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Index and Mechanical Properties of Collapsible Soils (A Case Study on Ziway-Arsi Negele Highway Project)

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A Thesis Submitted to the School of Graduate Studies in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Civil Engineering (Geotechnical Engineering)

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Declaration

I, the undersigned, certify that this research work titled "Index and Mechanical Properties of Collapsible Soils (A Case Study on Ziway-Arsi Negele Highway Project)" is my own work performed under the supervision of my research advisor Dr. Ing. Samuel Tadesse as part of Masters of Science Program in Geotechnical Engineering and has not been presented elsewhere for assessment and for a degree in any other university. All sources of materials used for this thesis have also been duly acknowledged in the text.

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Abstract

Collapsible Soils, unlike many other types of problematic soils found in Ethiopia, did not get the necessary attention in researches conducted to this date. However, due to their particular engineering characteristics, they exhibit large settlement upon wetting posing threat to embankments and foundations built over them.

Due to lack of researches on collapsible soils and the limited attention given in engineering design manuals of the country, many professionals in the construction industry fails to clearly understand their index and mechanical properties.

This research had the aim of assessing the index and mechanical properties of collapsible soils found between Zeway and Arsi Negele towns by taking the Modjo-Hawassa Dual Carriageway Highway Project, Lot 3; Zeway-Arsi Negele as a case study.

To this regard, a series of field and laboratory tests were conducted on representative collapsible soil samples collected from the above-mentioned study area. Field density test has been conducted to determine the in-situ densities of the sampling areas. Furthermore, laboratory tests like specific gravity, grading analysis, Atterberg limits, and moisture-density relationships have been carried out to determine their index as well as mechanical properties. In so doing, it was observed that the soils have very low in-situ densities, mainly consist of sands and silts and low plasticity.

Moreover, in order to investigate the collapse nature of the soil samples, collapse potential test has been carried out using one dimensional Oedometer by remolding the samples to their in-situ as well as maximum dry densities so as to perceive the nature of collapsibility of the soils at their natural state and at their compacted state. Accordingly, it was found out that the soils have moderate collapse potential at their natural state and does not exhibit any kind of collapsibility at their compacted state.

Key Words: Collapsible Soils, Collapse Index, Collapse Potential, Index Properties, Mechanical Properties

Contents

Lists of Abbreviations/Glossary

AASHTO……American association of State Highway and Transportation Officials

ASTM………..American Society for Testing of Materials

CBR…………..California Bearing Ratio

CI……………..Collapse Index

CP…………….Collapse Potential

ERA…………..Ethiopians Roads Authority

GPS…………...Global Positioning System

HEIC…………High Energy Impact Compaction

LL……………..Liquid Limit

MDD…………Maximum Dry Density

NMC…………Natural Moisture Content

NP……………..Non Plastic

OMC…………Optimum Moisture Content

PI………………Plasticity Index

PL……………...Plastic Limit

RIC…………….Rapid Impact Compaction

USCS…………Unified Soil Classification System

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1. Introduction

1.1 General

Problematic soils are types of soils which can pose serious threats to engineering structures built upon them. Expansive, collapsible, dispersive soils etc. are some examples of problematic soils.

Collapsible soils, one type of problematic soils, are described as soils that undergo a significant, sudden and irrecoverable decrease in volume upon wetting. These types of soils dominantly contain silt and sand with some clayey material and rock fragments.[9]

Collapsible soils, also known on some literatures as loess soils, show relatively high apparent strength in their dry state, but have low density, porous structure and are susceptible to large deformations upon wetting due to sudden large reduction in their volume.[10] These properties make collapsible soils unsuitable for construction of embankments in highways, building foundations or any other load bearing engineering structures in their natural state.

Different methods of treatment and/or improvement of collapsible soils are available. These methods include removal and full/partial replacement, excavation and re-compaction, pre-wetting and surcharging, impact compaction, dynamic compaction, treatment with cementitious materials, chemical grouting etc.[23]

Although many researches have been conducted in other countries where collapsible soils exist, such as the United States of America, China, Nigeria, South Africa etc., no satisfactory research and investigation has been made and published on collapsible soils found in our country. Many of the focus areas of the researches undertaken so far have given emphasis on expansive soils, among any other problematic soils, and less attention is given on the existence of collapsible soils.

Due to the above-mentioned reasons, there is no extensive knowledge within even the experienced engineering professionals on the methods of identification, classification and treatment of collapsible soils.

To this regard, this research thesis is aimed to investigate the index and mechanical properties of collapsible soils found in the Mojo-Hawassa Design and Build Highway Project, Lot-3; Zeway-Arsi Negele Section.

1.2 Objectives

1.2.1. General Objective

The general aim of this thesis work is to study the index and mechanical properties of collapsible soils.

1.2.2. Specific Objective

- To identify the collapse potential and severity of the soil found in the project at their natural state
- To identify the collapse potential and severity of the soil found in the project at their compacted state
- To study other index and mechanical properties of the soils and making correlation with their collapse nature

1.3 Statement of the Problem

Collapsible soils can pose serious problems to building foundations and road embankments built on them. Even though, their presence in Ethiopia is stated in few literatures, a thorough identification and research has not been done satisfactorily. Even the local manuals, except mentioning their identification and some properties, they fail to recommend/propose detailed suitable remedial measures for collapsible soils. Consequently, lack of knowledge and experience is observed in professionals working in the industry. Therefore, consecutive and extensive researches shall be conducted in construction areas covered by collapsible soils in order to identify the index and mechanical properties of the same and come up with suitable remedial measure and to alleviate the awareness problem found among construction professionals.

1.4 Scope and Limitations

The scope of this thesis is to the extent of examining relevant index and mechanical properties including natural moisture content, gradation, plasticity, specific gravity, in-situ and maximum densities and one dimensional Oedometer collapse potential tests on collapsible soil samples collected from Mojo-Hawassa Design and Build Highway Project, Lot-3; Zeway-Arsi Negele Section.

In conducting this research, the following limitations were encountered: -

- Only the index and mechanical properties of the soils which are directly related with collapsibility are investigated. Due to time and cost limitations other engineering tests (including shear strength and CBR tests) are omitted.

Therefore, the findings and recommendations found in this research should be addressed by considering the above-mentioned limitation. However, further studies have to be made by conducting full engineering tests (including shear strength tests, CBR etc.) on the problematic soils under consideration in order to have full insight on their properties.

In addition, research works focusing on verifying and checking the effectiveness of the proposed remedial measures have to be conducted. These works have to be supported by a series of pre and post treatment/improvement tests so as to confirm whether the utilized methods have been effective in improving the collapse nature of the soils.

1.5 Organization of the Thesis

The presentation of this thesis work is organized in six (6) chapters.

- The first Chapter contains a general introduction about collapsible soils, general and specific objectives of the work, statement of the problem, and Scope & limitations of the study.
- Chapter two reviews related literatures on the definitions, occurrence, collapse mechanism, identification and test methods and stabilization and improvement of collapsible soils.

- Chapter three includes the brief discussion on the general description (including the location, geology, temperature, rainfall and groundwater characteristics) of the study area, the tasks done to identify the properties and extent of the collapsible soils etc.
- Chapter four elaborates on how the sampling points are selected, samples were extracted and transported to laboratory. In addition, it also discusses how field and laboratory tests are conducted.
- Chapter five presents the test results obtained for the field and laboratory tests carried out and discussion of results from the theoretical and reviewed literatures standpoint.
- Chapter six finally gives conclusions and recommendations based on the work done and the result achieved thereof.

The detail test results of the field and laboratory works are presented in Annex.

2. Literature Review

2.1 Definitions

A collapsible soil is commonly referred to as a meta-stable structured soil. An increase in pore-water pressure results in swelling for a stable-structured soil, whereas an increase in pore-water pressure may cause a volume decrease for a metastable-structured soil. [12]

Collapsible soils pose potential threat to structures built on them when wetted. At their dry, natural state, they possess stiffness and high apparent shear strength; but upon wetting, they could exhibit a significant decrease in volume (collapsing, hydro-consolidation, hydro-compression). [1]

A collapsing soil structure consists of loosely packed soil grains held in position by clay bridges, capillary tension and cementing agents. Collapse occurs when the above factors are reduced or eliminated by wetting and/or surcharge loads. When a soil collapses, a major reorientation of the soil grains occurs which results in a change in the soil fabric and a reduction in the soil's porosity.

The addition of large amounts of water to an arid climate soil may cause several reactions: swelling of clays, reduction in shear strength, change in the soil-water system, and elimination of capillary forces between grains. Thus, the effect of a substantial increase in soil moisture may often be to change the soil structure. Applied boundary forces, such as a surcharge load, may also change the soil structure as the particles readjust to new stress conditions.

Based on the above statements, a collapsing soil is defined as a soil which rapidly undergoes an appreciable loss of volume due to a major readjustment of the soil fabric upon wetting or upon the application of a boundary force. A boundary force may or may not be required to induce collapse. [10]

2.2 Occurrence and Damage

Loess soil deposits are widely distributed in arid and semi-arid regions and constitute about 10% of land area of the world. Several countries including China, Russia, United States, France, Germany, New Zealand and Argentina, have a large area of loess soil deposits. [13]

Robert D. Evans and et al, similarly, has reported that, depositions of windblown silt has produced major surface formations of collapsible soil, often underlying areas of high population and major infrastructure works, in North and South America, Europe, central Asia and China, including an almost continuous deposit from North China to south-east England. [21]

In Africa, their presence has also been a subject of research in Nigeria (O. Titilayo and et al), Egypt (H. H. Abdelmohsen and et al), South Africa (George Brink and et al), etc.

In Ethiopia, they are present in the Southern part of the Omo River and in the Central land Southern part of the rift valley. Often, their existence around Zeway, Shashemene and Hawassa is manifested by the occurrence of ground cracks and potholes during heavy rains or floods due to hydro-compaction. In Afar region, collapsible soils are present in the form of sand dunes. [9]

According to S. L. Houston and et al, there have been several reported cases for which differential collapse has been cited as the cause of roadway or highway bridge distress in the United States. A few of these in the Arizona and New Mexico region include sections of I-10 near Benson, Arizona, and sections of I-25 in the vicinity of Algadonas, New Mexico. In addition to the excessive waviness of the roadway surface, bridge foundations failures, such as the Steins Pass Highway bridge, I-10, in Arizona, have frequently been identified with collapse of foundation soils. [22]

Collapse and other collapse associated problems, such as differential settlement, earth cracks, landslides and falls, have contributed to serious damages to the infrastructures that are constructed on loess soils, including loss of human lives in certain scenarios. During the period of 1974 to 1975, in China, it was reported that a total of 1505 buildings were damaged and 80 km-long underground pipeline ruptured due to collapse of loess soils. [13]

2.3 Collapse Mechanism

The usual mechanism for soils to collapse is loading and wetting; where these soils can show high apparent strength in its natural state but collapse occurred as the bonds between the grains break down when the soil is wetted or loaded. The increase in load or more properly stress, will typically derive from an accumulation of deposits over a long period of time, although dynamic stresses from an event such as an earthquake would provide an obvious trigger mechanism as would stress increases caused by construction operations (for example the stress increase due to construction of embankments). Wetting usually refers to an increase in saturation ratio, often approaching full saturation from a partially saturated state. [15]

Amer Ali Al-Rawas (2000) has put similar discussion on the collapse mechanism of collapsible soils. In his publication, he elaborated that the collapse phenomenon is primarily related to the open structure of the soil. He has presented Casagrande's (1932) demonstration which states that a portion of the fine-grained fraction of the soil exists as bonding material for the larger grained particles and that these bonds undergo local compression in the small gaps between adjacent grains resulting in the development of strength.

At natural moisture content, these soils compress slightly as a result of increasing overburden pressures due to construction. However, the structure remains sensibly unchanged. When the loaded soil is exposed to moisture, and a certain critical moisture content is exceeded, the fine silt or clay bridges that are providing the cementation will soften, weaken and/or dissolve to some extent. Eventually, the binders reach a stage where they no longer resist deformation forces and the structure collapses as shown in the following figure. [1]

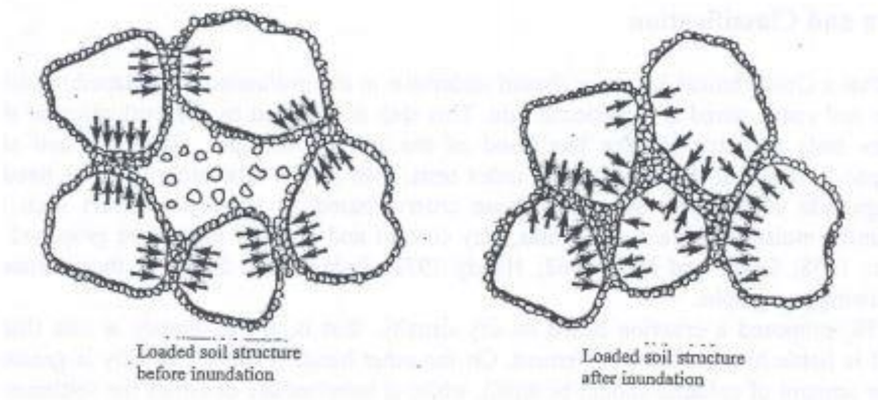

Figure 1: Loaded Structure Before and After Inundation (after Casagrande 1932)

2.4 Identification and Test Methods

2.4.1 Field Identification

S. L. Houston and et al (2002) discuss that local site geology and depositional processes should be considered first in the assessment of collapse potential of the soil profile. This information, together with climatological data, can be a very valuable tool in planning the site investigation. In the case of an existing roadway, the geomorphological clues can be coupled with knowledge of performance to help identify possible problem soil conditions. Geographical and geological information is strongly correlated with collapsibility and collapse potential. Engineering experience and familiarity with local infrastructure performance and geological conditions is an essential element of the site characterization. [23]

Besides predicting the presence of collapsible soils based on geographical and geological data, there is also simple test which can be carried out at the site. In this very simple field test, a block of material about hand size is taken from the side of a test pit, from auger boring cuttings, or other sources. It is broken into two pieces and each is trimmed until they are approximately equal in volume. One specimen is then wetted and molded in the hands to form a damp ball. The volume of this ball is then compared with the volume of the undisturbed specimen. If the wetted ball is obviously smaller than the undisturbed piece, there is potential for soil collapse. [19]

Furthermore, as elaborated on ERA's Site Investigation Manual, often the existence of collapsible soils is manifested by the occurrence of ground cracks and potholes during heavy rains or floods due to hydro compaction. [9]

Figure 2: Collapse holes near Shashemene (Source: ERA Site Investigation Manual)

2.4.2 Laboratory Identification

Many different techniques are used to identify collapsible soils. They may be divided into two main groups; indirect methods and direct methods. The indirect methods assess the collapse potential by correlating it with other index and mechanical properties such as unit weight, Atterberg limits or percent clay particles. Although these classifications can be useful, they provide little quantitative estimates of potential settlements.

One of such correlation is made by Clevenger (1958) who proposed a criterion based on dry density, that is, if the density is less than 1.28 g/cc, then the soil is liable to significant settlement. On the other hand, if the dry density is greater than 1.44 g/cc, then the amount of collapse should be small, while at intermediate densities the settlements are transitional.[1]

On the other hand, ERA's 2013 Geotechnical Design Manual discusses that usually, the collapse potential of the soil depends on density, gradation, the initial water content, composition, and the extent of loading at the time of wetting. A good indication of the potential for high collapse potential is a very low density. Typical collapsible soils have densities of less than 1.6 g/cc (mostly in the range of 1 to 1.585 g/cc). [8]

However, it should be noted that since unstable soils have a wide range of dry densities which also apply to non-collapsible soils, the dry density alone seems to be misleading and an unreliable indicator of collapse. Furthermore, other factors such as the soil moisture content, which greatly influences the soil collapse was not taken into account.

Bell and Bruyn (1997) reported a criterion suggested by Gibbs and Bara (1962) based on natural dry density and liquid limit, which distinguishes between collapsible soils and non-collapsible soils as shown in Figure 3. The method is based on the premise that a soil, which has enough void space to hold its liquid limit moisture at saturation is susceptible to collapse on wetting. Figure 3 shows that soils above the lines (labeled as potentially metastable) are in a loose condition, and when fully saturated will have a moisture content greater than liquid limit and are susceptible to collapse. [1]

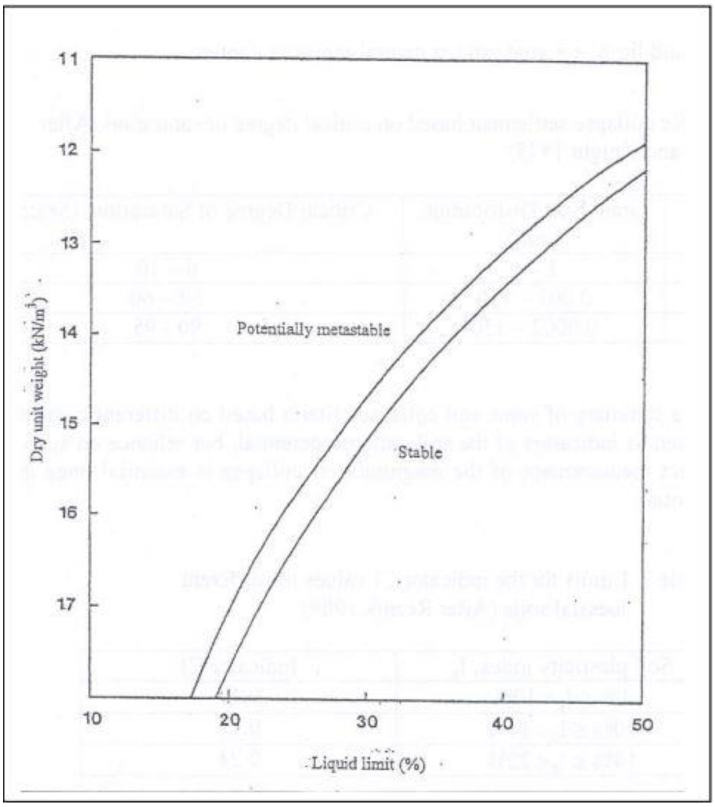

Figure 3: Classification of collapsing soils Based on Liquid Limit and Dry Unit Weight (after Gibbs and Bara 1962)

However, these criteria should be taken as indicators of the soil collapse potential, but reliance on such criteria can be misleading. Therefore, direct measurement of the magnitude of collapse is essential once the soil showed indications of collapse potential. [1]

Direct methods like single and double Oedometer tests and other in-situ collapse tests, however, involve actually wetting the soil, either in laboratory or in-situ, and measuring the corresponding strain. The results of such tests can be extrapolated to the entire soil deposit and the potential settlements can be predicted. [3]

ASTM D5333 (Standard Test Method for Measurement of Collapse Potential of Soils), outlines procedures for the determination of the magnitude of one-dimensional collapse that occurs when unsaturated soils are inundated with fluid. It defines two parameters; Collapse Index (Ic), percent-relative magnitude of collapse determined at 200 kPa and Collapse Potential (Ic), percent-relative magnitude of soil collapse determined at any stress level. Both parameters can be calculated by the same formula as given in equation 1.

$$I_c = \frac{\Delta e}{1+e_o} * 100 \qquad \text{(eq.1)}$$

Where Δe= change in void ratio resulting from wetting

e_o= initial void ratio

ASTM D5333 also specifies the severity of collapse based on the test result which is tabulated on Table 1.[4] The graph of one-dimensional collapse potential test has the typical behavior as shown on Figure 4. The portion of the graph from the starting point “A” to “B” shows the settlement caused due to load increments until the appropriate vertical stress, which depends on whether collapse index or potential is intended to be determined, is reached. The section from “B” to ”C” shows the settlement attributed to collapse merely due to inundation of the specimen at the constant appropriate vertical stress. This reading is inserted into eq.1 and the result will be reported as collapse potential/index. The portion from “C” to “D” depicts settlement due to further loading after the specimen collapsed.

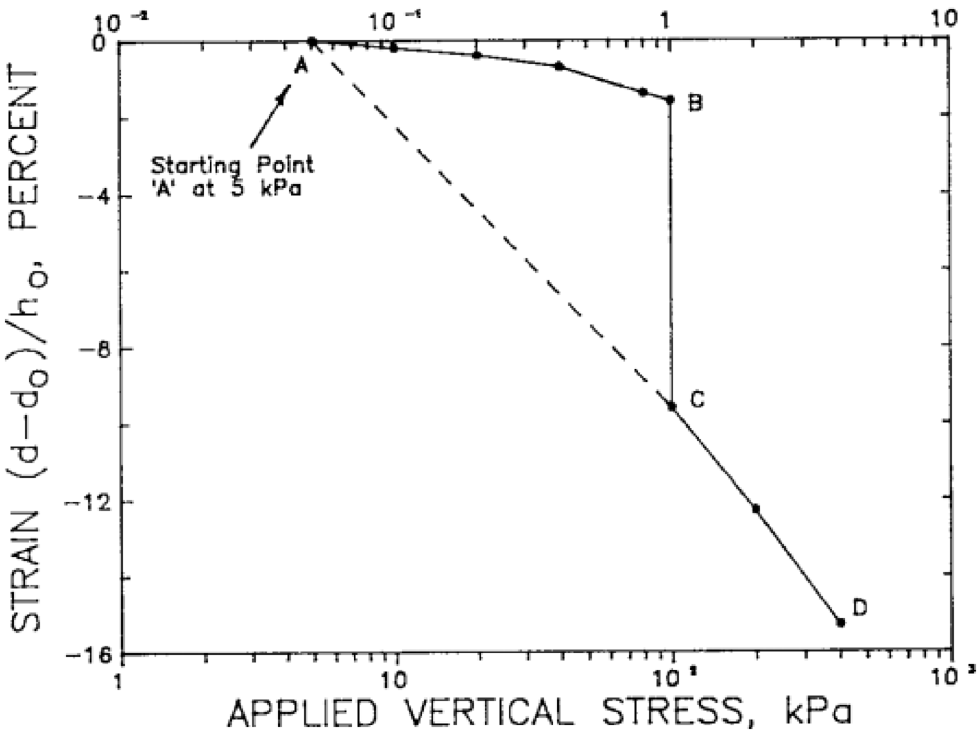

Figure 4: Typical Compression Curve for Collapsing Soil after Collapse Potential Test (Source: ASTM D5333)

Table 1: Classification of Collapse Index as per ASTM D5333

Degree of Collapse	Collapse Index (%)
None	0
Slight	0.1-2.0
Moderate	2.1-6.0
Moderately Severe	6.1-10.0
Severe	> 10.0

2.5 Ground Improvement Techniques

Many mitigation methods are available for collapsible soil problems. These include removal of the collapsible soil, removal and replacement, prevention of wetting, chemical stabilization, pre-wetting, controlled wetting, dynamic compaction, pile or pier foundations, and differential settlement resistant foundation and structural design. [24]

The method selected depends upon several factors such as the depth of the collapsible soil deposit, the type of structure to be built, the capability of the structure to withstand settlement, the likelihood of the foundation to become wetted, and the stresses imposed on the foundation by a structure. [19] Of these alternative mitigation methods, some are much more suited to pavement applications than others. [23]

In assessing mitigation measures, the issue of shallow or deep deposits must be considered, and that distinction is often not clear cut. If collapsible soils are identified in foundations for structures, a prime objective should be to stabilize those soils before construction. It is much less expensive to solve the problem at that point rather than after the project is built. Then maintenance costs are reduced; project operation is more efficient; and the lives of structures are extended. [19]

Hereunder, review of literatures on selected mitigation measures are presented on the following sections.

