartz sand									

KEY T.C.R = Total core recovery F = Fractures
S.C.R = Solid Core Recovery
R.Q.D = Rock Quality Designation

REMARKS:

STARTED: 19/01/2008 | COMPLETED: 29/01/2008 | Sheet: 2 of 5

DRILLING METHOD: Rotary Core Drilling	Elevation: (m) 1207	CO-ORDINATES (m) X:407803 Y:711389	BORE HOLE:GIBH-2
Drill Machine XY-1	Inclination 90°	Project : Gidabo dam site, Gidabo Dam and Irrigation project Location : Left bank of the river flood plain	



DESCRIPTION OF CORE SAMPLE	SYMBOLICLOG	Depth (m)	Core Run Information						Notes
			Core run	Longth of core: (m)	Core recovery %	SCR %	RQD %	F(no.)	
* See sheet 1 of 5		10.55	13	1.00	97				
Greenish gray,soft clayey sandy silt		11.02	14	0.45	96				
Light brown, soft, clayey silt with sand		12.02	15	0.95	95				
		12.59	16	0.55	96				
		13.59	17	0.93	93				
Brownish yellow, soft to firm, clayey silt contains gravel.		14.62	18	0.93	90				
		15.12	19	0.48	96				
		16.22	20	1.05	95				
Loose, greenish gray, silty quartz dominating sand		17.31	21	1.00	92				
Dark gray, soft, moderately plastic, silty clay containing organic matter (slightly organic soil)		18.00							
Dark gray, loose quartz silty sand containing gravel		18.41	22	1.05	95				
Dark gray, soft to firm, clayey silt with sand (slightly organic soil?)		19.21	23	0.75	94				
Dark gray, firm, moderately plastic silty clay									

KEY T.C.R = Total core recovery
S.C.R = Solid Core Recovery
R.Q.D = Rock Quality Designation
F = Fractures

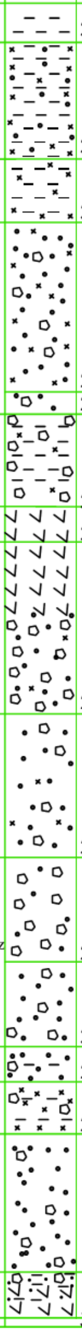
REMARKS:

STARTED: 19/01/2008

COMPLETED: 29/01/2008

Sheet: 3 of 5

DRILLING METHOD: Rotary Core Drilling	Elevation: (m) 1207	CO-ORDINATES (m) X:407803 Y:711389	BORE HOLE:GIBH-2
Drill Machine XY-1	Inclination 90°	Project : Gidabo dam site, Gidabo Dam and Irrigation project Location :Left bank of the river, flood plain	



DESCRIPTION OF CORE SAMPLE	Depth (m)	Core Run Information						Notes
		Core run	Length of core: (m)	Core recovery %	SCR %	RQD %	F(no.)	
	20.30	24	1.03	94				
Dark gray, soft, silty clay containing band of sand (slightly organic soil?)	21.18	25	0.83	94				
Dark gray, soft to firm, weakly plastic clayey silt	21.68	26	0.50	100				
Dark gray, soft, gravel sandy silt	22.98	27	1.20	92				
	23.15							
Dark gray, soft to firm gravelly clayey silt (silty loam)	23.88	28	0.88	98				
Highly weathered, fractured(fragmented) volcanic rock	24.13							
Layers of yellowish gray, highly weathered fragmented pyroclatic rock and dark gray, very loose, gravelly sand.	25.45	29	1.47	94				
Gray loose , quartz rich silty sand with gravel.	26.55	30	1.00	93				
Dark gray, loose, gravelly to coarse quartz sand.	27.35	31	0.75	94				
	28.00	32	0.60	92				
Dark gray. loose, clayey quartz sand with gravel	28.25							
Dark gray, soft, gravelly clayey silt	28.67	33	0.63	94				
Dark gray, loose medium to coarse grained sand to gravelly sand								
Loose, silty clayey sand containing decomposed gravelly volcanic rock	30.08	34	1.27	94				