2.5.1 Excavation and Re-Compaction

A typical solution to poor ground conditions encountered in foundation soils (e.g., low bearing strength or high compressibility) is to simply replace the unfavorable soils. This method, known as over excavation and replacement, involves the removal of unsuitable soils and the subsequent replacement with more suitable fill material.

The fill material can be the same excavated material re-compacted to a satisfactory state or a select fill material transported from outside of the project site. Because of economic reasons, excavation and replacement depths are practically limited to approximately 2 m (7 ft) below the ground surface. [5]

2.5.2 High Energy Impact Compaction

Rolling Dynamic Compaction, otherwise known as High Energy Impact Compaction, is a relatively new technology and is becoming more popular because it is able to compact the ground more effectively, i.e. to greater depths than its static and vibrating roller counterparts, and more efficiently because of its greater speed – 12 km/h compared with 4 km/h using traditional rollers. It has been successfully implemented worldwide with different module designs having 3, 4, and 5 sides. [16]

High-impact energy impact compaction ground treatment imparts vertical energy into the ground to depths ranging from 2–5 m. In view of the near vertical energy input, the spread of energy along the ground surface as surface waves in minimized. It is the surface waves that generally cause the vibration to be transmitted along the ground to adjacent structures. [18]

Due to the combination of kinetic and potential energies, and the relatively large mass of the module, HEIC has demonstrated compactive effort to more than one meter below the ground surface (and more than 3 m in some soils) – far deeper than conventional static or vibratory rolling, which is generally limited to depths of less than 0.5 m. In addition, HEIC is unique in that it is able to compact large areas of open ground at depth, both effectively and efficiently. [16]

Figure 5: Three, Four and Five Sided Impact Rollers (Sources: M.B. Jaksa and et al [2012] and Landpac)

High energy impact compaction technique has been implemented in several projects with collapse prone subgrades. M. I, Pinard and S. Ookeditse (1988) has conducted trial impact compaction for the 215 km long road construction project Serowe-Orapa found 300 km north of Gaborone, Botswana, aiming at evaluating the operational effectiveness of impact compaction roller for the improvement of collapsible soils and determining its technical and economic viability vis-à-vis conventional compaction rollers.

In their trial, the compressibility characteristics of the sandy subgrade, after various increments of roller passes, were assessed from consolidation tests that were carried out on undisturbed samples, saturated at pressures of 36 kPa (approx. equivalent to the total of overburden, construction and traffic stresses) and 200 kPa. Typical results from these tests are shown in the following figure and clearly indicate that the collapse potential of the sand significantly reduced from its before compaction values of approx. 11% to about 3.7% and 2.0% after 30 and 60 passes respectively of the impact roller. [17]

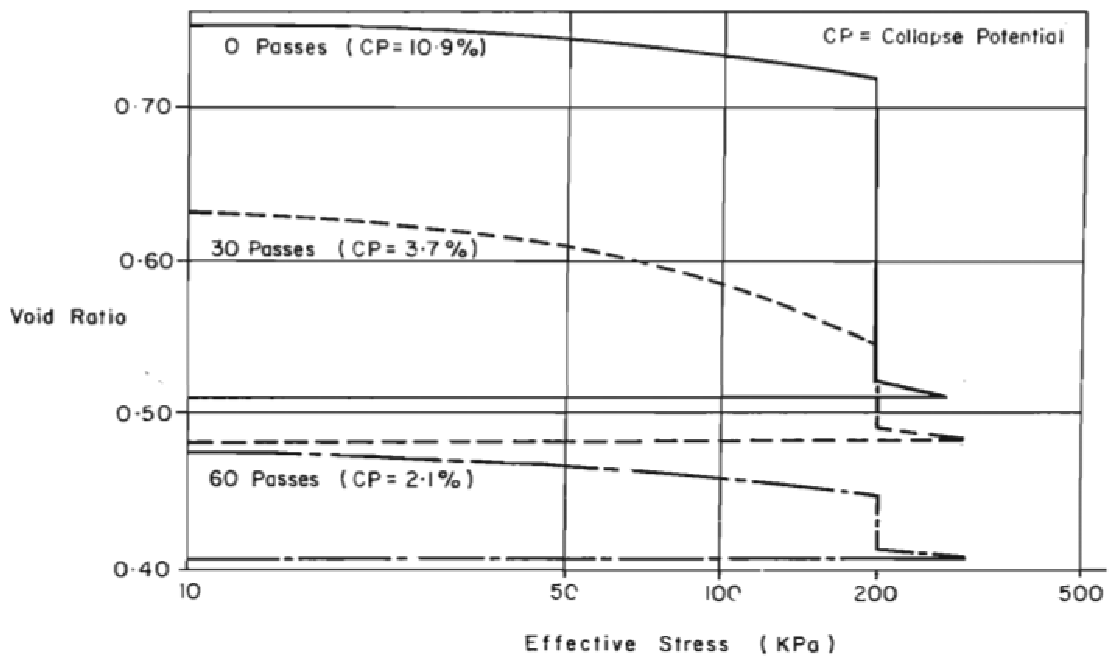

Figure 6: Collapse Potential Vs. Number of Passes (Source: M. I, Pinard and et al [1988])

Similarly, in the design and construction of Omo-F6 (Including Omo River Bridge) road project, a trial impact compaction has been conducted in order to improve the collapse nature of the soil found in the project's vicinity. After carrying out the trial works, the number of passes for the regular treatment was fixed to be 20. By adopting this number of pass, it was able to meet the specified quality standard viz. a maximum settlement of 30mm, a minimum degree of compaction of 90% and a minimum allowable bearing capacity of 120 kPa. [6]

2.5.3 Rapid Impact Compaction

Rapid Impact Compaction, was developed in early 1990's by British Sheet piling in Conjunction with British Army as an improvement on the process of Deep Dynamic Compaction. RIC is rapid, cost effective and can reach challenging locations. [14]

Rapid-impact compactor (RIC) is generally fitted to a tracked base excavator of 35–70 ton weight, which provides the dual benefit of allowing improved mobility and site accessibility.

Dynamic energy is imparted by dropping a weight by hydraulics from a controlled height onto a foot plate of 1 or 1.5 m^2 or a circular plate. The RIC impacts the soil at a rate of 10–60 blows per minute using a 5-, 7-, 9-, or 12-ton drop weight. The drop height varies from 1–2 m. Energy is transferred to the ground safely and efficiently as the RIC's foot remains in contact with the ground. No flying debris is ejected.

The compaction parameters (i.e., energy, blow counts, and soil penetration) are automatically controlled and monitored from the RIC's cab with an onboard data acquisition system. The rapid-impact compactor employs an onboard computer to control impact set termination criteria and to record critical data. Acquired data at each impact point include total energy input, total penetration, and penetration of final set. [5]

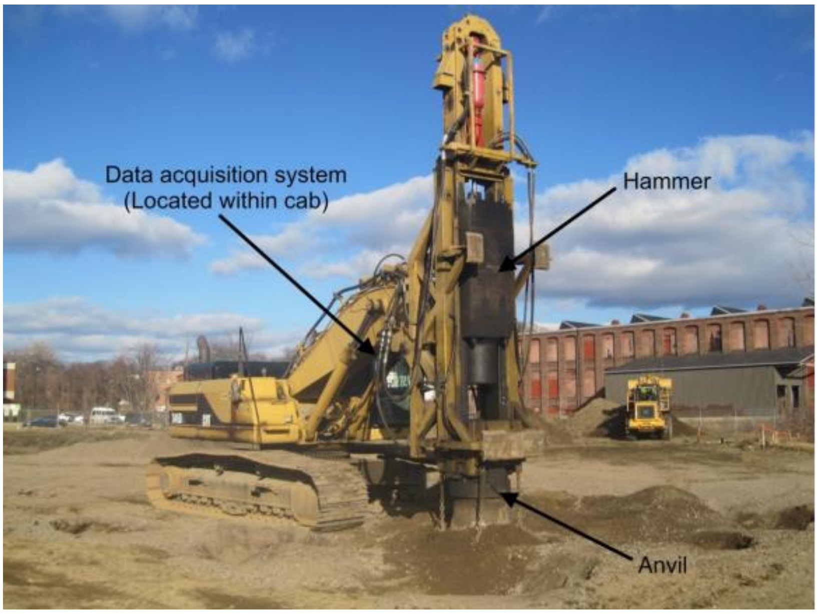

Figure 7: A Typical RIC Unit (Source: P.J. Becker [2011])

The compaction depth range of RIC generally depends on the properties of the soil to be compacted, the weight of the hammer and its drop height, the number of blows per impact point and the spacing of the impact points over the area being treated. Several researchers have reported that an influence/compaction depth range of 2 to 9m can be achieved using RIC.

Even tough RIC has been used in several land reclaiming projects, for liquefaction mitigation and for the construction projects of different plants and facilities, there are lack of case histories which involve RIC for use on the transportation infrastructure. However, P.J. Becker (2011) discussed that RIC is applicable for construction of new embankments and roadways over areas of unsuitable soils including collapsible soils, widening and expansion of existing roadways and embankments and improvement and stabilization of the support beneath the pavement structure. [5]

One of the projects in which RIC was used to treat collapsible soils is the construction of petrochemical processing plant in Karachaganak, Kazakhstan. This work was reported by C.J. Serridge & O. Synac (2006). The site was atop by loess extending to about 17 m below ground surface. Testing revealed that the upper 3.2 to 4 m of the loess soil profile had collapse potential.

The detail design of the project was to provide a required bearing capacity of 150 kPa with a long-term settlement requirement less than 25 mm for foundations not exceeding 10 m in width. In order to specify compaction variables, a trial compaction was done. After the variables were set, the main compaction work was commenced.

In-situ testing occurred before RIC treatment, in between RIC treatment passes and after RIC treatment. At each testing phase, dynamic probe tests (DPTs) were conducted to monitor the effectiveness of the treatment. After RIC treatment, plate bearing tests (PBTs) were conducted in addition to DPTs to more accurately appraise the bearing characteristics of the RIC treated soil.

Both the compaction trials and the main works verification testing showed that the RIC technique was successful in reducing collapse potential in loess soil. The recorded/observed depth of improvement was typically of the order of 3 m from the treatment commencement level, (i.e. from the base of the "crust", with level of improvement diminishing with depth).[7]

3. Description and Selection of the Study Area

3.1 Location

The Modjo - Hawassa road project (approximately 202 km long) is part of the Cape Town to Cairo road corridor connecting Southern, Eastern and Northern Africa. The road section is divided into four lots and is designed to a four lane dual carriageway highway standard with an area separated median 9.0 meter wide (swale ditch) and will have controlled access with grade separated intersections to establish links with the existing roads.

Among the other lots, Lot -3 has a total length of ±57.1 km and is located entirely in Oromia National Regional State in the southern part of the country. The route starts west side of Batu (Zeway) town at km 92+896 and ends at km 150+000 before Arsi Negele town.

Figure 8: Project Location (Source: ERA 2013a)

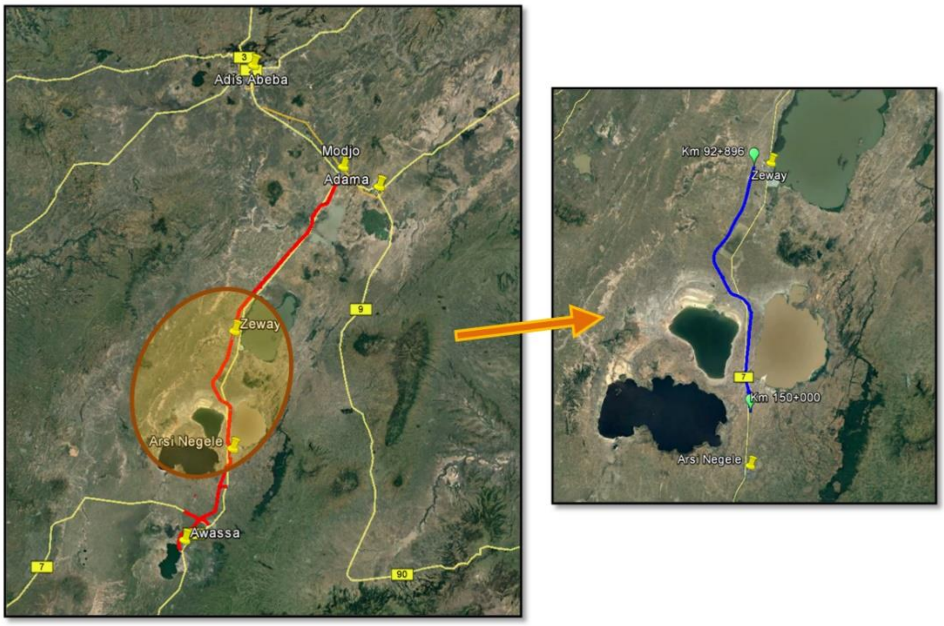

Figure 9: Arial View of the Project's Location

3.2 Temperature

The climate of the project area characteristically falls within the mid-altitude climatic division (1500 – 2200 m.a.s.l.), locally known as 'Weina Dega' agro-climatic zone. The mean daily temperature ranges between 10°C - 26°C with maximum shading temperature reaching 32°C in the hottest months of January and May. The area generally records mean daily temperature of 20°C and shows very little variation throughout the year. On the other hand, monthly night temperature commonly falls below 11°C. The lowest temperature occurs during the main rainy season.[25]

3.3 Rainfall

The project area is found in the axial zone of the central sector of the Main Ethiopian Rift (MER) in the eastern Arsi zone of the Oromia Regional. The average annual rainfall ranges from about 1150mm in the eastern and western high lands to about 650mm in the rift floor. This indicates that the amount of rain fall is controlled by altitude. Most of the precipitation occurs between the months of June and mid-September with short rainfall in March and April. The rift valley is characterized by relatively high evapotranspiration with an average value of 1200mm indicating water deficiency in general. [25]

The driest period of the year falls within the months of November, December and January, when the prevailing winds are the northeast trades, bringing dry air from central Asia. The months of October to March are relatively dry & rather windy. [25]

3.4 Geology

The route alignment passes through the central part of the Cenozoic rift zone where the four of main rift valley lakes, namely Ziway, Langano Abijata and Shala are situated at closer distances. The lakes were considered to have been once connected forming one great lake in the geologic past.

The area is covered by a variety of lithologies representing bimodal volcanic rocks and volcano-clastic sedimentary deposits. As seen on Figure 10, the road generally traverses through sandy silt/clay (Qld), sandy silt (Qsd) and welded tuff (Nwt) lithostratigraphic units.

Grey to grayish white and brown Sandy Silt/Clay (Qld) is the most extensive unit occupying the plain land bounded to the east and west by high rising mountains and ridge line. Good outcrops are mainly found along the banks of the Bulbula River. Elsewhere, it is covered by a veneer of dark grey Sandy Silt/Clay. [25]

The unit is marked by an alternation of loose to moderately compacted volcanoclastic deposits that are of few centimeters to a meter thick. The layers appear in different shades of grey and grayish white and show textural variations from fine mud to coarse sand. In some places there are also layers containing reworked pumice. Greyish white diatomaceous ash is another component of the deposit on the beaches of Lake Abijata. It is being mined for production of soda ash at the factory site few km west of Bulbula town. [25]

The project area is also covered by grey to dark grey sandy silt and gravely sand (Qsd). This unit represents loose material deposited alongside of a stream channel in the northwest corner of the area and small valley in the southeast. It is composed of very loose silt and sand with various grey shades and rarely shows brown colors. [25]

The area at the project end consists of Ignimbrite and partially to moderately welded tuff lithostratigraphic unit (Nwt) which is part of the oldest rift sequence also known as Nazreth Group.

Ground water was not encountered within the investigation depths, except for investigation points located between km 118+000 to km 130+000. [24]

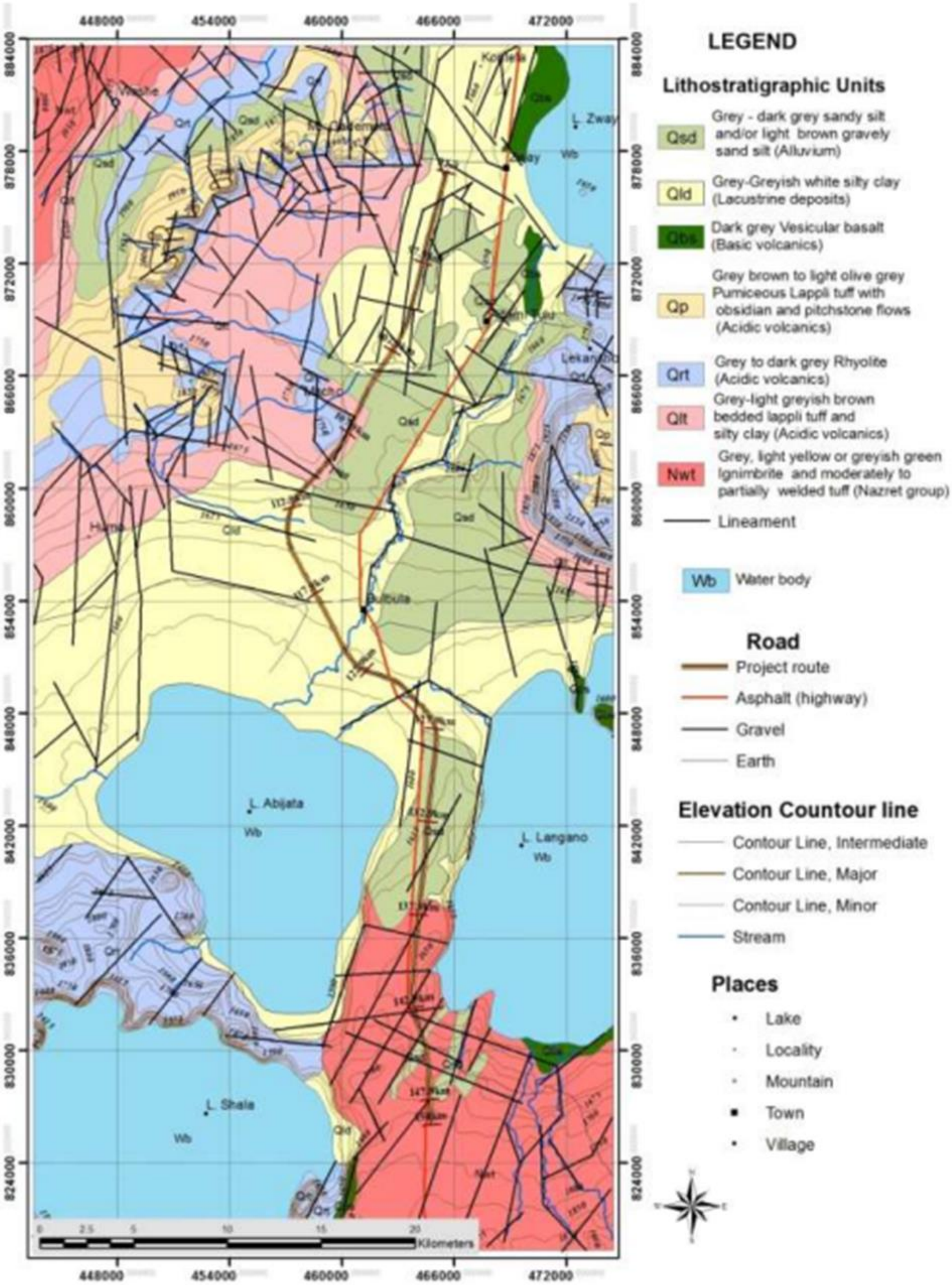

Figure 10: Geological Map of the Area (Source: Site Investigation of the project [2018])

3.5 Collapsible Soils in the Project's Vicinity

As part of the subgrade investigation of the project and in accordance with ASTM D5333, tests to determine collapse index (determined at a stress level of 200 kPa) and collapse potential (determined at stress level other than 200 kPa) were conducted by the project's contractor. Such distinction in terms of stress levels has been made by taking in to account that ASTM was originally developed for buildings, and somewhat different stress levels are expected from embankments, pavement and surcharge loads at highway subgrades.

Totally, around one hundred samples were tested at different stress levels. The collapse tests were carried out using undisturbed samples taken from bore holes drilled for investigation purposes. The collapse test results against chainage and depth are tabulated on Table 2.

As seen on the table, there are slight variations on collapse severity for samples taken at the same location but at different depths. This may be accounted to the geological formation of the area. As discussed on section 3.4, the soil formations in the area are majorly of volcanic and sedimentary by nature, the latter being deposited from lake. The materials deposited from the lakes are brought from different places via the streams in the vicinity, hence, they are very erratic and varying in nature.

In addition, all the tests were done based on remolded samples. The process of remolding the samples to their in-situ density is subjected to errors, as a very small loss (in grams of the sample to be remolded to the consolidation ring) can have a significant effect on the collapse potential of the same.

Therefore, the variability in nature and the layered formation of the sedimentary soils due to slow process of deposition together with the error that may arise while remolding the soil samples, may have resulted in the disparities of their collapse nature.

Table 2: Collapse Index/Potential vs. Chainage (Source: Earthworks-Geotechnical Design and Recommendation report of the project [2018])

Station	Sampling Depth			Collapse Index/Potential with Varying Stresses				
	From	To	Avg.	100	200	300	400	800
093+440	1.50	1.78	1.64		0.23			
093+440	3.00	3.20	3.10		0.48			
093+560	1.50	1.70	1.60	4.21				
093+560	3.00	3.25	3.13		4.52			
093+680	1.50	1.70	1.60		1.36			
093+680	3.00	3.15	3.08		1.58	2.61		
093+920	1.50	1.75	1.63		1.94			
094+040	1.50	1.90	1.70		2.94			
094+280	1.50	1.85	1.68		2.05			
094+280	3.00	3.40	3.20		2.03			
094+280	8.20	8.50	8.35		2.71	3.33		
094+400	3.00	3.17	3.09		0.28			
094+520	1.50	1.80	1.65		2.42			
095+080	4.50	5.00	4.75		0.34			
095+200	1.50	1.90	1.70		0.88			
095+200	6.00	6.15	6.07		1.69			
095+440	2.00	2.35	2.18		1.31			
095+560	3.00	3.30	3.15		2.45			
095+680	1.50	1.75	1.63		1.55			
095+680	3.00	3.35	3.18		1.19			
095+680	6.00	6.40	6.20		3.45			
095+800	4.50	4.80	4.65		3.04			
095+920	1.50	1.70	1.60		0.56			
095+920	3.00	3.25	3.13		0.42			
096+540	4.50	4.73	4.62		2.02			
096+660	2.50	2.68	2.59	2.15	3.80			
096+660	5.50	5.85	5.68	2.32	4.28			
096+780	1.50	1.70	1.60	3.93	6.25			
096+780	3.00	3.25	3.13		0.76			
096+780	8.00	8.20	8.10		1.70			
096+900	1.50	1.80	1.65		3.26			
097+020	3.00	3.25	3.13		1.24			
098+260	1.50	1.80	1.65		0.79			
099+420	4.00	4.18	4.09		1.11			
099+650	1.50	1.85	1.68		0.65			
102+940	3.00	3.25	3.13		1.32			
104+540	1.50	1.90	1.70		1.20			
104+540	4.50	4.80	4.65		0.97			
105+960	3.00	3.30	3.15		2.04			
107+680	4.50	4.73	4.62		2.76			

Station	Sampling Depth			Collapse Index/Potential with Varying Stresses				
	From	To	Avg.	100	200	300	400	800
108+480	1.50	2.00	1.75	1.64				
108+480	7.50	7.70	7.60	0.26				
109+280	3.00	3.40	3.20		3.24			
110+060	1.50	2.00	1.75	1.78				
110+420	4.50	4.85	4.68		0.79			
110+600	2.00	2.50	2.25	0.18				
111+500	3.00	3.50	3.25		1.01			
115+700	1.50	1.65	1.58		2.55			
117+800	3.00	3.25	3.13		1.75			
126+900	1.5	1.75	1.63		0.11			
126+900	4.5	4.71	4.61		0.49			
129+000	1	1.2	1.10		0.87			
129+000	2.5	2.75	2.63		0.76			
129+000	4	4.37	4.19		1.61			
129+000	5.5	5.66	5.58		2.38			
131+900	1.5	1.75	1.63		2.38			
131+900	3	3.3	3.15		0.04			
131+900	4.5	4.87	4.69		0.14			
131+900	6	6.2	6.10		0.99			
133+380	1	1.33	1.18		0.29			
135+780	4.50	4.70	4.60					2.44
136+020	3.3	3.5	3.40		0.15			
136+020	4.5	4.8	4.65		1.32			
136+020	6	6.3	6.15		0.33			
137+600	5.00	5.35	5.18		2.43			
138+000	10.00	10.85	10.43		0.23			
138+000	1.50	1.75	1.63	0.35				
139+196	2.50	2.75	2.63					1.14
142+300	3.00	3.25	3.13		0.70			
142+300	12.00	12.15	12.08		5.65			
142+300	7.50	7.70	7.60		1.37			
142+300	10.50	10.85	10.68		2.25			
143+080	2.50	2.80	2.65			0.79		
144+080	8.00	8.15	8.08					1.70
144+080	4.50	4.75	4.63			0.92		
144+080	8.00	8.14	8.07				1.77	
145+806	3.50	3.75	3.63		1.06			
145+806	3.50	3.70	3.60		7.26			
145+806	6.20	6.55	6.38		7.41			
145+806	10.50	10.70	10.60		0.60			
147+219	2.50	3.00	2.75		3.16			

Station	Sampling Depth			Collapse Index/Potential with Varying Stresses				
	From	To	Avg.	100	200	300	400	800
147+219	4.50	4.65	4.58		2.23			
147+219	4.50	4.70	4.60		0.48			
147+219	6.00	6.20	6.10		5.41			
147+219	10.50	10.70	10.6		1.98			
147+219	2.50	3.00	2.75		1.79			
147+219	4.50	4.80	4.65		0.57			
147+219	10.50	10.65	10.58		0.98			
148+052	6.00	6.50	6.25		0.40			
148+052	8.00	8.20	8.10		0.64			
148+052	4.00	4.30	4.15		2.02			
148+052	6.00	6.35	6.18		1.67			
148+600	3.00	3.35	3.18		1.74			
148+900	4.50	4.75	4.63		4.97			
148+900	3.00	3.20	3.10	2.99				
148+900	1.50	1.75	1.63	0.75				

The index as well as the collapse potential laboratory test results of the samples are attached on Annex B. From the test results, it can be observed that 48.4% and 50% of the samples are classified as SM and ML in accordance with USCS soil classification system respectively. As per AASHTO classification system, 71% and 19.2%of the samples were classified as A4 and A-2-4 respectively.

Furthermore, based on the Atterberg limits, 91.2% of the samples did not show plastic property; and hence, were classified as non-plastic (NP). In addition, the soil samples had dry densities ranging from 0.437g/cc to 1.404 g/cc.

The collapse potential tests conducted as per ASTM D5333 also showed that more than 39% of the samples have moderately and moderately severe collapse potential values. The maximum collapse value recorded is 7.41% at station km 145+806.

According to Table 2, although there are only three points with moderate collapse indices between stations km 108+00 to km 116+500, two zones can be defined for which many of the moderate and moderately severe values are highly concentrated and the soils are more prone to collapse. These zones are identified between km 93+000 to km 97+500 and between km 144+500 to km 149+500.

3.6 Selection of the Study Area

The design and construction of Mojo-Hawassa Dual Carriageway Highway Project, after its completion, is expected to improve the connectivity of high potential farming and tourism areas, while consolidating the Corridor Addis Ababa-Moyale Nairobi-Mombasa, of strategic importance to diversify Ethiopia's international links and sea outlets.

Such crucial scope of the road project, makes it very critical and sensitive, among any other projects. Therefore, a better and an effective way to treat collapsible soils should be proposed so that the construction of the road project can safely serve its purpose and in order to protect this multi-million dollar project from any failure that may arise because of collapsible soils.

4. Materials and Testing Methods

4.1. Materials

Collapsible soil samples were taken by excavating two pits manually using pick axe and shovel at stations km 93+560 and km 96+660 at the centerline of the road alignment of the Modjo-Hawassa Design and Build Highway Project, Lot-3; Zeway-Arsi Negele Section.

The test pits had dimensions of 1 m x 1 m x 1 m (lengthxwidthxdepth). The selection of sampling depth for collapsible soils need not be limited to the influence depth of the traffic load (which is usually thought to be with in 1.5m depth below the pavement, as per ERA's 2013 Site Investigation Manual) as done to investigate the bearing capacity of subgrade soils under a road. This is due to the fact that a collapse occurring well below the suggested influence depth can trigger settlements on the pavement layers. Therefore, collapsible soils have to be investigated to the extent of their existence, regardless of the traffic load influence depth. For this work, due to economic reasons, the investigation depth is limited to 1 m below the natural subgrade level.

Representative disturbed samples for soil classification, establishing moisture-density relationship and collapse potential test have been collected. Disturbed soil samples were collected for the purpose of conducting tests mainly due to the fact that arise from the difficulty of obtaining undisturbed soil samples, since the soils in the test pits area are mostly made up of sand and silt particles and lack cohesive nature.

Field visual identification of the soils was made based on color, texture as well as on the assessment of their plasticity, and fine content and coarse fractions. In addition to that in situ density tests have been performed before the test pits are back filled compacted and reinstated. The position of each test pit was located by hand held GPS.

Then after, the samples to be used for conducting natural moisture content test were sealed tightly in a moisture bag and the remaining samples to be used for other classification tests were put into sacks and all the samples were transported to Gondwana Engineering Laboratory for conducting the necessary tests. Table 3 shows the location of the sampling points.

Table 3: Coordinated of Sampling Points

Sampling Station	Easting	Northing	Depth of Sampling
93+560	465466	876494	1 m
96+660	464632	873509	1 m

Figure 11: Test pits dug at km 93+560 and km 96+660

The sampling stations were selected by carefully studying the collapse potential test results summarized under Table 2. As indicated in that section, two zones are defined in which soils are more prone to collapse. These zones are identified between km. 93+000 to km 97+500, and between km 144+500 to km 149+500.

In the first zone, it can be observed that most of the highest values of the collapse potential are found slightly above 4%. There is, however, one test result with a collapse potential of 6.25% i.e. at km 96+780 sampled at a depth of 1.50-1.70 m. This value is considered as an outliner because there are no other consistent data to support the same. Even for samples taken at the same location but at depth 3 m and

8 m have a collapse potential of 0.76 % and 1.70 % respectively. Such discrepancy in test results may be attributed to the erratic nature of the soil at the sampling area and errors during sampling and handling undisturbed soil samples. The detailed test results are incorporated on Appendix B. Therefore, this station was not selected to take the sample for this thesis.

Hence, two station with the second and third highest collapse potential (CP) values viz. km 93+560 with CP 4.52% and km 96+660 with CP 4.28% were selected to take samples for this thesis.

The second zone i.e. from km 144+500 to km 149+500 was not considered to take samples for this thesis as the highest value of collapse were obtained at relatively greater depths. For example, for the borehole located at km 145+806, a CP of 7.26% and 7.41% were obtained at average depths of 3.6 m and 6.38 m respectively. And for borehole located at km 147+219, a CP of 5.41% and was obtained at an average depth of 6.1 m.

Since the highest CP values obtained at the first zone are located at shallower depths, the samples taken for this thesis were obtained from km 93+560 and km 96+660 as discussed above. All the collapse potential test results done as per ASTM D5333 for the project are included in the annexure.

4.2 Testing Methods

4.2.1 Field Testing

As shown on Figure 12, in order to acquire the in-situ state of compaction, in-place density tests have been conducted inside the 1 m deep test pits by sand replacement method in accordance with AASHTO T-191 (1993).

The material from the hole to a depth of 150 mm was carefully collected in a polyethylene bag, weighed, tightly sealed, and labeled for subsequent natural moisture content determination. Dry, free-flowing sand of known density was then poured into the hole from the standard sand cone apparatus. From the weight of the sand the volume of the hole was determined.

Figure 12: In-Situ Density Determination

4.2.2 Laboratory Testing

Representative disturbed soil samples collected from the test pits were safely transported to Gondwana Engineering Plc laboratory for testing. The laboratory testing for soil samples included moisture content, Atterberg Limits, sieve analyses, specific gravity and proctor compaction tests. Each soil sample is classified on the basis of texture and plasticity in accordance with the AASHTO and Unified Soil Classification Systems (USCS). The standards used to conduct the laboratory tests are summarized in Table 4.

Table 4: Standards Adopted for Laboratory Tests

It. No.	Laboratory Test Type	Standard
1	Natural Moisture Content	AASHTO T-265
2	Atterberg Limits	AASHTO T89/T90
3	Grain Size Analysis	ASTM D422
4	Specific Gravity	ASTM D854-00
5	Moisture-Density Relationship	AASHTO T-180

Further to the above-mentioned laboratory tests, as per ASTM D5333, one dimensional Oedometer test has been conducted in order to determine collapse index and potential on remolded soil samples. The collapse index tests conducted at a stress level of 200 kPa, were carried out on two conditions.

The first one is using the in-situ bulk density obtained from sand replacement method, in order to simulate the collapse of the sample at its natural moisture and density conditions. The second one is using the maximum dry density and optimum moisture content obtained from proctor compaction test, in order to observe the response of the sample when subjected to inundating water and stress at its compacted state.

The collapse potential tests were carried out based on the stress level expected to be encountered at sampling locations. The stress level to be applied in the tests was calculating by considering the overburden stress of the subgrade above the sampling point, the embankment and the pavement loads and the surcharge load from the traffic. The calculation is shown on Appendix A. The collapse potential test was also carried out using both bulk and maximum dry densities. All the loading scheme are shown on Table 5.

Table 5: Loading Conditions for Collapse Potential Tests

Station	Stress_Condition	Remolding Density	Remolding Moisture
93+560	At 200 kPa	Bulk	NMC
		MDD	OMC
	At Over Burden Stress (67 kPa)	Bulk	NMC
		MDD	OMC
96+660	At 200 kPa	Bulk	NMC
		MDD	OMC
	At Over Burden Stress (75 kPa)	Bulk	NMC
		MDD	OMC

5. Results and Discussions

As discussed in the previous chapter, samples were collected from two test pits located at km 93+560 and km 96+660 at a depth of 1 m below the existing ground level. Measurements of field densities were conducted on site. Furthermore, disturbed samples were extracted and transported to Gondwana Engineering to carry out the relevant laboratory tests. In this chapter the test results obtained from field and laboratory tests are discussed in detail.

5.1 Field Density

The in-situ bulk density characteristics of the subgrade at the sampling locations were investigated by carrying out density test based on the testing procedure for density of soils using sand replacement method in accordance with AASHTO T-191 (1993). In addition, the natural moisture contents of the soil samples were determined as per AASHTO T-265. These moisture content values were used in order to calculate the field dry densities. The results of these tests are shown on Table 6.

Table 6: Field Density Test Results

Station	Bulk Density (g/cc)	NMC (%)	Dry Density (g/cc)
93+560	**1.24**	**17.27**	**1.06**
96+660	**1.18**	**20.92**	**0.98**

As it is can easily been observed from the results, the subgrade soils have very low bulk as well as dry densities. Both densities from the two test pits are below the suggested margins of Clevenger (1958)-1.28 g/cc, which indicate, as discussed in the literature review, that the soil is susceptible to significant settlement.

Similarly, based on ERA's 2013 Geotechnical Design Manual, the densities are below 1.6 g/cc, which makes them, as per the manual, prone to collapse.

5.2 Specific Gravity

Specific gravity tests were conducted on the samples collected from both test pits. The specific gravities of the samples are 2.51 and 2.56 for test pits located at km 93+560 and km 96+660 respectively.

Specific gravity of a soil gives indication on the mineral type it composes. T.W. Lambe and R. V. Whitman (1969) has presented typical specific gravity values for different soil mineral types.[26] Based on the typical values, it can be implied that the soil samples are most likely made up of potassium feldspars. Feldspars, according to R.C. Makenzie and J. E. Gieseking (1975), are known to be a major constitutes of sand and silt fractions of soils. [20] Therefore, it can be concluded that the obtained specific gravity values of the soil samples show that they possess sandy and silty natures.

Figure 13: Specific Gravity Test on Progress

5.3 Grain Size Analysis

Wet sieve and Hydrometer analysis tests were conducted to determine the percentage of different grain sizes contained within a soil. Wet sieve analysis was performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method was used to determine the distribution of the finer particles. The test results show that percentage passing sieve no. 200 are 62% and

64.6% for samples collected from test pits at km 93+560 and km 96+660 respectively. The soils are also classified as A4(6) and A4(7), on their respective order as per AASHTO soil classification system. In addition, both soils are categorized as ML (low plastic silt) based on USCS soil classification system. The particle size distribution shows the following compositions.

Figure 14: Sieve and Hydrometer Test on Progress

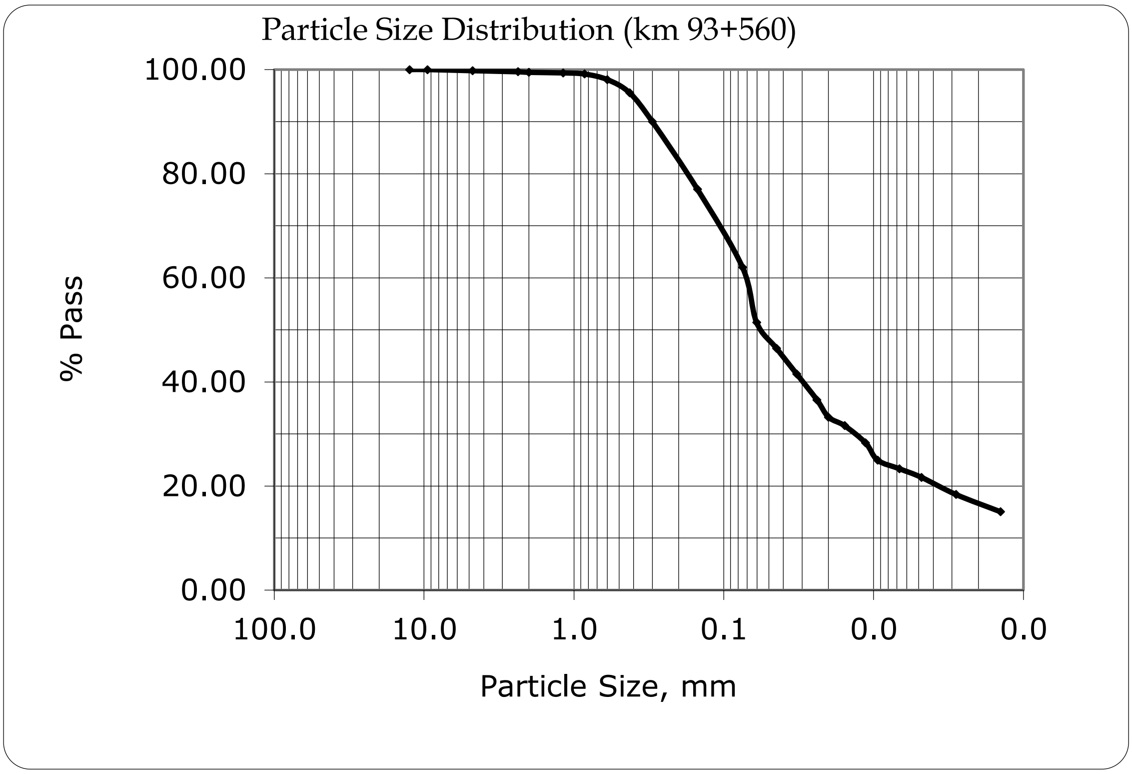

Figure 15: Particle Size Distribution (km 93+560)

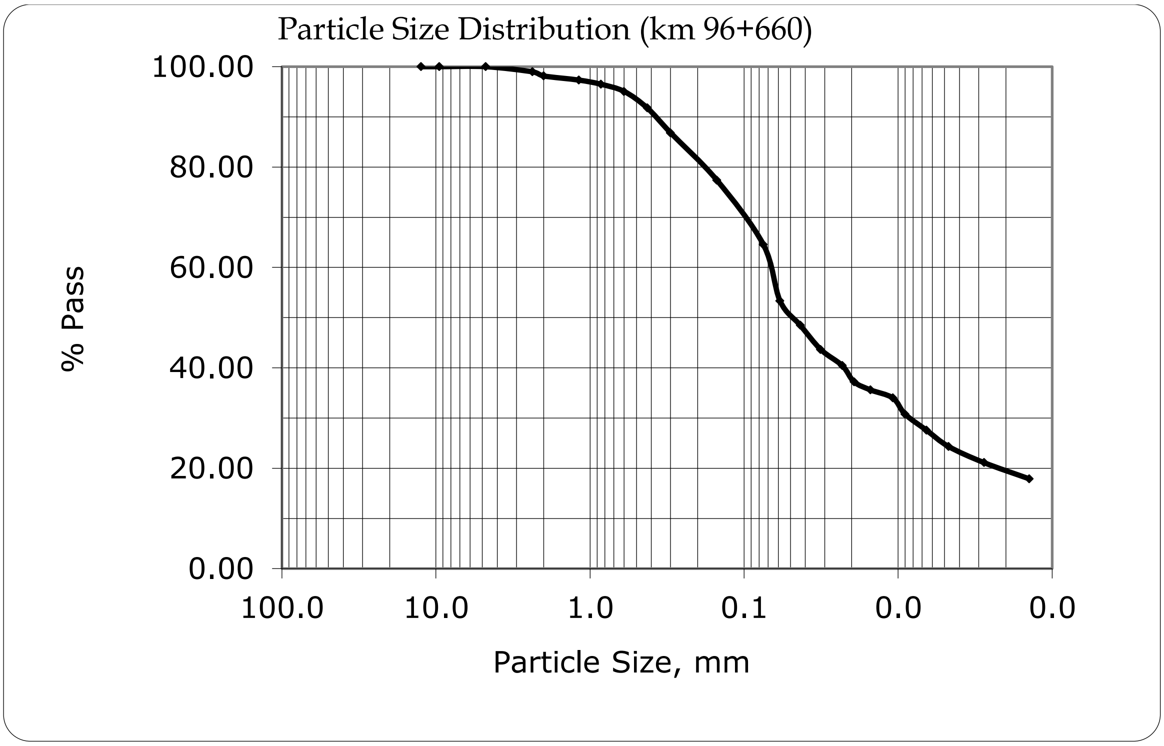

Figure 16: Particle Size Distribution (km 96+660)

Table 7: Soil Type Composition in Percentage (km 93+560)

Station	Depth Sampled(m)	Gravel % (>2.0mm)	Sand % (2.0mm-0.06mm)	Silt% (0.06mm-0.002mm)	Clay %(<0.002mm)	Remark
93+560	1m	0.53	48.0	34.72	16.7	Silty Sand Soil

Table 8: Soil Type Composition in Percentage (km 96+660)

Station	Depth Sampled(m)	Gravel % (>2.0mm)	Sand % (2.0mm-0.06mm)	Silt% (0.06mm-0.002mm)	Clay %(<0.002mm)	Remark
96+660	1m	1.85	44.8	33.82	19.5	Silty Sand Soil

As it was mentioned in the previous chapters and as discussed also in many literatures, including ERA's 2013 Site Investigation Manual, collapsible soils mainly consist of sands and silts with a trace amount of clay content. The same trend has been observed in the particle size analysis of both samples.