KEY T.C.R = Total core recovery
S.C.R = Solid Core Recovery
R.Q.D = Rock Quality Designation

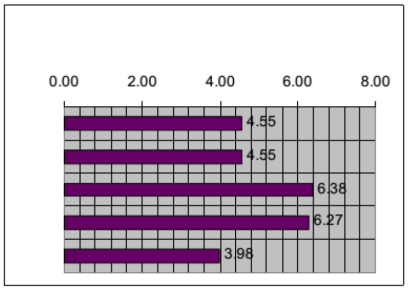
F = Fractures

REMARKS:

STARTED: 19/01/2008 COMPLETED: 29/01/2008 Sheet: 4 of 5

DRILLING METHOD: Rotary Core Drilling	Elevtion: (m) 1207	CO-ORDINATES (m) X:407803 Y:711389	BORE HOLE:GIBH-2
Drill Machine XY-1	Inclination 90°	**Project :** Gidabo dam site, Gidabo Dam and Irrigation project **Location** :Left bank of the river	

DESCRIPTION OF CORE SAMPLE	SYMBOLICLOG	Depth (m)	Core Run Information					
			Core run	Length of core: (m)	Core recovery %	SCR %	RQD %	F
Gray, fragmented, highly to completely weathered volcanic rock.		30.40	35	0.30	94			
		30.70						
Brownish gray, loose to very loose, quartz dominating sand containing highly to completely weathered porphyritic volcanic rock.		31.40	36	0.90	90			
		32.97	37	1.57	96			
Pinkish gray, moderately to highly weathered and fractured, ignimbritr or rhyolitic ignimbrite.		33.57	38	0.54	90	78	75	2
Yellowish gray to gray, weakly weathered, moderately strong, porphyritic ignimbrite or rhyolite ignimbrite?		34.54	39	0.90	93	74	62	11
		35.64	40	1.00	91	83	77	
Brownish gray, moderately weathered, weakly strong porphyritic vesicular rhyolitic ignimbrite, towards the bottom fragmented.		36.74	41	1.00	91	89	68	
		38.58	42	1.66	47	13	0	
Gray, moderately to highly weathered, weak, porphyritic rhyolite or rhyolitic ignimbrite		39.67	43	0.98	75	50	46	13
		40.0	44	1.08	65	37		>7



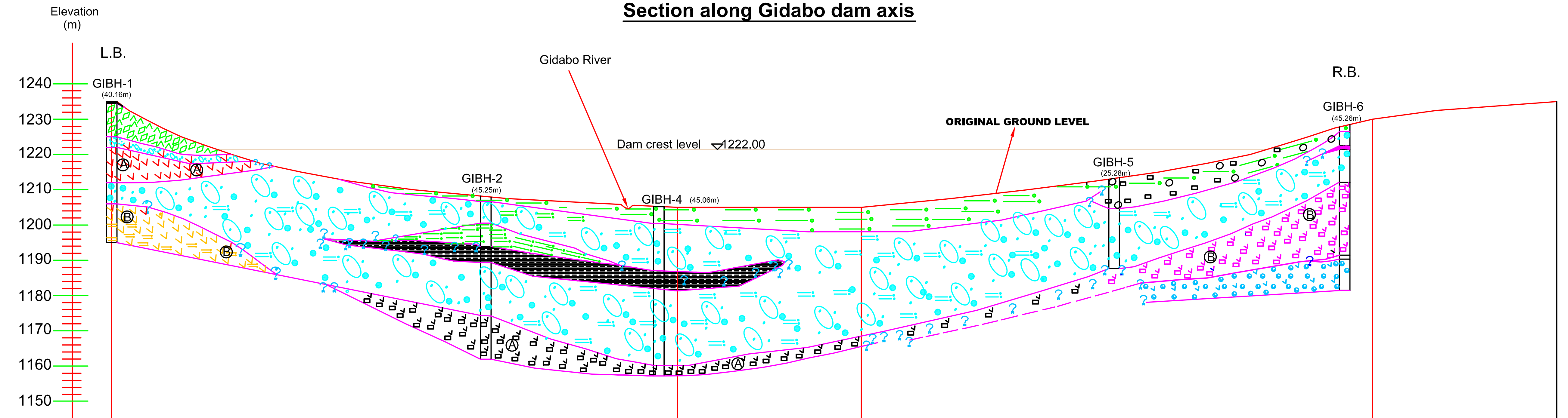
KEY T.C.R = **Total core recovery**
S.C.R = **Solid Core Recovery**
R.Q.D = **Rock Quality Designation**
F = Fractures