5.4 Atterberg Limits

Atterberg limits were determined from the soil samples. The results of the Atterberg limit of the soil samples are presented in Figures 17 and 18 and in Table 9.

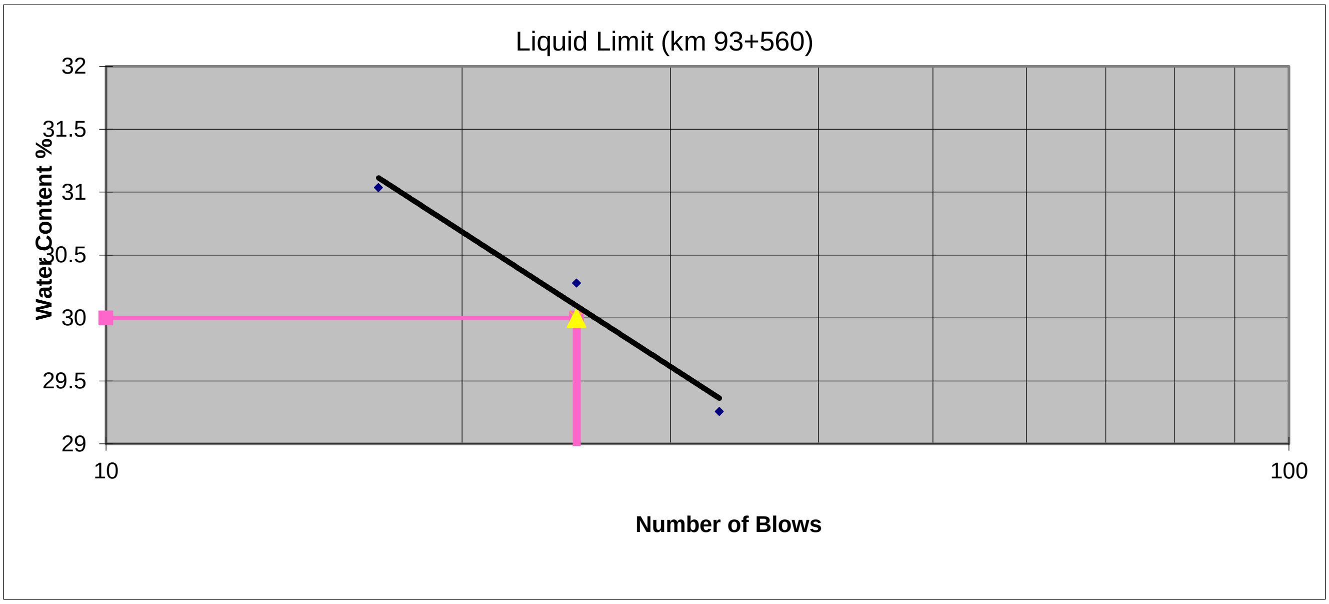

Figure 17: Liquid Limit (km 93+560)

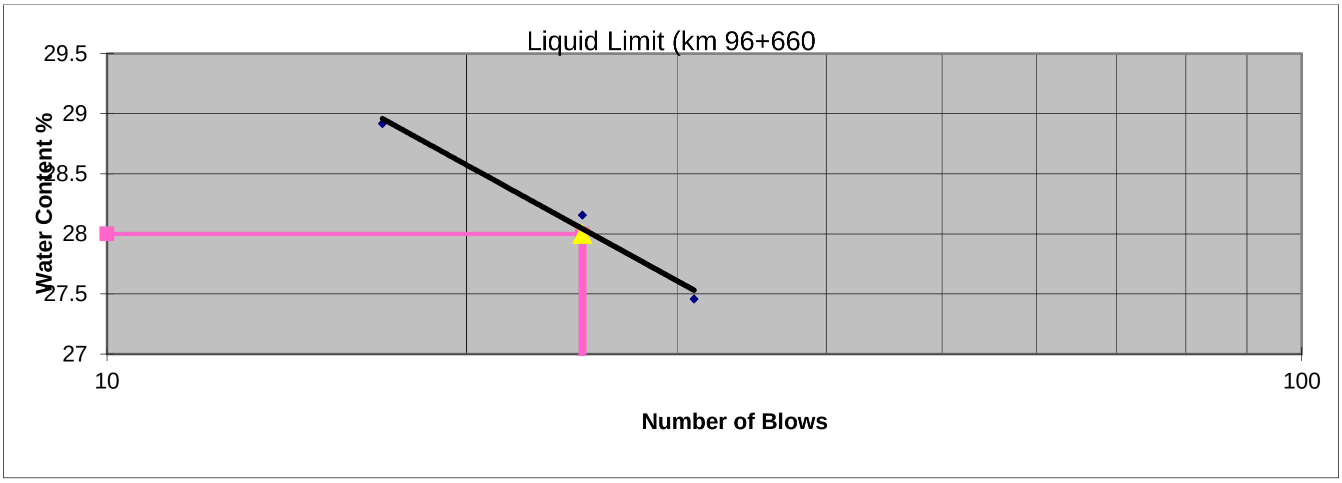

Figure 18: Liquid Limit (km 96+660)

Table 9: Atterberg Limits

Station	Liquid Limit (LL), %	Plastic Limit (PL), %	Plasticity Index (PI)
93+560	30	27	3
96+660	28	26	2

Using dry density values from field tests (Table 6) and Atterberg limits (Table 9) it can be observed that for both stations, the soils fall under the "potentially metastable" portion of the plot suggested by Bell and Bruyn (1997) (Figure 3).

5.5 Moisture-Density Relationship

Modified Proctor tests were conducted on the collected soil samples to determine the relationship between the moisture content and dry densities. The results of the maximum dry densities and the respective optimum moisture contents of the samples of soil is presented in Table 10. Furthermore, the degree of compaction of the soils have been included by making comparison to the in-situ bulk density obtained from sand replacement method and the maximum dry densities.

Table 10: Moisture-Density Relationship Test Results

Station	In-situ Dry Densities (g/cc)	MDD (g/cc)	OMC (%)	Degree of Compaction (%)
93+560	1.06	1.38	24.40	76.81
96+660	0.98	1.24	26.20	79.03

5.6 Collapse Index/Potential

In order to determine the collapse property of the soil, one dimensional Oedometer test have been carried out in accordance with testing procedures described in ASTM D 5333. The testing scheme was as described in section 4. All the tests were conducted on remolded samples due to the difficulty of obtaining undisturbed block sample of the soils due to their sandy and silty natures. The test results obtained are discusses as follows.

5.6.1 Collapse Index at In-Situ Bulk and Maximum Dry Density

The first attempt towards investigating the collapse property of the soil under investigation was concentrated in determining the collapse index at a stress level of 200 kPa, as per recommendation on ASTM D5333, at their in-situ bulk densities and natural moisture contents. Then, the collapse indices were also determined at the samples' maximum dry densities and optimum moisture contents. The results are depicted on Figures 19-22.

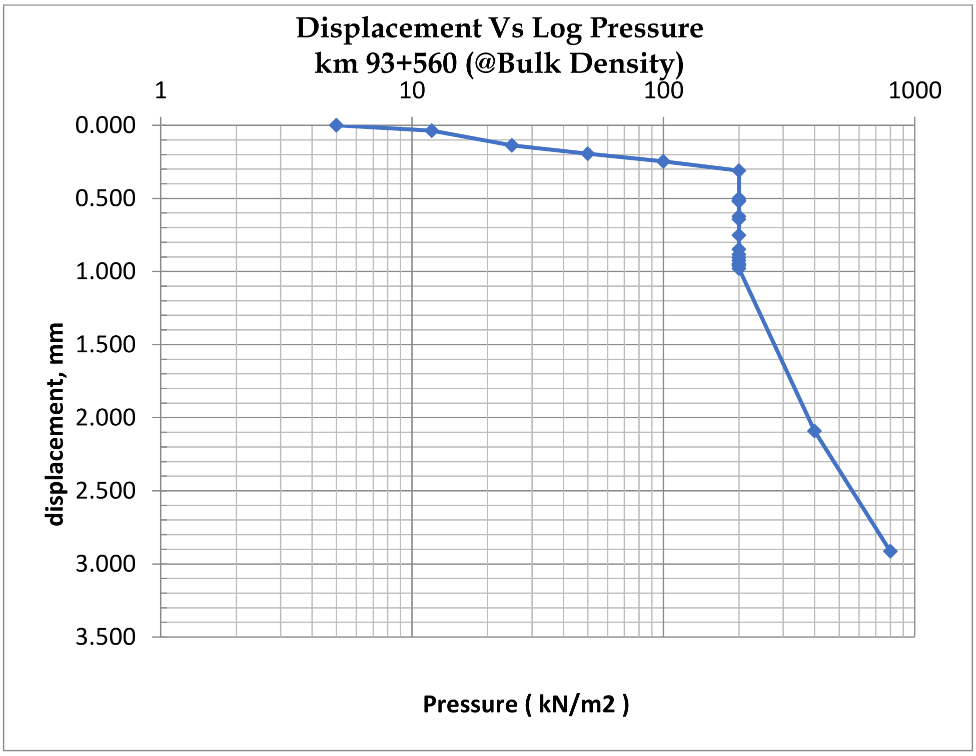

Figure 19: Collapse Index for km 93+560 (@Bulk Density)

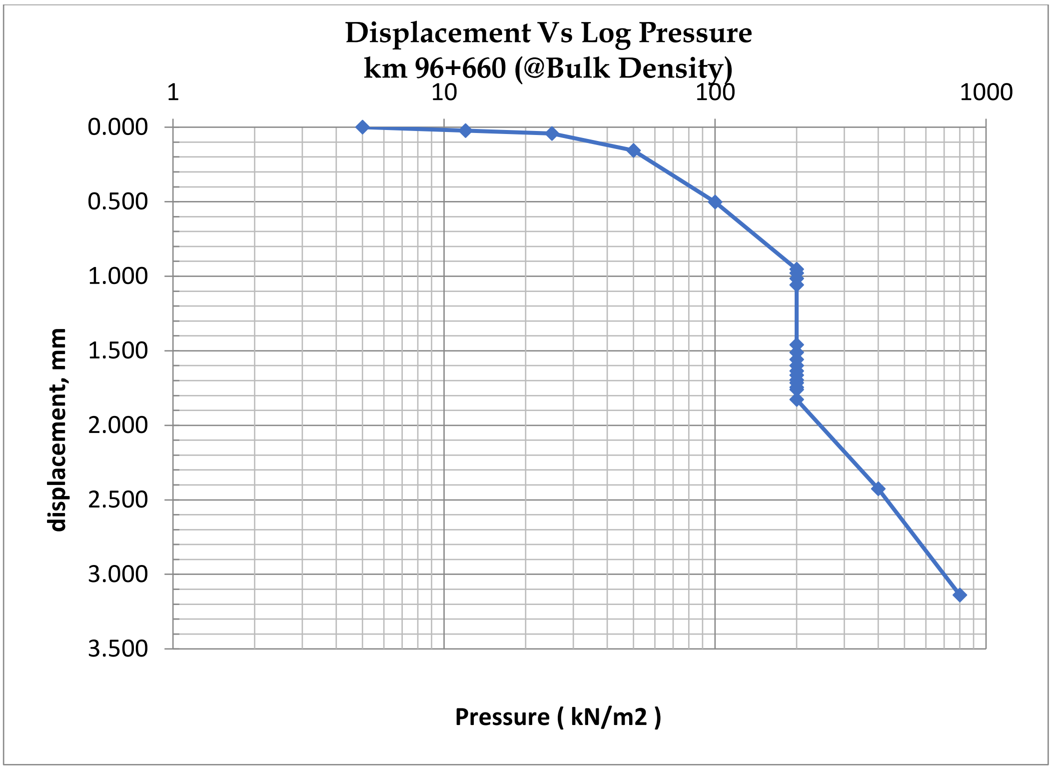

Figure 20: Collapse Index for km 96+660 (@Bulk Density)

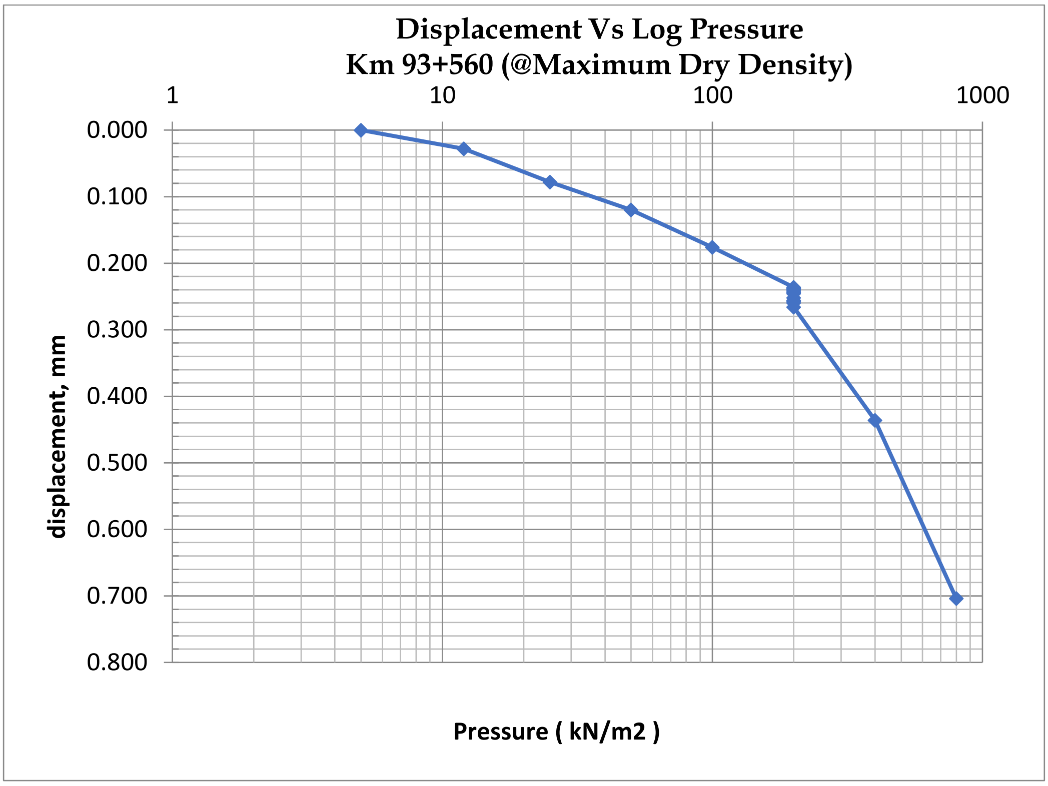

Figure 21: Collapse Index for km 93+560 (@MDD)

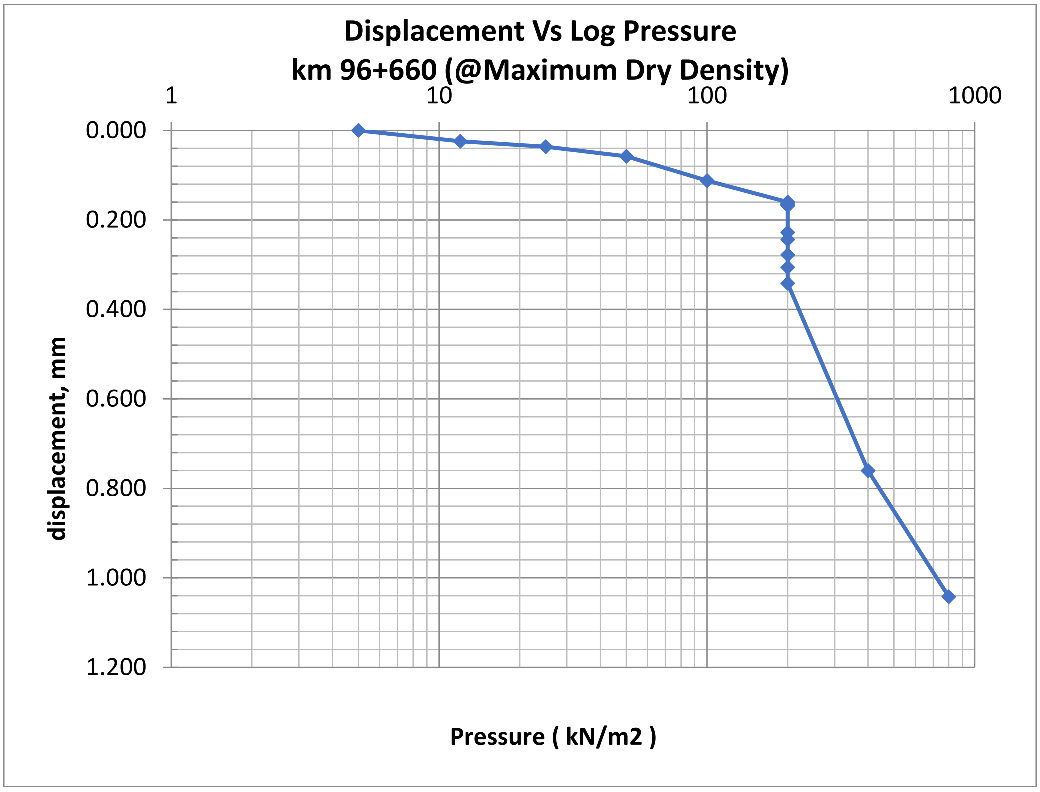

Figure 22: Collapse Index for km 96+660 (@MDD)

The obtained test results were checked against the collapse severity as specified by ASTM D5333. The collapse index ratings are tabulated in Table 11.

Table 11: Collapse Index Rating as per ASTM D5333

Station	Remolding State	Density (g/cc)	Collapse Index Value (%)	Rating as per ASTM D5333
93+560	Bulk	1.24	3.35	Moderate
	MDD	1.38	0.15	Slight
96+660	Bulk	1.18	4.37	Moderate
	MDD	1.24	0.91	Slight

As it is shown on Table 11, the classification of the collapse indices was moderate when the samples are remolded at their bulk densities. However, when the samples were remolded at their maximum dry densities, the collapse indices for both stations were significantly reduced. This is a clear indication that the property of collapsible soils can be highly improved by compaction.

5.6.2 Collapse Potential at In-Situ Bulk and Maximum Dry Density

A discussion has been made in the literature review that, ASTM defines two parameters regarding the collapse property of soils viz. collapse index (conducted at a stress level of 200 kPa) and collapse potential (conducted at the actual stress level the soil is anticipated to be subjected to).

Among the two parameters, collapse potential suits best to show the collapse nature of subgrade soils for highway projects. This is mainly due to the fact that soil specimen shall be loaded, during the one-dimensional Oedometer collapse potential test, at the actual anticipated stress in which the subgrade soils will incur. For highway subgrade soils these stress levels are calculated by taking into account the overburden pressure coming from the load from vehicles, the pavement, the embankment, and the subgrade stress (from top of subgrade to the depth of sampling). Detail calculation of the stress levels is attached in the Appendix A.

To this regard, the samples were again tested by subjecting them to the actual anticipated stress level. The test results of the collapse potential values are shown on Figures 23-26.

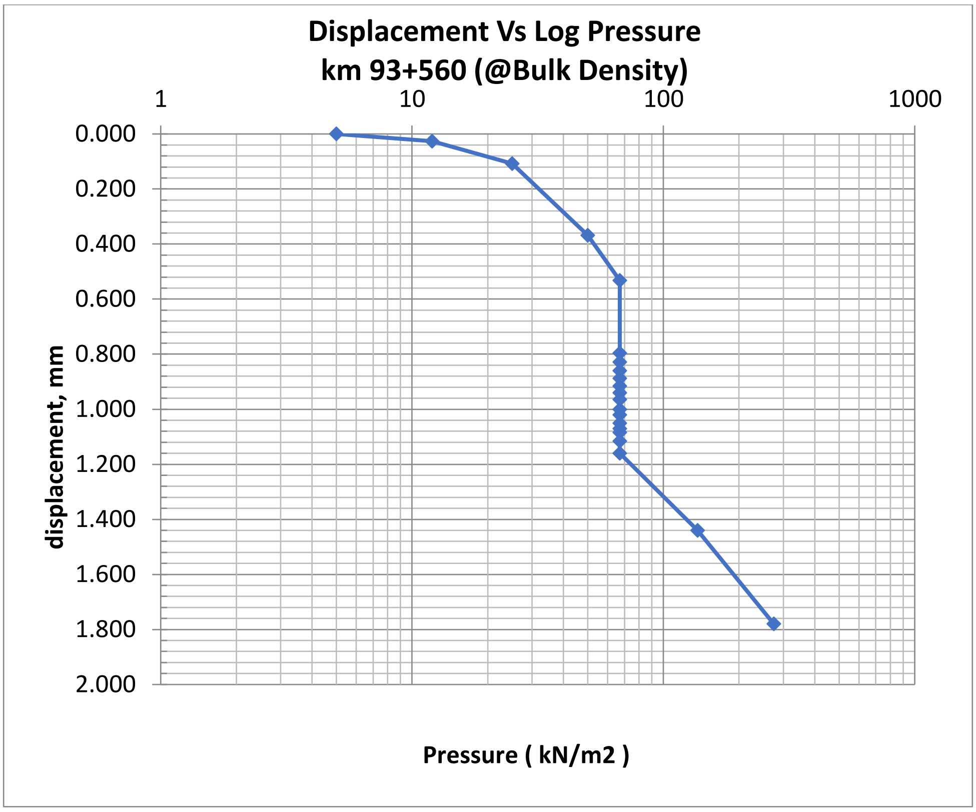

Figure 23: Collapse Potential for km 93+560 (@Bulk Density)

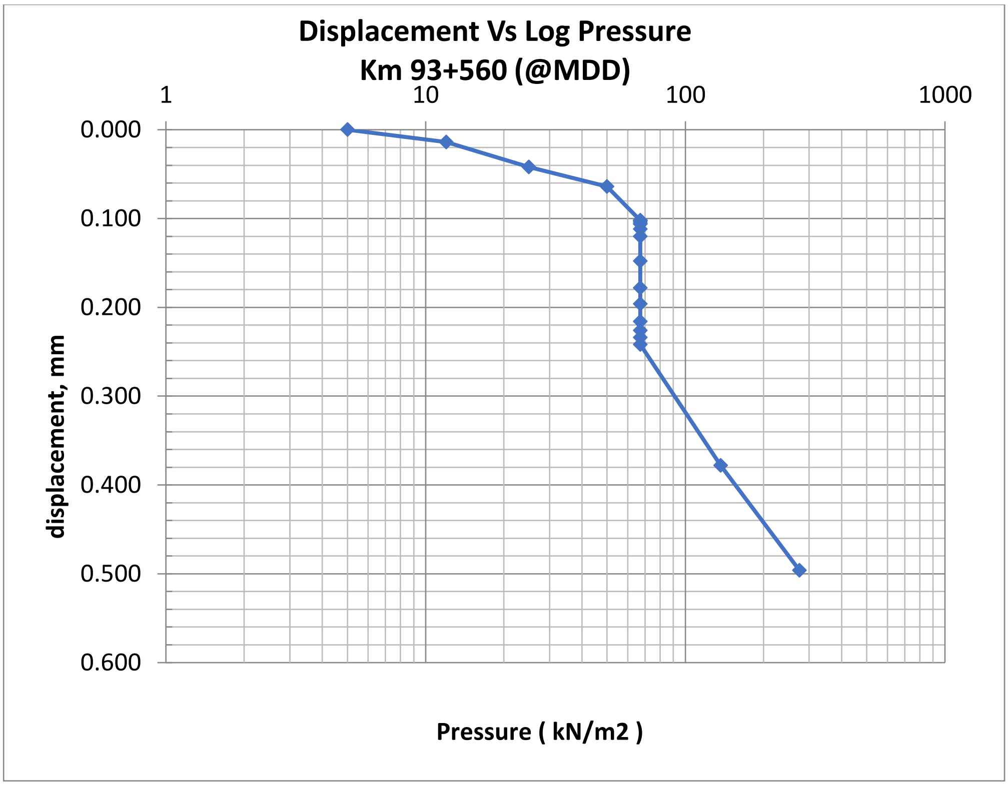

Figure 24: Collapse Potential for km 93+560 (@ MDD)

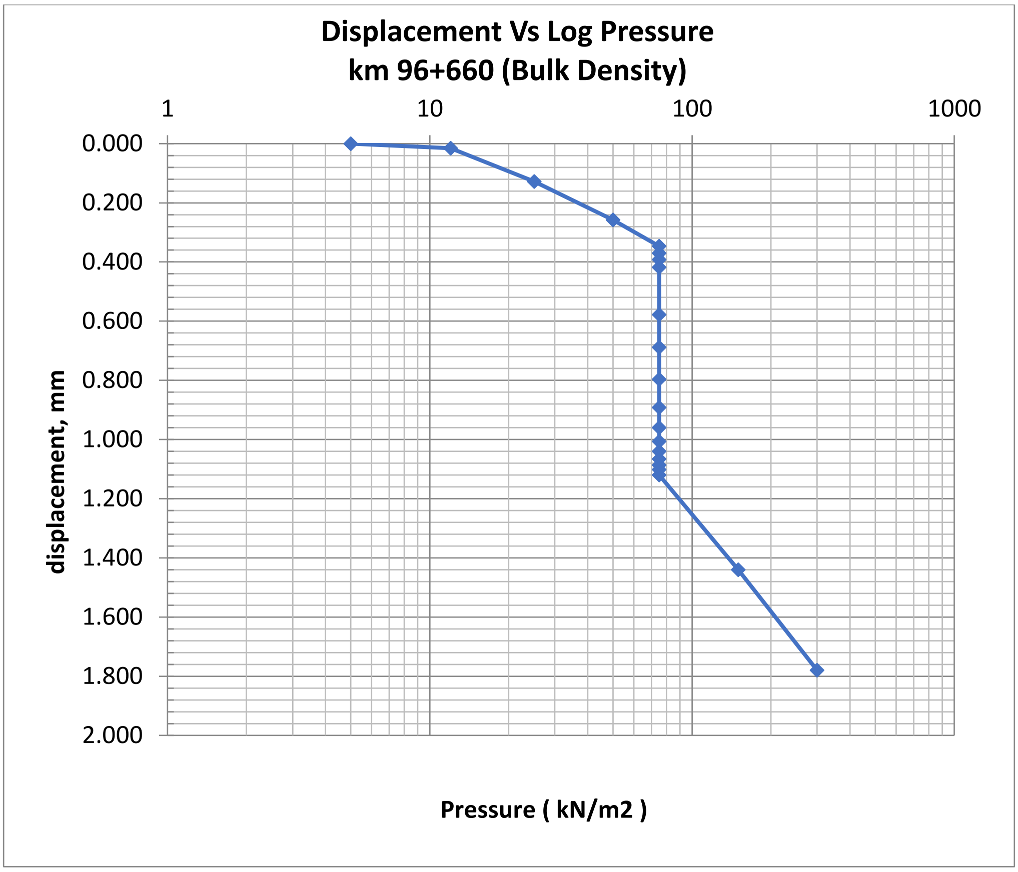

Figure 25: Collapse Potential for km 96+660 (@Bulk Density)

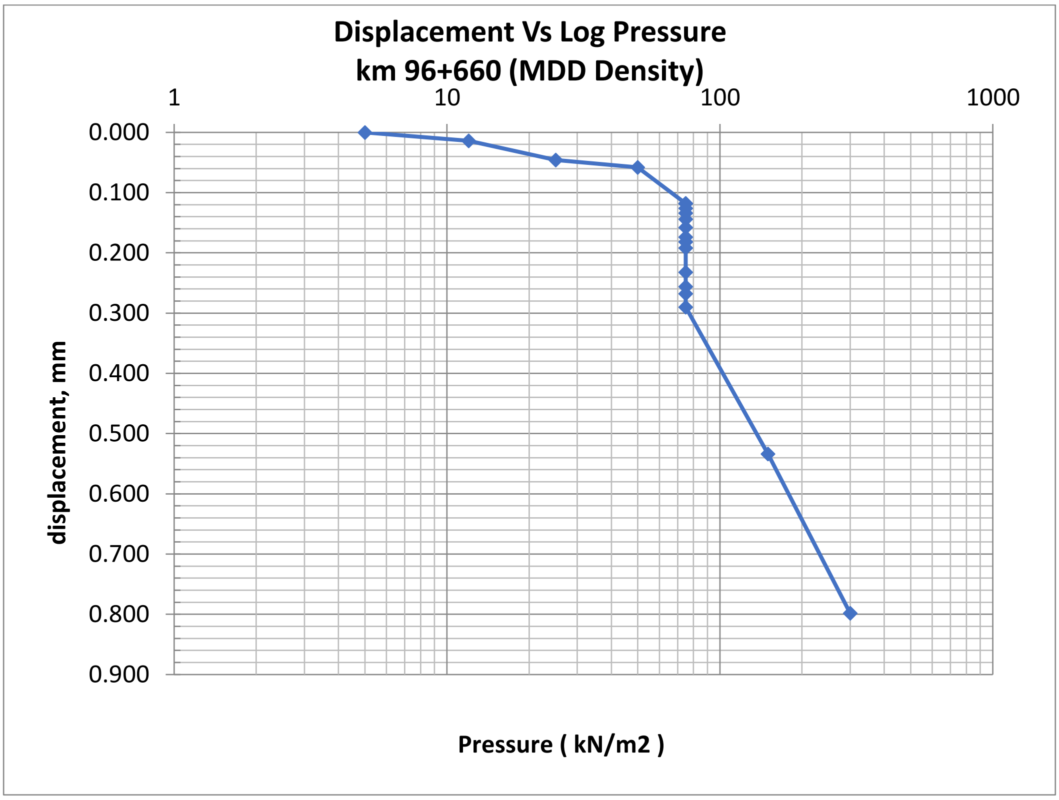

Figure 26: Collapse Potential for km 96+660 (@MDD)

Similar to the collapse indices, the obtained test results were checked against the collapse severity as specified by ASTM D5333. The collapse index ratings are tabulated in Table 12.

Table 12: Collapse Potential Rating as per ASTM D5333

Station	Remolding State	Density (g/cc)	Collapse Potential Value (%)	Rating as per ASTM D5333
93+560	Bulk	1.24	3.14	Moderate
	MDD	1.38	0.70	Slight
96+660	Bulk	1.18	3.87	Moderate
	MDD	1.24	0.86	Slight

In the collapse potential tests, the collapse property of the specimen when compacted at their maximum dry densities have significantly improved. This will lead us to a conclusion that the nature of collapsible soils can be highly improved by compaction.

As compared to the collapse indices obtained from the specimen subjected to a stress level of 200 kPa, the collapse potential obtained at a lower stress level are lesser. This can be an indication that the stress level does affect the obtained collapse property and that a higher stress level means a higher collapse index/potential.

5.7 Suggested Remedial Measures

There are numerous possible alternatives of improving the properties of collapsible soils. They mainly focus on either avoiding collapse triggering mechanisms (like using geotextile as a drainage filter to prohibit water reaching to the collapsible soils), or inducing collapse prior to construction (like pre-wetting and surcharging) or heavy compaction or using grouting and blending techniques.

The selection of the treatment method mainly depends on the type of the structure. Alternative treatment options recommended for highway subgrade may not be suitable for treating collapsible ground below bridge or building foundations.

For example, solutions like grouting, or pre-wetting and surcharging are more relevant to foundations since foundations occupy a small amount of area. In addition, the solution of precluding the presence of water, like using geotextile membranes, is almost impractical, as discussed on ERA's 2013 Geotechnical Design Manual.

Furthermore, blending the collapsible soils with selected material may not be the best solution as the collapsible soils by their nature, except for their looseness, do not pose a serious problem and have a high strength; and hence, can be an excellent subgrade, if properly compacted. Therefore, alternative solutions involving compaction like excavation and re-compaction, high energy impact compaction and rapid impact compaction are recommended.

6. Conclusions and Recommendations

6.1. Conclusions

Based on the study and results of the investigation, the following conclusions are drawn:

- Field and laboratory tests on samples taken from the representative stations located at km 93+560 and km 96+660 have revealed that the soils are in low density at their in-situ state, mainly composed of sands and silts and have low plastic nature.
- The moisture-density relationship established from proctor compaction test has showed that the samples have maximum dry densities of 1.380 g/cc and 1.24 g/cc and optimum moisture contents of 24.4% and 26.2% for samples collected from test pits located at km 93+560 and km 96+660 respectively.
- One dimensional collapse potential tests done for the samples remolded at their bulk densities and subjected to stress level of 200 kPa as well as to in-situ stress levels anticipated at site have shown that the samples are moderately collapsible as per ASTM classification.
- One dimensional collapse potential tests done for the samples remolded at their maximum dry densities and subjected to stress level of 200 kPa as well as to in-situ stress levels anticipated at site has shown that the samples are slightly collapsible as per ASTM classification.
- One-dimensional test results obtained at the same density but at different stress levels revealed that the magnitude of the loading stress level had effect on the collapse potential of the samples. A higher stress level resulted a higher collapse index/potential.
- Observing the collapse potential test results for samples remolded at their bulk and maximum dry densities, it was learnt that the collapse nature of the samples can be highly enhanced by improving their densities.

6.2. Recommendations

From the study it is recommended that:-

- The present work has tried to study the collapsible soils found in the vicinity of Mojo-Hawassa Design and Build Highway Project, Lot-3; Zeway-Arsi Negele Section. In so doing, index and mechanical properties which are thought to be directly related with the collapse nature of the soils were studied. However, other engineering properties such as shear strength and CBR were not studied. Researches on collapsible soils that consider and study such properties have to be done.
- Intensive studies on the alternative treatment methods of collapsible soils based on the magnitude of the collapsibility, the effectiveness of the methods and the cost to be incurred have to be undertaken.

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Appendices

Appendix A: Field and Laboratory Test Results

Sand Calibration for Density Determination Using Sand Replacement Method

Calibrating Cylinder	
Height(cm)=	11.2
Diameter(cm)=	9.8
Volume(cm3)=	844.9
Weight (g)=	82

	Trial 1	Trial 2	Trial 3
W(sand+calibrating cylinder) (g)	1150	1156	1153
Avg. W (sand+calibrating cylinder) (g)		1153	
W(sand in the calibrating cylinder) (g)		1071	
Unit wight of sand=		1.268	

	Trial 1	Trial 2	Trial 3
W(density jar+sand, before pouring) (g)=	8760	8122	7482
W(density jar+sand, after pouring) (g)=	8123	7482	6842
W(sand in cone) (g)=	637	640	640
Avg. Weight of Sand in Cone (g)=		639	

In Situ Density Determination Using Sand Replacement Method (AASHTO T-191)		
Unit weight of sand, Ɣs=	1.268	
Sand in Cone=	639	
Density Determination		
Station	93+560	96+660
Mass of wet soil from hole, Mw (g)	2701	2477
Mass of sand+jar before pouring, M1	8928	9630
Mass of sand+jar after pouring, M2	5508	6326
Mass of sand in cone, M3 (g)	639	639
Mass of sand in hole, M4=M1-M2-M3	2781	2665
Bulk density, Ɣwet=Mw/M4*Ɣs (g/cc)	1.24	1.18
Moisture Determination		
Mass of wet soil+Container (g), A	238.71	274.85
Mass of dry soil+Container (g), B	210.17	234.12
Mass of Container (g), C	44.85	39.38
Mass of water (g), D=A-B	28.54	40.73
Mass of dry soil (g), E=B-C	165.32	194.74
Moisture Content, ω(%), F=D/E*100	17.27	20.92
Dry Density, Ɣdry=Ɣwet*100/(100+ω)	1.06	0.98

Determination of Specific Gravity (ASTM D854-00)		
	Station	93+560
Trial	1	2
Pycnometer Bottle Number	N	V
Wp=Mass of empty, clean pycnometer (g)	40.15	39.8
Wps=Mass of empty pycnometer + dry soil (g)	50.15	49.8
Wb=Mass of pycnometer + dry soil + water (g)	92.95	93.52
Wa=Mass of Pycnometer + water (g)	86.94	87.51
Specific Gravity (Gs)=(Wps-Wp)/(Wps-Wp+Wa-Wb)	2.51	2.51
Average Specific Gravity	2.51	
	Station	96+660
Trial	1	2
Pycnometer Bottle Number	S	I
Wp=Mass of empty, clean pycnometer (g)	40.78	41.97
Wps=Mass of empty pycnometer + dry soil (g)	50.78	51.97
Wb=Mass of pycnometer + dry soil + water (g)	93.81	92.84
Wa=Mass of Pycnometer + water (g)	87.71	86.77
Specific Gravity (Gs)=(Wps-Wp)/(Wps-Wp+Wa-Wb)	2.57	2.55
Average Specific Gravity	2.56	

Particle Size Analysis of Soils (ASTM D422)

Weight of sample before washing (g)=	235
Weight of sample after washing (g) =	89.3

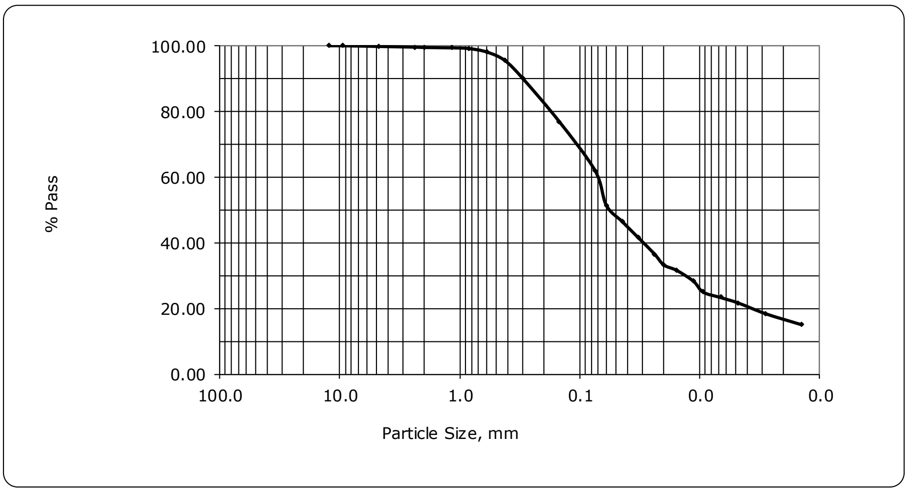
Sieve No.	Sieve Openeing (mm)	Cummulative Weight Retained (g)	Cummulative % Retained	% Pass
3''	75.0	0.0	0.0	100.0
2''	50.0	0.0	0.0	100.0
1.5"	37.5	0.0	0.0	100.0
1"	25.0	0.0	0.0	100.0
3/4''	19.0	0.0	0.0	100.0
1/2"	12.5	0.0	0.0	100.0
3.8"	9.5	0.0	0.0	100.0
No 4	4.75	0.5	0.2	99.8
No 8	2.36	1.0	0.4	99.6
No 10	2	1.3	0.5	99.5
No 16	1.18	1.5	0.6	99.4
No 20	0.85	2.0	0.9	99.1
No 30	0.6	4.5	1.9	98.1
No 40	0.425	10.5	4.5	95.5
No 50	0.3	23.5	10.0	90.0
No 100	0.15	54.0	23.0	77.0
No 200	0.075	89.3	38.0	62.0
pan	-------	145.7	--------	-------

Dry Wt. of Sample =	100
%pass No. 10=	99.5
W =	100.53

Sp. Gravity soil (G)=	2.51
Sp. Gravity liquid (G1)=	1

Elapsed Time T(min)	Actual Hy.reading	Composite Correction	(R) Hydro. Reading with com.	Temperature (^{0}C)	Effective depth Of Hyd. L (cm) table(7-4)	K	Diameter of soil particle D(mm)	Soil in suspension P(%of soil finer)
0.5	1.0270	0.0041	1.0311	20	9.2	0.01408	0.060	51.42
1	1.0240	0.0041	1.0281	20	10.0	0.01408	0.045	46.46
2	1.0210	0.0041	1.0251	20	10.7	0.01408	0.033	41.50
4	1.0180	0.0041	1.0221	20	11.5	0.01408	0.024	36.54
6	1.0160	0.0041	1.0201	20	12.1	0.01408	0.020	33.23
10	1.0150	0.0041	1.0191	20	12.3	0.01408	0.016	31.58
20	1.0130	0.0041	1.0171	20	12.9	0.01408	0.011	28.27
30	1.0110	0.0041	1.0151	20	13.4	0.01408	0.009	24.97
60	1.0100	0.0041	1.0141	20	13.7	0.01408	0.007	23.31
120	1.0090	0.0041	1.0131	20	13.9	0.01408	0.005	21.66
360	1.0070	0.0041	1.0111	20	14.4	0.01408	0.003	18.35
1440	1.0050	0.0041	1.0091	20	14.7	0.01408	0.001	15.05

ASTM SIEVE (MM)	% Pass
12.5	100.00
9.5	100.00
4.75	99.79
2.36	99.57
2.00	99.47
1.18	99.36
0.85	99.15
0.6	98.09
0.425	95.53
0.300	90.00
0.150	77.02
0.075	62.00
0.060	51.42
0.045	46.46
0.033	41.50
0.024	36.54
0.020	33.23
0.016	31.58
0.011	28.27
0.009	24.97
0.007	23.31
0.005	21.66
0.003	18.35
0.001	15.05



Soil Classification

AASHTO	A4 (6)
USCS	ML

	No.	Station	Depth Sampled(M)	Gravel % (>2.0mm)	Sand % (2.0mm-0.06mm)	Silt% (0.06mm-0.02mm)	Clay %(<0.002)	Remark
MIT Textural Soil Classification	1	93+560	1m	0.53	48.0	34.72	16.7	Silty Sand Soil

Particle Size Analysis of Soils (ASTM D422)

Weight of sample before washing (g)=	243
Weight of sample after washing (g) =	86.1

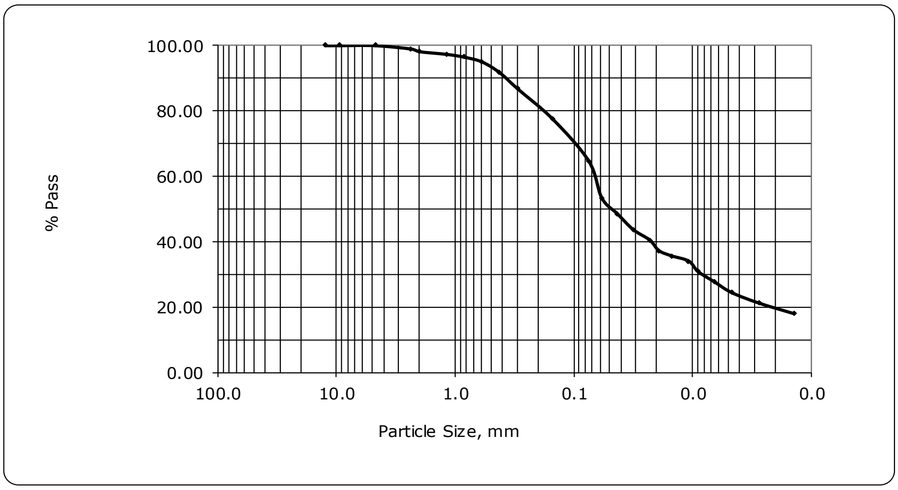
Sieve No.	Sieve Openeing (mm)	Cummulative Weight Retained (g)	Cummulative % Retained	% Pass
3''	75.0	0.0	0.0	100.0
2''	50.0	0.0	0.0	100.0
1.5"	37.5	0.0	0.0	100.0
1"	25.0	0.0	0.0	100.0
3/4''	19.0	0.0	0.0	100.0
1/2"	12.5	0.0	0.0	100.0
3.8"	9.5	0.0	0.0	100.0
No 4	4.75	0.0	0.0	100.0
No 8	2.36	2.5	1.0	99.0
No 10	2	4.5	1.9	98.1
No 16	1.18	6.5	2.7	97.3
No 20	0.85	8.5	3.5	96.5
No 30	0.6	12.0	4.9	95.1
No 40	0.425	20.0	8.2	91.8
No 50	0.3	32.0	13.2	86.8
No 100	0.15	55.0	22.6	77.4
No 200	0.075	86.1	35.4	64.6
pan	-------	156.9	--------	-------