REMARKS:

CLIENT:MOW CONSULTANT:WWDSE CONTRACTOR:CJCIETC

Annexure V Borehole loge of GIBH-2

Section along Gidabo dam axis



GROUND ELEVATION (m)	TOTAL DISTANCE (m)	PARIAL DISTANCE (m)
1235	0.0	0.0
1232.5	4.9	4.9
1230	9.5	4.7
1227.5	14.4	4.9
1225	19.7	5.3
1222.5	26.7	7.0
1220	34.1	7.4
1217.5	42.0	8.0
1215	53.7	11.7
1212.5	67.1	13.4
12110	85.5	18.4
1207.5	112.2	26.7
1205.5	160.3	48.1
1205.5	212.4	52.1
1207.5	237.7	25.3
1210	259.5	21.8
1212.5	278.9	19.4
1215	296.5	17.7
1217.5	310.4	13.8
1220	322.5	12.1
1222.5	330.9	8.3
1225	338.9	8.1
1227.5	346.5	7.6
1230	357.3	10.8
1232.5	375.4	18.1
1235	409.4	34.0

NOTES:-

1. All dimentions are in millimeters, unless otherwise stated.
2. All levels are given in meters.

LEGEND (Lithological)

Soil unit

▨	Silty gravel (Gravel:- volcanic rock fragments), residual soil.
▨	Clayey silt with sand contains recent plant remains (Alluvial soil)
▨	Coarse to gravelly sand with intermittant layers of silty and clayey soil
▨	Layers of organic and inorganic fine soil (silty clay and/or clayey silt with sand)

Rock unit

▨	IGNIMBRITE: Greenish gray, weakly weathered, moderate to highly strong porphyritic ignimbrite
▨	Fraible Lithic Tuff: Moderately weathered, weakly strong, fraible tuff (lithic)
▨	Ash: very thin ash layer found in GIBH-6
▨ Ⓐ Ⓑ	(A) Rhyolitic ignimbrite:weakly weathered, moderately strong and porphyritic rock. (B) Weathered volcanic rock (ignimbrite?): Highly weathered, weak, crystal containing volcanic rock
▨ Ⓐ Ⓑ	(A) Lithic tuff: Light red or brown, weakly weathered, moderately strong, lithic tuff (B) Lithic tuff + residual soil: Gray, moderately weathered, weak lithic tuff and completely weathered tuff and residual soil towards the bottom.
▨	Basalt: fresh to moderately weathered, moderately strong aphanitic basalt.
? ?	Not define (Contact or lateral lithological extent-not known)

REVISION	DATE	DESCRIPTION

FEDERAL DEMOCRATIC REPUBLIC OF ETHIOPIA
MINISTRY OF WATER RESOURCES

WATER WORKS DESIGN AND SUPERVISION ENTERPRISE
IN ASSOCIATION WITH CONSULTING ENGINEERING SERVICE (INDIA) PVT. LTD

PROJECT:– GIDABO IRRIGATION PROJECT
FINAL FEASIBILITY STUDY

TITLE:– **Geological profile along Gidabo Dam axis**

DRAWN	TY.	DATE	MAY 2008
DESIGNED		SCALE	1:100
CHECKED		SHEET NO.	
APPROVED		DRAWING NR.	OR-SNN/GIDIP/FIG. 4.2