Dry Wt. of Sample =	100
%pass No. 10=	98.1
W =	101.89

Sp. Gravity soil (G)=	2.56
Sp. Gravity liquid (G1)=	1

Elapsed Time T(min)	Actual Hy.reading	Composite Correction	(R) Hydro. Reading with com.	Temperature (^{0}C)	Effective depth Of Hyd. L (cm) table(7-4)	K	Diameter of soil particle D(mm)	Soil in suspension P(%of soil finer)
0.5	1.0290	0.0041	1.0331	20	8.6	0.01408	0.058	53.31
1	1.0260	0.0041	1.0301	20	9.4	0.01408	0.043	48.48
2	1.0230	0.0041	1.0271	20	10.2	0.01408	0.032	43.65
4	1.0210	0.0041	1.0251	20	10.7	0.01408	0.023	40.43
6	1.0190	0.0041	1.0231	20	11.3	0.01408	0.019	37.21
10	1.0180	0.0041	1.0221	20	11.5	0.01408	0.015	35.60
20	1.0170	0.0041	1.0211	20	11.8	0.01408	0.011	33.98
30	1.0150	0.0041	1.0191	20	12.3	0.01408	0.009	30.76
60	1.0130	0.0041	1.0171	20	12.9	0.01408	0.007	27.54
120	1.0110	0.0041	1.0151	20	13.4	0.01408	0.005	24.32
360	1.0090	0.0041	1.0131	20	13.9	0.01408	0.003	21.10
1440	1.0070	0.0041	1.0111	20	14.4	0.01408	0.001	17.88

ASTM SIEVE (MM)	% Pass
12.5	100.00
9.5	100.00
4.75	100.00
2.36	98.97
2.00	98.15
1.18	97.33
0.85	96.50
0.6	95.06
0.425	91.77
0.300	86.83
0.150	77.37
0.075	64.57
0.058	53.31
0.043	48.48
0.032	43.65
0.023	40.43
0.019	37.21
0.015	35.60
0.011	33.98
0.009	30.76
0.007	27.54
0.005	24.32
0.003	21.10
0.001	17.88



Soil Classification

AASHTO	A4 (7)
USCS	ML

MIT Textural Soil Classification	No.	Station	Depth Sampled(M)	Gravel % (>2.0mm)	Sand % (2.0mm-0.06mm)	Silt% (0.06mm-0.02mm)	Clay %(<0.002)	Remark
	1	96+660	1m	1.85	44.8	33.82	19.5	Silty Sand Soil

Determination of Liquid Limit, Plastic Limit and Plasticity Index (AASHTO T89) Station 93+560								
Liquid Limit W_L								
Run Number	1	2	3					
Tare Number	A-3	A-5	A-7					
A. Weight of Wet Soil + Tare	28.28	27.52	30.04					
B. Weight of Dry Soil + Tare	24.62	23.92	25.76					
C. Weight of Water (A – B)	3.66	3.60	4.28					
D. Weight of Tare	12.11	12.03	11.97					
E. Weight of Dry Soil (B -D)	12.51	11.89	13.79					
Water Content % (C / E x 100)	29.3	30.3	31.0					
Number of Blows	33	25	17					
Liquid Limit %	*30.0*							

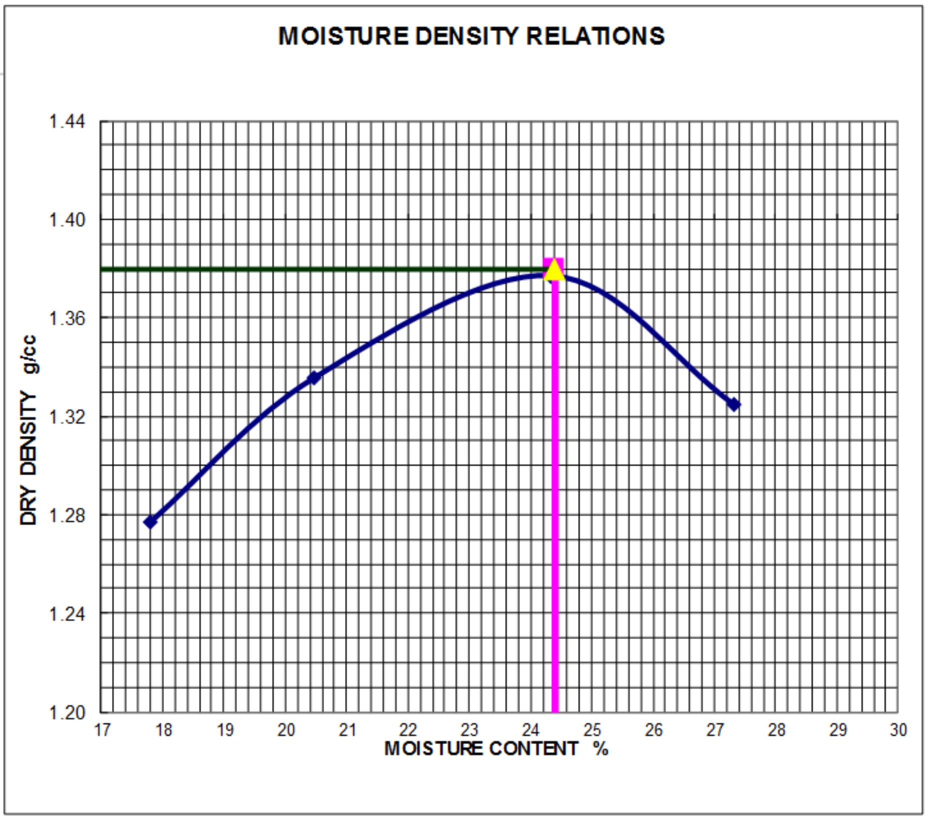
Plastic Limit W_P			
Run Number	1	2	
Tare Number	B-2	B3	
F. Weight of Wet soil + Tare(gm)	14.97	15.84	
G. Weight of Dry soil + Tare(gm)	14.33	15.20	
H. Weight of water (F – G)	0.64	0.64	
I. Weight of Tare	11.96	12.83	
J. Weight of Dry soil (G – I)	2.37	2.37	
Water Content % (H / J x 100)	27.00	27.00	
Plastic Limit % (Average)	*27.0*		
Plasticity Index ($PI = W_L - W_P$) :	*3.0*		

Determination of Liquid Limit, Plastic Limit and Plasticity Index (AASHTO T89) Station 96+660								
Liquid Limit W_L								
Run Number	1	2	3					
Tare Number	A-4	A-9	A-1					
A. Weight of Wet Soil + Tare	27.00	26.58	30.98					
B. Weight of Dry Soil + Tare	23.79	23.39	26.70					
C. Weight of Water (A – B)	3.21	3.19	4.28					
D. Weight of Tare	12.10	12.06	11.90					
E. Weight of Dry Soil (B -D)	11.69	11.33	14.80					
Water Content % (C / E x 100)	27.5	28.2	28.9					
Number of Blows	31	25	17					
Liquid Limit %	***28.0***							

Plastic Limit W_P			
Run Number	1	2	
Tare Number	B-5	B-8	
F. Weight of Wet soil + Tare(gm)	15.79	15.83	
G. Weight of Dry soil + Tare(gm)	15.00	15.21	
H. Weight of water (F – G)	0.79	0.62	
I. Weight of Tare	11.95	12.84	
J. Weight of Dry soil (G – I)	3.05	2.37	
Water Content % (H / J x 100)	25.90	26.16	
Plastic Limit % (Average)	***26.0***		
Plasticity Index (PI = W_L - W_P) :	***2.0***		

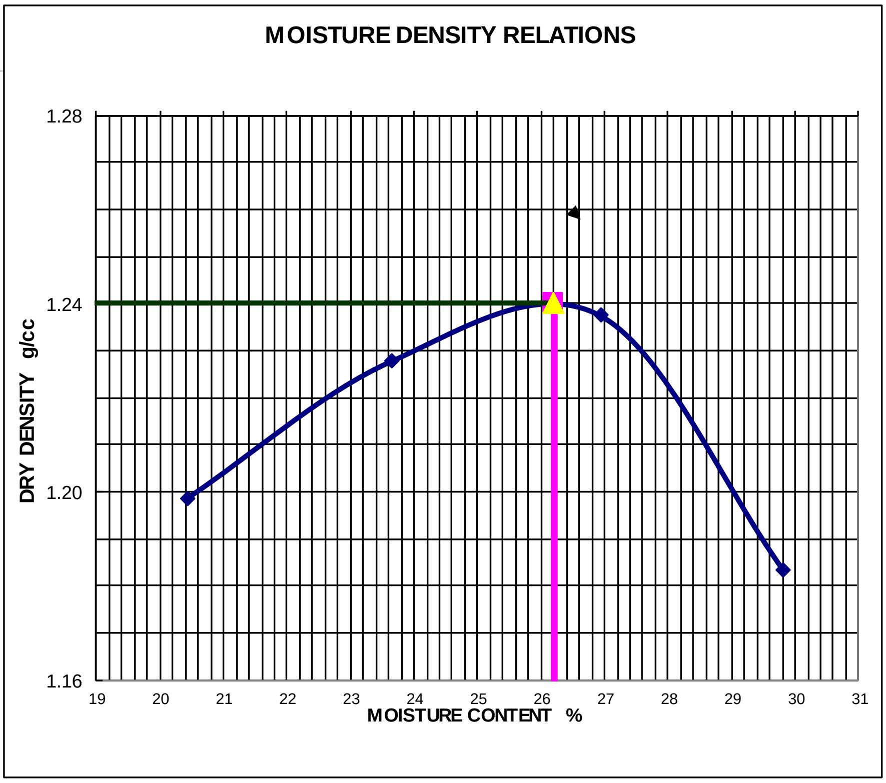
Moisture Density Relationship of Soil (AASHTO T-180)-Station Km 93+560							
DENSITY	Trial Number		1	2	3	4	
	Weight of soil + Mold	g	8544	8766	8984	8930	
	Weight of mold	g	5348	5348	5348	5348	
	Weight of soil	g	3196	3418	3636	3582	
	Volume of mold	cc	2124	2124	2124	2124	
	Wet density of soil	g/cc	1.505	1.609	1.712	1.686	
MOISTURE	Container number		A11	A8	B1	A6	
	Wet soil + container	g	262.2	310.3	348.8	375.9	
	Dry soil + Container	g	230.7	266.1	290.4	306.0	
	Weight of water	g	31.5	44.2	58.4	69.9	
	Weight of container	g	53.8	50.3	50.4	50.0	
	Weight of dry soil	g	176.9	215.8	240.0	256.0	
	Moisture content	%	17.8	20.5	24.3	27.3	
Dry density of soil		**g/cc**	1.277	1.336	1.377	1.325	

MDD (g/cc) :	1.380
OMC (%) :	24.4



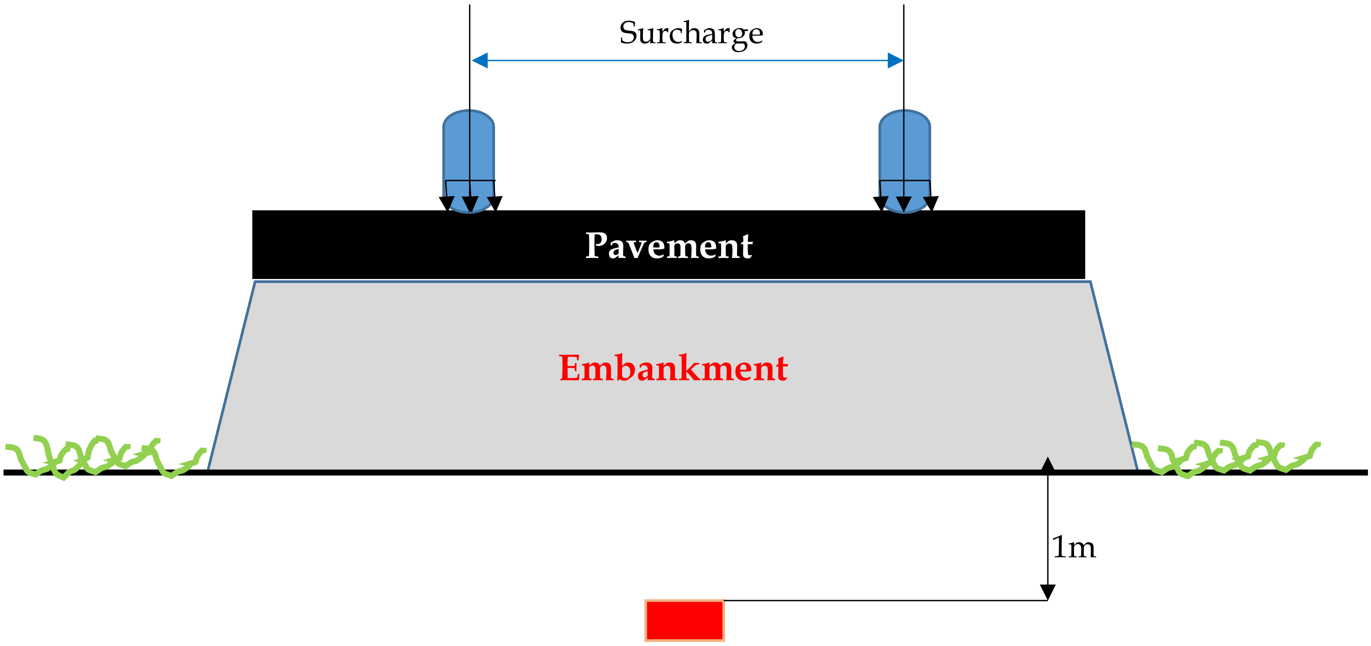
Moisture Density Relationship of Soil (AASHTO T-180)-Station Km 96+660							
DENSITY	Trial Number		1	2	3	4	
	Weight of soil + Mold	g	8414	8572	8684	8611	
	Weight of mold	g	5348	5348	5348	5348	
	Weight of soil	g	3066	3224	3336	3263	
	Volume of mold	cc	2124	2124	2124	2124	
	Wet density of soil	g/cc	1.444	1.518	1.571	1.536	
MOISTURE	Container number		A11	A8	B1	A6	
	Wet soil + container	g	341.3	355.2	215.3	298.7	
	Dry soil + Container	g	291.9	297.3	180.8	242.4	
	Weight of water	g	49.4	57.9	34.5	56.3	
	Weight of container	g	50.1	52.4	52.7	53.6	
	Weight of dry soil	g	241.8	244.9	128.1	188.8	
	Moisture content	%	20.4	23.6	26.9	29.8	
Dry density of soil		**g/cc**	1.199	1.228	1.237	1.183	

MDD (g/cc) :	1.240
OMC (%) :	26.2



Calculation of Stress Level used in Collapse Potential Test

Station:- 93+560



Stress from Surcharge = σ_{sc} = 10 KN/m²

Unit weight of the pavement = Υ_p= 22.5 KN/m³

Height of Pavement = H_p= 0.63m

Unit weight of the Embankment = Υ_E= 17 KN/m³

Height of Embankment = H_E = 2.408m

Density of the Subgrade = d_s= 1.2315 g/cm³=0.126 KN/m³

Depth of Sampling = D_s = 1m below subgrade level

Calculation:-

Stress from Pavement = σ_p= H_p* Υ_p= 0.63m*22.5 KN/m³= 14.175 kPa

Stress from Embankment = σ_E= H_E* Υ_E= 2.408m*17 KN/m³= 40.936 kPa

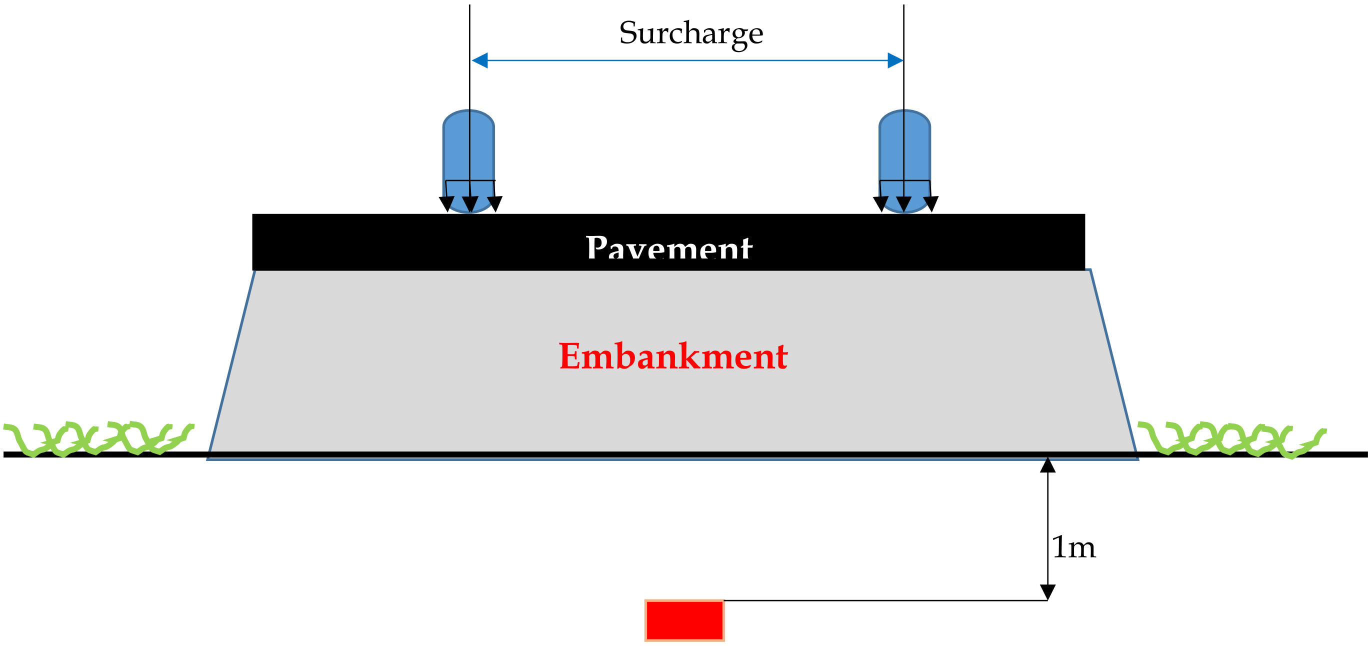
Stress of Subgrade up to sampling depth = $\sigma_{S.G.}$= D_s* d_s= 1m*0.126 KN/m³= 0.126 kPa

Total stress at the sampling depth = σ_{sc}+ σ_p + σ_E + $\sigma_{S.G}$ = (10+14.175+40.936+0.126) kPa=

Total stress at the sampling depth = 65.237 kPa

Calculation of Stress Level used in Collapse Potential Test

Station:- 96+660



Stress from Surcharge = σ_{sc} = 10 KN/m^2

Unit weight of the pavement = Υ_p= 22.5 KN/m^3

Height of Pavement = H_p= 0.63m

Unit weight of the Embankment = Υ_E= 17 KN/m^3

Height of Embankment = H_E = 2.678m

Density of the Subgrade = d_s= 1.3114 g/cm^3=0.134 KN/m^3

Depth of Sampling = D_s = 1m below subgrade level

Calculation:-

Stress from Pavement = σ_p= H_p* Υ_p= 0.63m*22.5 KN/m^3= 14.175 kPa

Stress from Embankment = σ_E= H_E* Υ_E= 2.678m*17 KN/m^3= 45.526 kPa

Stress of Subgrade up to sampling depth = $\sigma_{S.G.}$= D_s* d_s= 1m*0.134 KN/m^3= 0.134 kPa

Total stress at the sampling depth = σ_{sc}+ σ_p + σ_E + $\sigma_{S.G}$ = (10+14.175+45.526+0.134) kPa

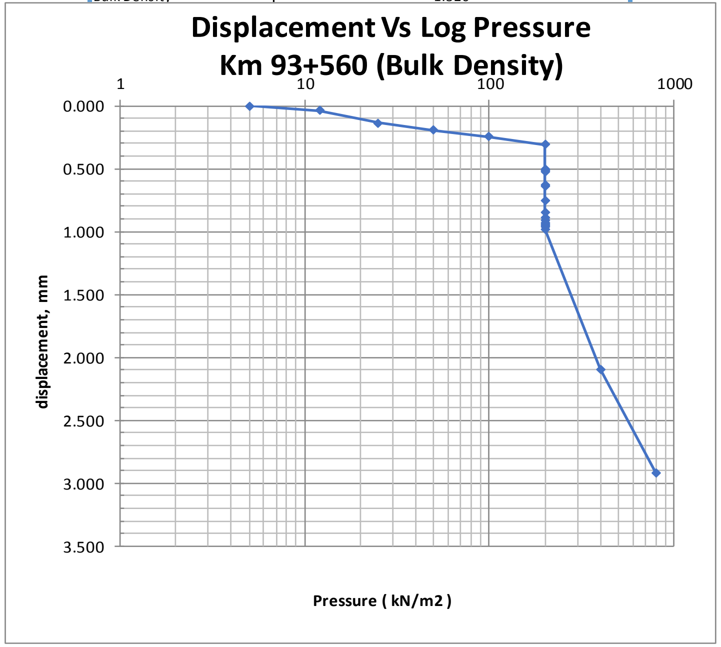
Total stress at the sampling depth = 69.835 kPa

ASTM D 5333 Measurement of Collapse Potential of Soil

Station: 93+560 (Bulk)

Specific Gravity	2.51	Overall settlement	0.980	Diameter of sample	50
Vs=DD/Gs*Vt(cm3)	17.21	Volume Change	1.92	Thickness of sample	20
Vvi=Vt-Vs	22.06	Final volume	37.35	Area of sample	1963
intial Void ratio e0=Vv/Vs	1.2818	Final bulk density	1.532	Volume of Sample	39.27
intial So= Vw/Vvi	39.163	Final Dry Density	1.12		
Volume change factor=F	0.1141	Final void ratio	1.17		
		Vvf=Vtf-Vs	20.14		
		Final saturation Sf, %	76.58		

Moisture Determination			
Before Test		After Test	
Wt.of can +wet sample(gm)	40.81	Wt.of ring +wet sample(gm)	126.5
Wt.of can +Dry sample(gm)	37.64	Wt.of ring +Dry sample(gm)	111.08
Wt.can(gm)	16.58	Wt.ring(gm)	69.27
Wt.of dry sample	21.06	Wt.of dry sample	41.81
Wt.of wet sample	24.23	Wt.of wet sample	57.23
Wt.of water(gm)	3.17	Wt.of water(gm)	15.42
Moisture Content(%)	15.05	Moisture Content(%)	36.88
Bulk Density	1.320		



Elapsed Time h	Time, Min	Applied Stress, Kpa	Dial Reading, mm	Cumulative Compression, mm	Void Ratio Δe=F*ΔH	e1=eo-Δe
	0	5	0.0	0.000	0.000	1.282
	5	12	18.0	0.036	0.004	1.278
1		25	69.0	0.138	0.016	1.266
2		50	97.0	0.194	0.022	1.260
3		100	123.0	0.246	0.028	1.254
4		200	155.0	0.310	0.035	1.246
	0.1	200	250.0	0.500	0.057	1.225
	0.25	200	257.0	0.514	0.059	1.223
	0.5	200	260.0	0.520	0.059	1.222
	1	200	312.0	0.624	0.071	1.211
	2	200	321.0	0.642	0.073	1.209
	4	200	376.0	0.752	0.086	1.196
	8	200	424.0	0.848	0.097	1.185
	15	200	442.0	0.884	0.101	1.181
	30	200	453.0	0.906	0.103	1.178
1		200	463.0	0.926	0.106	1.176
2		200	473.0	0.946	0.108	1.174
4		200	476.0	0.952	0.109	1.173
8		200	480.0	0.960	0.110	1.172
24		200	490.0	0.980	0.112	1.170
32		400	1046.0	2.092	0.239	1.043
38		800	1457.0	2.914	0.332	0.949

Collapse Index, Ie Computation	
d_o, *dial reading at seating stress, mm*	0.0
h_o, initial specimen height, mm	20
d_f, dial reading at 200KPa after wetting ,mm	0.980
d_i, dial reading at 200KPa before wetting, mm	0.310
collapse Index $I_e=((d_f - d_i)/h_o)*100$	3.35

ASTM D 5333 Measurement of Collapse Potential of Soil

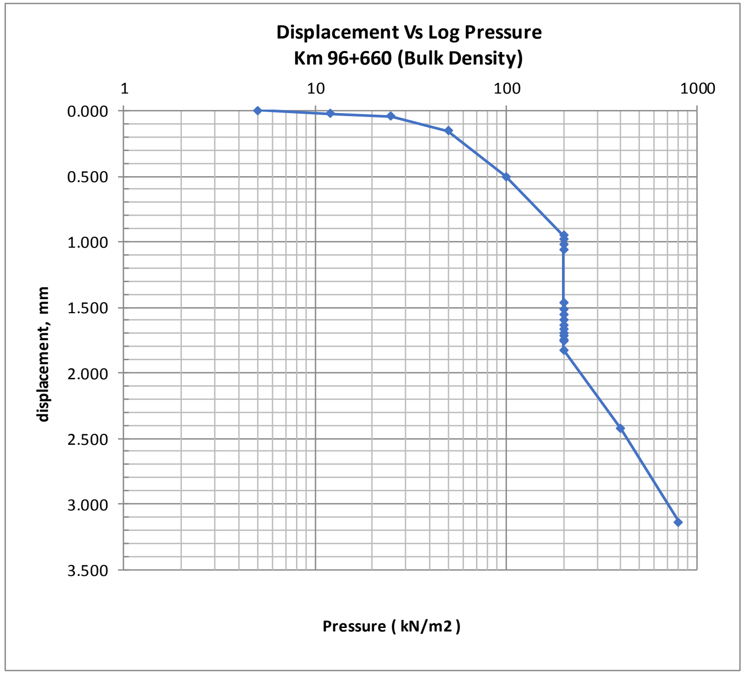
Station: 96+660 (Bulk)

Specific Gravity	2.56	Overall settlement	1.826	Diameter of sample	50
Vs=DD/Gs*Vt(cm3)	16.874	Volume Change	3.59	Thickness of sample	20
Vvi=Vt-Vs	22.40	Final volume	35.68	Area of sample	1963
intial Void ratio e0=Vv/Vs	1.3273	Final bulk density	1.558	Volume of Sample	39.27
intial So= Vw/Vvi	38.575	Final Dry Density	1.16		
Volume change factor=F	0.1164	Final void ratio	1.11		
		Vvf=Vtf-Vs	18.81		
		Final saturation Sf, %	75.54		

Moisture Determination			
Before Test		After Test	
Wt.of can +wet sample(gm)	36.57	Wt.of ring +wet sample(gm)	124.87
Wt.of can +Dry sample(gm)	33.18	Wt.of ring +Dry sample(gm)	110.66
Wt.can(gm)	15.40	Wt.ring(gm)	69.27
Wt.of dry sample	17.78	Wt.of dry sample	41.39
Wt.of wet sample	21.17	Wt.of wet sample	55.6
Wt.of water(gm)	3.39	Wt.of water(gm)	14.21
Moisture Content(%)	19.07	Moisture Content(%)	34.33
Bulk Density	1.320		
Dry density	1.100		

Elapsed Time h	Time, Min	Applied Stress, Kpa	Dial Reading, mm	Cumulative Compression, mm	Void Ratio Δe=F*ΔH	e1=eo-Δe
	0	5	0.0	0.000	0.000	1.327
	5	12	12.0	0.024	0.003	1.324
1		25	21.0	0.042	0.005	1.322
2		50	78.0	0.156	0.018	1.309
3		100	251.0	0.502	0.058	1.269
4		200	476.0	0.952	0.111	1.216
	0.1	200	489.0	0.978	0.114	1.213
	0.25	200	508.0	1.016	0.118	1.209
	0.5	200	529.0	1.058	0.123	1.204
	1	200	730.0	1.460	0.170	1.157
	2	200	755.0	1.510	0.176	1.152
	4	200	779.0	1.558	0.181	1.146
	8	200	799.0	1.598	0.186	1.141
	15	200	818.0	1.636	0.190	1.137
	30	200	832.0	1.664	0.194	1.134
1		200	848.0	1.696	0.197	1.130
2		200	857.0	1.714	0.199	1.128
4		200	873.0	1.746	0.203	1.124
8		200	880.0	1.760	0.205	1.122
24		200	913.0	1.826	0.212	1.115
32		400	1213.0	2.426	0.282	1.045
38		800	1569.0	3.138	0.365	0.962

Collapse Index, Ie Computation	
d_o, *dial reading at seating stress, mm*	0.0
h_o, initial specimen height, mm	20
d_f, dial reading at 200KPa after wetting ,mm	1.826
d_i, dial reading at 200KPa before wetting, mm	0.952
collapse Index $I_e=((d_f - d_i)/h_o)*100$	4.37



ASTM D 5333 Measurement of Collapse Potential of Soil

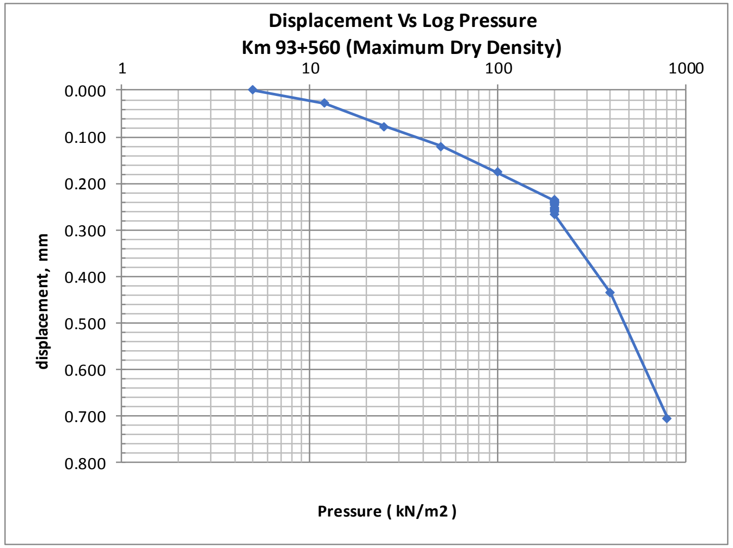
Station: 93+560 (MDD)

Specific Gravity	2.51	Overall settlement	0.266	Diameter of sample	50
Vs=DD/Gs*Vt(cm3)	17.21	Volume Change	0.52	Thickness of sample	20
Vvi=Vt-Vs	22.06	Final volume	38.75	Area of sample	1963
intial Void ratio e0=Vv/Vs	1.2818	Final bulk density	1.599	Volume of Sample	39.27
intial So= Vw/Vvi	39.163	Final Dry Density	1.21		
Volume change factor=F	0.1141	Final void ratio	1.25		
		Vvf=Vtf-Vs	21.54		
		Final saturation Sf, %	70.90		

Moisture Determination			
Before Test		After Test	
Wt.of can +wet sample(gm)	44.43	Wt.of ring +wet sample(gm)	131.24
Wt.of can +Dry sample(gm)	39.04	Wt.of ring +Dry sample(gm)	115.97
Wt.can(gm)	14.76	Wt.ring(gm)	69.27
Wt.of dry sample	24.28	Wt.of dry sample	46.7
Wt.of wet sample	29.67	Wt.of wet sample	61.97
Wt.of water(gm)	5.39	Wt.of water(gm)	15.27
Moisture Content(%)	22.20	Moisture Content(%)	32.70
Bulk Density	1.320		
Dry density	1.100		

Elapsed Time h	Time, Min	Applied Stress, Kpa	Dial Reading, mm	Cumulative Compression, mm	Void Ratio Δe=F*ΔH	e1=eo-Δe
	0	5	0.0	0.000	0.000	1.282
	5	12	14.0	0.028	0.003	1.279
1		25	39.0	0.078	0.009	1.273
2		50	60.0	0.120	0.014	1.268
3		100	88.0	0.176	0.020	1.262
4		200	118.0	0.236	0.027	1.255
	0.1	200	119.0	0.238	0.027	1.255
	0.25	200	119.0	0.238	0.027	1.255
	0.5	200	119.0	0.238	0.027	1.255
	1	200	120.0	0.240	0.027	1.254
	2	200	120.0	0.240	0.027	1.254
	4	200	120.0	0.240	0.027	1.254
	8	200	121.0	0.242	0.028	1.254
	15	200	122.0	0.244	0.028	1.254
	30	200	123.0	0.246	0.028	1.254
1		200	126.0	0.252	0.029	1.253
2		200	128.0	0.256	0.029	1.253
4		200	129.0	0.258	0.029	1.252
8		200	130.0	0.260	0.030	1.252
24		200	133.0	0.266	0.030	1.251
32		400	218.0	0.436	0.050	1.232
38		800	352.0	0.704	0.080	1.201

Collapse Index, Ie Computation	
d_o, *dial reading at seating stress, mm*	0.0
h_o, initial specimen height, mm	20
d_f, dial reading at 200KPa after wetting ,mm	0.266
d_i, dial reading at 200KPa before wetting, mm	0.236
collapse Index $I_e=((d_f - d_i)/h_o)*100$	0.15



ASTM D 5333 Measurement of Collapse Potential of Soil

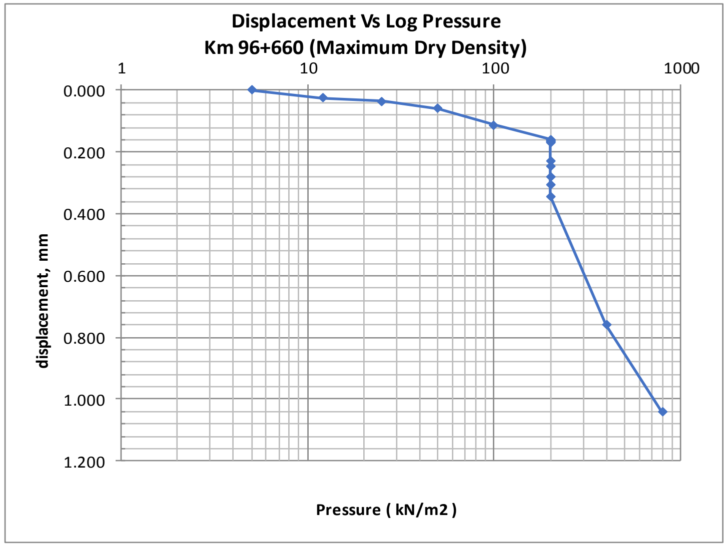
Station: 96+660 (MDD)

Specific Gravity	2.56	Overall settlement	0.342	Diameter of sample	50
Vs=DD/Gs*Vt(cm3)	15.033	Volume Change	0.67	Thickness of sample	20
Vvi=Vt-Vs	24.24	Final volume	38.60	Area of sample	1963
intial Void ratio e0=Vv/Vs	1.6122	Final bulk density	1.613	Volume of Sample	39.27
intial So= Vw/Vvi	32.405	Final Dry Density	1.21		
Volume change factor=F	0.1306	Final void ratio	1.57		
		Vvf=Vtf-Vs	23.57		
		Final saturation Sf, %	65.31		

Moisture Determination			
Before Test		After Test	
Wt.of can +wet sample(gm)	53.46	Wt.of ring +wet sample(gm)	131.52
Wt.of can +Dry sample(gm)	45.04	Wt.of ring +Dry sample(gm)	116.13
Wt.can(gm)	15.17	Wt.ring(gm)	69.27
Wt.of dry sample	29.87	Wt.of dry sample	46.86
Wt.of wet sample	38.29	Wt.of wet sample	62.25
Wt.of water(gm)	8.42	Wt.of water(gm)	15.39
Moisture Content(%)	28	Moisture Content(%)	32.84
Bulk Density	1.180		
Dry density	0.980		

Elapsed Time h	Time, Min	Applied Stress, Kpa	Dial Reading, mm	Cumulative Compression, mm	Void Ratio Δe=F*ΔH	e1=eo-Δe
	0	5	0.0	0.000	0.000	1.612
	5	12	12.0	0.024	0.003	1.609
1		25	18.0	0.036	0.005	1.608
2		50	29.0	0.058	0.008	1.605
3		100	56.0	0.112	0.015	1.598
4		200	80.0	0.160	0.021	1.591
	0.1	200	81.0	0.162	0.021	1.591
	0.25	200	81.0	0.162	0.021	1.591
	0.5	200	82.0	0.164	0.021	1.591
	1	200	82.0	0.164	0.021	1.591
	2	200	82.0	0.164	0.021	1.591
	4	200	82.5	0.165	0.022	1.591
	8	200	83.0	0.166	0.022	1.591
	15	200	84.0	0.168	0.022	1.590
	30	200	84.0	0.168	0.022	1.590
1		200	114.0	0.228	0.030	1.582
2		200	122.0	0.244	0.032	1.580
4		200	139.0	0.278	0.036	1.576
8		200	153.0	0.306	0.040	1.572
24		200	171.0	0.342	0.045	1.568
32		400	380.0	0.760	0.099	1.513
38		800	521.0	1.042	0.136	1.476

Collapse Index, Ie Computation	
d_o, dial reading at seating stress, mm	0.0
h_o, initial specimen height, mm	20
d_f, dial reading at 200KPa after wetting ,mm	0.342
d_i, dial reading at 200KPa before wetting, mm	0.160
collapse Index $I_e=((d_f - d_i)/h_o)*100$	0.91



ASTM D 5333 Measurement of Collapse Potential of Soil

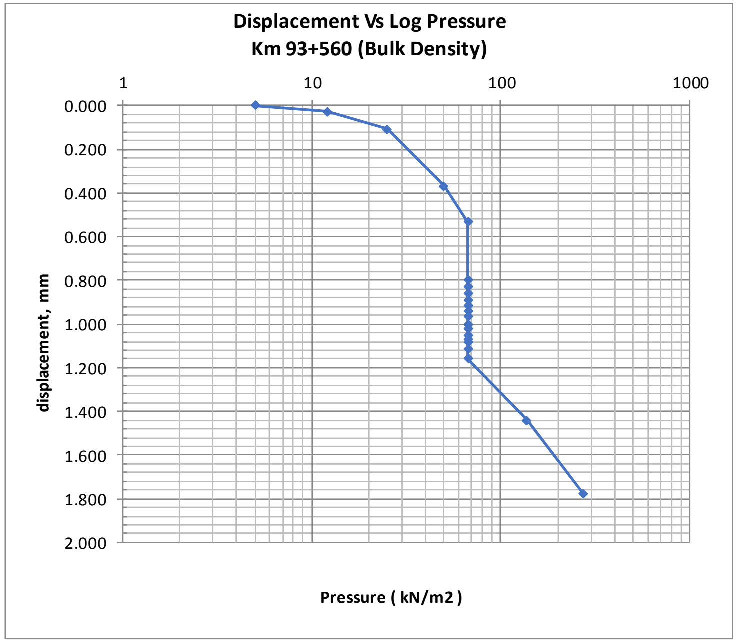
Station: 93+560 (Bulk)

Specific Gravity	2.51	Overall settlement	1.160	Diameter of sample	50
Vs=DD/Gs*Vt(cm3)	16.584	Volume Change	2.28	Thickness of sample	20
Vvi=Vt-Vs	22.69	Final volume	36.99	Area of sample	1963
intial Void ratio e0=Vv/Vs	1.3679	Final bulk density	1.589	Volume of Sample	39.27
intial So= Vw/Vvi	31.159	Final Dry Density	1.11		
Volume change factor=F	0.1184	Final void ratio	1.23		
		Vvf=Vtf-Vs	20.41		
		Final saturation Sf, %	87.17		

Moisture Determination			
Before Test		After Test	
Wt.of can +wet sample(gm)	29.29	Wt.of ring +wet sample(gm)	128.06
Wt.of can +Dry sample(gm)	27.57	Wt.of ring +Dry sample(gm)	110.27
Wt.can(gm)	15.84	Wt.ring(gm)	69.27
Wt.of dry sample	11.73	Wt.of dry sample	41
Wt.of wet sample	13.45	Wt.of wet sample	58.79
Wt.of water(gm)	1.72	Wt.of water(gm)	17.79
Moisture Content(%)	14.66	Moisture Content(%)	43.39
Bulk Density	1.240		
Dry density	1.060		

Elapsed Time h	Time, Min	Applied Stress, Kpa	Dial Reading, mm	Cumulative Compression, mm	Void Ratio Δe=F*ΔH	e1=eo-Δe
	0	5	0.0	0.000	0.000	1.368
	5	12	13.0	0.026	0.003	1.365
1		25	54.0	0.108	0.013	1.355
2		50	184.0	0.368	0.044	1.324
3		67	266.0	0.532	0.063	1.305
	0.1	67	398.0	0.796	0.094	1.274
	0.25	67	414.0	0.828	0.098	1.270
	0.5	67	430.0	0.860	0.102	1.266
	1	67	444.0	0.888	0.105	1.263
	2	67	458.0	0.916	0.108	1.259
	4	67	470.0	0.940	0.111	1.257
	8	67	482.0	0.964	0.114	1.254
	15	67	500.0	1.000	0.118	1.250
	30	67	510.0	1.020	0.121	1.247
1		67	525.0	1.050	0.124	1.244
2		67	535.0	1.070	0.127	1.241
4		67	542.0	1.084	0.128	1.240
8		67	558.0	1.116	0.132	1.236
24		67	580.0	1.160	0.137	1.231
32		137	720.0	1.440	0.170	1.197
38		275	890.0	1.780	0.211	1.157

Collapse Index, Ie Computation	
d_o, *dial reading at seating stress, mm*	0.0
h_o, initial specimen height, mm	20
d_f, dial reading at 200KPa after wetting ,mm	1.160
d_i, dial reading at 200KPa before wetting, mm	0.532
collapse Index $I_e=((d_f - d_i)/h_o)*100$	3.14



ASTM D 5333 Measurement of Collapse Potential of Soil

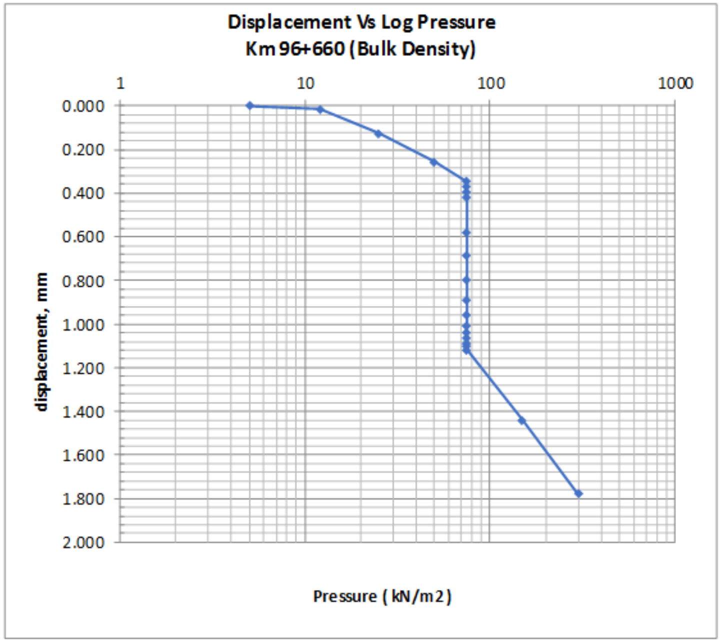
Station: 96+660 (Bulk)

Specific Gravity	2.51	Overall settlement	1.120	Diameter of sample	50
Vs=DD/Gs*Vt(cm3)	15.332	Volume Change	2.20	Thickness of sample	20
Vvi=Vt-Vs	23.94	Final volume	37.07	Area of sample	1963
intial Void ratio e0=Vv/Vs	1.5612	Final bulk density	1.571	Volume of Sample	39.27
intial So= Vw/Vvi	32.81	Final Dry Density	1.11		
Volume change factor=F	0.1281	Final void ratio	1.42		
		Vvf=Vtf-Vs	21.74		
		Final saturation Sf, %	78.20		

Moisture Determination			
Before Test		After Test	
Wt.of can +wet sample(gm)	43.28	Wt.of ring+wet sample(gm)	127.5
Wt.of can +Dry sample(gm)	38.62	Wt.of ring+Dry sample(gm)	110.5
Wt.can(gm)	16.61	Wt.ring(gm)	69.27
Wt.of dry sample	22.01	Wt.of dry sample	41.23
Wt.of wet sample	26.67	Wt.of wet sample	58.23
Wt.of water(gm)	4.66	Wt.of water(gm)	17
Moisture Content(%)	21.17	Moisture Content(%)	41.23
Bulk Density	1.180		
Dry density	0.980		

Elapsed Time h	Time, Min	Applied Stress, Kpa	Dial Reading, mm	Cumulative Compression, mm	Void Ratio Δe=F*ΔH	e1=eo-Δe
	0	5	0.0	0.000	0.000	1.561
	5	12	8.0	0.016	0.002	1.559
1		25	64.0	0.128	0.016	1.545
2		50	129.0	0.258	0.033	1.528
3		75	173.0	0.346	0.044	1.517
	0.1	75	185.0	0.370	0.047	1.514
	0.25	75	196.0	0.392	0.050	1.511
	0.5	75	209.0	0.418	0.054	1.508
	1	75	289.0	0.578	0.074	1.487
	2	75	344.0	0.688	0.088	1.473
	4	75	398.0	0.796	0.102	1.459
	8	75	446.0	0.892	0.114	1.447
	15	75	480.0	0.960	0.123	1.438
	30	75	503.0	1.006	0.129	1.432
1		75	520.0	1.040	0.133	1.428
2		75	533.0	1.066	0.137	1.425
4		75	543.0	1.086	0.139	1.422
8		75	551.0	1.102	0.141	1.420
24		75	560.0	1.120	0.143	1.418
32		150	720.0	1.440	0.184	1.377
38		300	890.0	1.780	0.228	1.333

Collapse Index, Ie Computation	
d_o, *dial reading at seating stress, mm*	0.0
h_o, initial specimen height, mm	20
d_f, dial reading at 200KPa after wetting, mm	1.120
d_i, dial reading at 200KPa before wetting, mm	0.346
collapse Index $I_e=((d_f - d_i)/h_o)*100$	3.87



ASTM D 5333 Measurement of Collapse Potential of Soil

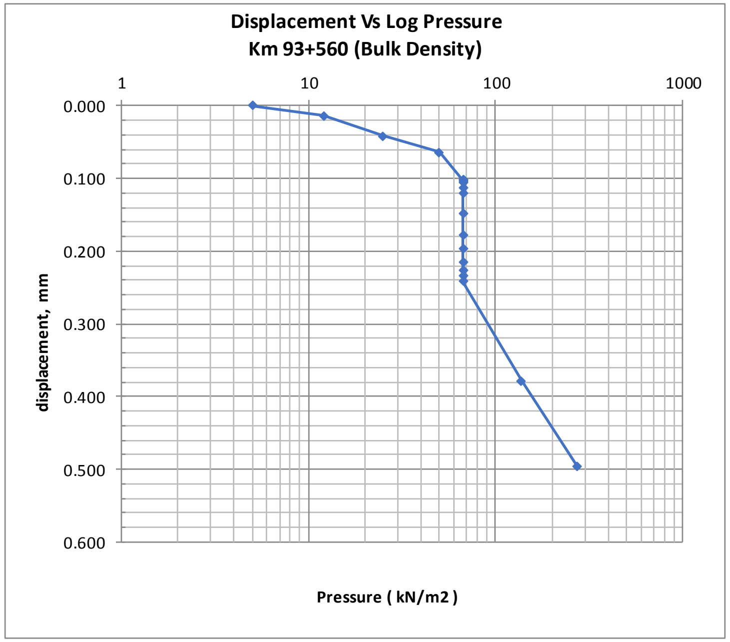
Station: 93+560 (MDD)

Specific Gravity	2.51	Overall settlement	0.242	Diameter of sample	50
Vs=DD/Gs*Vt(cm3)	16.584	Volume Change	0.48	Thickness of sample	20
Vvi=Vt-Vs	22.69	Final volume	38.79	Area of sample	1963
intial Void ratio e0=Vv/Vs	1.3679	Final bulk density	1.513	Volume of Sample	39.27
intial So= Vw/Vvi	31.159	Final Dry Density	1.11		
Volume change factor=F	0.1184	Final void ratio	1.34		
		Vvf=Vtf-Vs	22.21		
		Final saturation Sf, %	70.01		

Moisture Determination			
Before Test		After Test	
Wt.of can +wet sample(gm)	26.18	Wt.of ring +wet sample(gm)	127.95
Wt.of can +Dry sample(gm)	24.06	Wt.of ring +Dry sample(gm)	112.4
Wt.can(gm)	14.40	Wt.ring(gm)	69.27
Wt.of dry sample	9.66	Wt.of dry sample	43.13
Wt.of wet sample	11.78	Wt.of wet sample	58.68
Wt.of water(gm)	2.12	Wt.of water(gm)	15.55
Moisture Content(%)	21.95	Moisture Content(%)	36.05
Bulk Density	1.240		
Dry density	1.060		

Elapsed Time h	Time, Min	Applied Stress, Kpa	Dial Reading, mm	Cumulative Compression, mm	Void Ratio Δe=F*ΔH	e1=eo-Δe
	0	5	0.0	0.000	0.000	1.368
	5	12	7.0	0.014	0.002	1.366
1		25	21.0	0.042	0.005	1.363
2		50	32.0	0.064	0.008	1.360
3		67	51.0	0.102	0.012	1.356
	0.1	67	51.0	0.102	0.012	1.356
	0.25	67	51.0	0.102	0.012	1.356
	0.5	67	52.0	0.104	0.012	1.356
	1	67	53.0	0.106	0.013	1.355
	2	67	53.0	0.106	0.013	1.355
	4	67	56.0	0.112	0.013	1.355
	8	67	60.0	0.120	0.014	1.354
	15	67	74.0	0.148	0.018	1.350
	30	67	89.0	0.178	0.021	1.347
1		67	98.0	0.196	0.023	1.345
2		67	108.0	0.216	0.026	1.342
4		67	113.0	0.226	0.027	1.341
8		67	117.0	0.234	0.028	1.340
24		67	121.0	0.242	0.029	1.339
32		137	189.0	0.378	0.045	1.323
38		275	248.0	0.496	0.059	1.309

Collapse Index, Ie Computation	
d_o, *dial reading at seating stress, mm*	0.0
h_o, initial specimen height, mm	20
d_f, dial reading at 200KPa after wetting ,mm	0.242
d_i, dial reading at 200KPa before wetting, mm	0.102
collapse Index $I_e=((d_f - d_i)/h_o)*100$	0.7



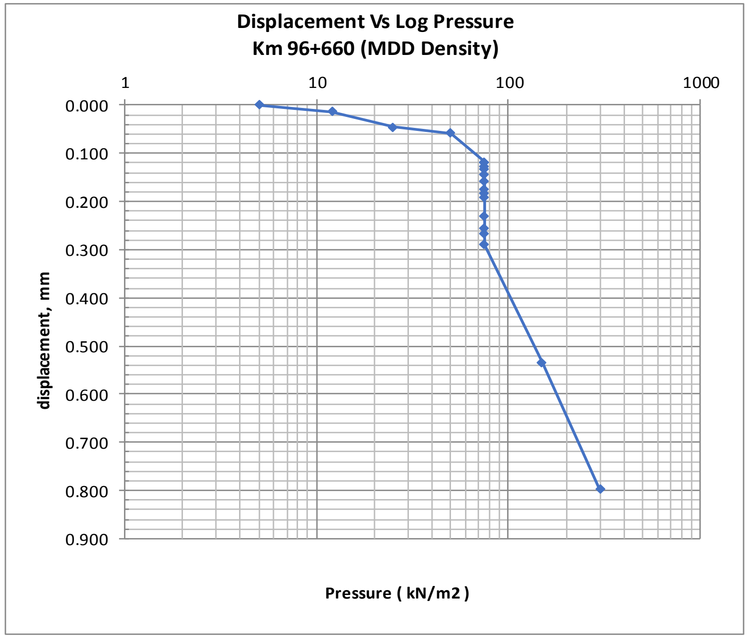
ASTM D 5333 Measurement of Collapse Potential of Soil

Station: 96+660 (MDD)

Specific Gravity	2.51	Overall settlement	0.290	Diameter of sample	50
Vs=DD/Gs*Vt(cm3)	15.332	Volume Change	0.57	Thickness of sample	20
Vvi=Vt-Vs	23.94	Final volume	38.70	Area of sample	1963
intial Void ratio e0=Vv/Vs	1.5612	Final bulk density	1.518	Volume of Sample	39.27
intial So= Vw/Vvi	32.81	Final Dry Density	1.11		
Volume change factor=F	0.1281	Final void ratio	1.52		
		Vvf=Vtf-Vs	23.37		
		Final saturation Sf, %	67.23		

Moisture Determination			
Before Test		After Test	
Wt.of can +wet sample(gm)	39.35	Wt.of ring +wet sample(gm)	128.01
Wt.of can +Dry sample(gm)	35.47	Wt.of ring +Dry sample(gm)	112.3
Wt.can(gm)	18.10	Wt.ring(gm)	69.27
Wt.of dry sample	17.37	Wt.of dry sample	43.03
Wt.of wet sample	21.25	Wt.of wet sample	58.74
Wt.of water(gm)	3.88	Wt.of water(gm)	15.71
Moisture Content(%)	22.34	Moisture Content(%)	36.51
Bulk Density	1.180		
Dry density	0.980		

Elapsed Time h	Time, Min	Applied Stress, Kpa	Dial Reading, mm	Cumulative Compression, mm	Void Ratio Δe=F*ΔH	e1=eo-Δe
	0	5	0.0	0.000	0.000	1.561
	5	12	7.0	0.014	0.002	1.559
1		25	23.0	0.046	0.006	1.555
2		50	29.0	0.058	0.007	1.554
3		75	59.0	0.118	0.015	1.546
	0.1	75	59.0	0.118	0.015	1.546
	0.25	75	59.0	0.118	0.015	1.546
	0.5	75	59.0	0.118	0.015	1.546
	1	75	63.0	0.126	0.016	1.545
	2	75	67.0	0.134	0.017	1.544
	4	75	72.0	0.144	0.018	1.543
	8	75	79.0	0.158	0.020	1.541
	15	75	87.0	0.174	0.022	1.539
	30	75	91.0	0.182	0.023	1.538
1		75	96.0	0.192	0.025	1.537
2		75	116.0	0.232	0.030	1.532
4		75	128.0	0.256	0.033	1.528
8		75	134.0	0.268	0.034	1.527
24		75	145.0	0.290	0.037	1.524
32		150	267.0	0.534	0.068	1.493
38		300	399.0	0.798	0.102	1.459



Collapse Index, Ie Computation	
d_o, *dial reading at seating stress, mm*	0.0
h_o, initial specimen height, mm	20
d_f, dial reading at 200KPa after wetting ,mm	0.290
d_i, dial reading at 200KPa before wetting, mm	0.118
collapse Index $I_e=((d_f - d_i)/h_o)*100$	0.86

Appendix B: Laboratory Test Results from the Project

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	Document No.: LAB-F-035	Revision No. 0	Effective Date: 10/01/2013

Project:- Modjo–Hawasa Express Road, Ziway–Arsi Negele Section (Lot - III)

Date: 13/06/18

Client:- SBI, International Holding AG

Sample of:- Undisturbed Samples

Test Specified by:- Gondwana Engineering Plc.

Sr. No	Borehole No.	Sample ID	Depth (m)		Sieve Pass % (mm)				LL %	PL %	PI %	AASHTO Soil Class	USCS	Moisture Content %	Bulk Density kg/m³	Dry Density kg/m³	Sp. Gra (Gs)	Saturation %		e_o	Collapse Potential I_e at Kpa			Direct Shear	
			From	To	4.750	2.000	0.425	0.075										Before	After		100	200	300	C (KPa)	Ø Degree
1	BH-93+440	UDS1-93+440	1.50	1.78	100	98	92	49		NP		A-4 (3)	SM	20	1475	1234	2.542	47	87	1.06		0.23		0	27
2		UDS2-93+440	3.00	3.20	100	98	96	94	79	60	19	A-7-5 (16)	MH	19	1286	1081	2.542	36	116	1.35		0.48		6	21
3		UDS3-93+440	4.50	4.73	100	100	93	67		NP		A-4 (6)	ML	-	-	-	-	-	-			-		-	-
4	BH-93+560	UDS1-93+560	1.50	1.70	100	98	94	84		NP		A-4 (8)	ML	26	1164	921	2.589	38	70	1.81	4.21			0	32
5		UDS2-93+560	3.00	3.25	100	100	96	52		NP		A-4 (3)	ML	15	1016	881	2.585	20	83	1.93		4.52		0	29
6		UDS3-93+560	4.50	4.80	100	98	84	60		NP		A-4 (5)	ML	-	-	-	-	-	-			-		-	-
7	BH-93+680	UDS1-93+680	1.50	1.70	98	96	88	68		NP		A-4 (7)	ML	47	923	630	2.607	39	87	3.14		1.36		31	36
8		UDS2-93+680	3.00	3.15	100	100	90	32		NP		A-2-4 (0)	SM	9	1066	977	2.573	14	78	1.63		1.58	2.61	-	-
9	BH+93+800	UDS1-93+800	1.50	1.73	100	100	92	87		NP		A-4 (8)	ML	-	-	-	-	-	-			-		-	-
10		UDS2-93+800	4.50	4.70	99	99	92	46		NP		A-4 (2)	SM	3	944	918	2.560	-	-			-		12	29
11	BH+93+920	UDS1-93+920	1.50	1.75	100	97	95	85		NP		A-4 (8)	ML	28	939	736	2.587	28	93	2.51		1.94		33	21
12	BH-94+040	UDS1-94+040	1.50	1.90	100	100	98	94		NP		A-4 (8)	ML	16	765	659	2.571	14	90	2.90		2.94		47	21
13		UDS2-94+040	4.50	4.70	99	97	89	73		NP		A-4 (8)	ML	-	-	-	-	-	-			-		-	-
14	BH-94+280	UDS1-94+280	1.50	1.85	99	97	94	87		NP		A-4 (8)	ML	22	902	741	2.586	23	109	2.49		2.05		12	32
15		UDS2-94+280	3.00	3.40	100	98	90	67		NP		A-4 (6)	ML	19	1290	1088	2.581	35	91	1.37		2.03		0	32
16		UDS4-94+280	6.50	6.90	100	97	86	77		NP		A-4 (8)	ML	-	-	-	-	-	-			-		-	-
17		UDS5-94+280	8.20	8.50	100	98	85	66		NP		A-4 (6)	ML	4	1261	1214	2.564	9	85	1.11		2.71	3.33	41	22
18	BH-94+400	UDS2-94+400	3.00	3.17	100	100	97	95		NP		A-4 (8)	ML	21	1022	846	2.557	26	95	2.02		0.28		52	25
19	BH-94+520	UDS1-94+520	1.50	1.80	85	80	71	52		NP		A-4 (3)	ML	17	1354	1160	2.565	35	78	1.21		2.42		0	31
20	BH-95+080	UDS2-95+080	4.50	5.00	100	100	97	65		NP		A-4 (6)	ML	29	1396	1080	2.619	54	92	1.43		0.34		0	29
21	BH-95+200	UDS1-95+200	1.50	1.90	100	100	87	47		NP		A-4 (2)	SM	12	1572	1401	2.530	38	86	0.81		0.88		0	21
22		UDS4-95+200	6.00	6.15	100	100	90	59		NP		A-4 (5)	ML	26	1248	992	2.540	42	102	1.56		1.69		0	22

Remark

Reported by:- (Lab. Eng.) Besufekad.T

Approved by:- (Mat. Eng.) 19/06/18

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Gondwana Engineering PLC.

Title	Soil Test Summary	
Document No.: LAB-F-035	Revision No. 0	Effective Date: 10/01/2013

Project:- Modjo–Hawasa Express Road, Ziway–Arsi Negele Section (Lot - III)

Date: 13/06/18

Client:- SBI, International Holding AG

Sample of:- Undisturbed Samples

Test Specified by:- Gondwana Engineering Plc.

Sr. No	Borehole No.	Sample ID	Depth (m) From	Depth (m) To	Sieve Pass % (mm) 4.750	2.000	0.425	0.075	LL %	PL %	PI %	AASHTO Soil Class	USCS	Moisture Content %	Bulk Density kg/m³	Dry Density kg/m³	Sp. Gra (Gs)	Saturation % Before	Saturation % After	e_o	Collapse Potential I_e at Kpa 100	200	300	Direct Shear C (KPa)	Ø Degree
23	BH-95+320	UDS2-95+320	3.00	3.30	100	100	95	11		NP		A-2-4 (0)	SM	3	996	972	2.629							0	22
24	BH-95+440	UDS1-95+440	2.00	2.35	100	84	19	14		NP		A-1b (0)	SM	12	1284	1142	2.607	25	74	1.28		1.31		0	27
25		UDS3-95+440	5.50	5.75	100	100	90	12		NP		A-2-4 (0)	SM	8	988	916	2.663							0	25
26	BH-95+560	UDS1-95+560	3.00	3.30	100	100	87	20		NP		A-2-4 (0)	SM	9	1122	1033	2.560	15	84	1.48		2.45		0	27
27	BH-95+680	UDS1-95+680	1.50	1.75	100	100	88	47		NP		A-4 (2)	SM	8	1265	1173	2.607	17	74	1.22		1.55		0	27
28		UDS2-95+680	3.00	3.35	100	100	100	90		NP		A-4 (8)	ML	10	1163	1057	2.555	18	84	1.42		1.19		0	19
29		UDS4-95+680	6.00	6.40	100	100	96	91		NP		A-4 (8)	ML	27	941	741	2.571			2.47		3.45		0	25
30	BH-95+800	UDS3-95+800	4.50	4.80	100	100	94	65		NP		A-4 (6)	ML	9	1057	967	2.599	14	80	1.69		3.04			
31	BH-95+920	UDS1-95+920	1.50	1.70	100	100	91	45		NP		A-4 (2)	SM	22	1590	1306	2.575	58	76	0.97		0.56		0	37
32		UDS2-95+920	3.00	3.25	100	100	98	19		NP		A-2-4 (0)	SM	10	1107	1009	2.584	16	82	1.56		0.42		0	20
33	BH-96+540	UDS2-96+540	4.50	4.73	100	98	95	46		NP		A-4 (2)	SM	10	1281	1162	2.610	21	84	1.25		2.02		0	27
34	BH-96+660	UDS1-96+660	2.50	2.68	99	96	86	68		NP		A-4 (7)	ML	11	1445	1297	2.621	29	96	1.02	2.15	3.80		0	34
35		UDS3-96+660	5.50	5.85	100	100	100	91		NP		A-4 (8)	ML	11	1171	1053	2.590	20	85	1.46	2.32	4.28		0	22
36	BH-96+780	UDS1-96+780	1.50	1.70	81	74	63	32		NP		A-2-4 (0)	SM	11	1300	1176	2.557	23	91	1.17	3.93	6.25		0	33
37		UDS2-96+780	3.00	3.25	100	100	98	83		NP		A-4 (8)	ML	13	1284	1135	2.557	27	48	1.25		0.76		0	23
38		UDS5-96+780	8.00	8.20	100	96	88	40		NP		A-4 (1)	SM	18	1306	1105	2.676	34	79	1.42		1.70		0	27
39	BH-96+900	UDS1-96+900	1.50	1.80	100	98	90	50		NP		A-4 (3)	ML	17	1440	1228	2.588			1.11		3.26		0	29
40		UDS3-96+900	4.50	4.90	97	95	91	89		NP		A-4 (8)	ML												
41	BH-97+020	UDS2-97+020	3.00	3.25	100	100	97	63		NP		A-4 (6)	ML	6	1207	1136	2.550	13	90	1.25		1.24		0	20
42	BH-98+260	UDS1-98+260	1.50	1.80	100	100	89	45		NP		A-4 (2)	SM	5	1209	1150	2.530	11	77	1.20		0.79		0	22
43	BH-99+420	UDS2-99+420	4.00	4.18	100	95	81	63		NP		A-4 (6)	ML	13	1098	973	2.612	20	83	1.68		1.11		0	34

Remark

Reported by:- (Lab. Eng.) Bereketeab T.

Approved by:- (Mat. Eng.) 19/06/18

Gondwana Engineering Private Limited Company

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		Document No.: LAB-F-035	Revision No. 0		Effective Date: 10/01/2013

Project:- Modjo–Hawasa Express Road, Ziway–Arsi Negele Section (Lot - III)

Date: 13/06/18

Client:- SBI, International Holding AG

Sample of:- Undisturbed Samples

Test Specified by:- Gondwana Engineering Plc.

Sr. No	Borehole No.	Sample ID	Depth (m) From	Depth (m) To	Sieve Pass % (mm) 4.750	2.000	0.425	0.075	LL %	PL %	PI %	AASHTO Soil Class	USCS	Moisture Content %	Bulk Density kg/m³	Dry Density kg/m³	Sp. Gra (Gs)	Saturation % Before	Saturation % After	e_o	Collapse Potential I_e at Kpa 100	200	300	Direct Shear C (KPa)	Ø Degree
44	BH-99+650	UDS1-99+650	1.50	1.85	100	97	89	61	28	24	6	A-4 (5)	ML	16	1628	1404	2.574	49	84	0.83		0.65		0	26
45	BH-102+940	UDS1-102+940	3.00	3.25	100	97	82	35	29	23	6	A-2-4 (0)	SM-SL	9	1162	1063	2.540	17	69	1.39		1.32		0	34
46	BH-104+540	UDS1-104+540	1.50	1.90	93	92	91	85		NP		A-4 (8)	ML	20	1065	890	2.618	26	89	1.94		1.20		2	21
47		UDS2-104+540	4.50	4.80	100	100	95	92		NP		A-4 (8)	ML	23	1052	855	2.573	30	83	2.01		0.97		0	37
48	BH-105+960	UDS1-105+960	3.00	3.30	100	100	92	88		NP		A-4 (8)	ML	34	892	667	2.585			2.87		2.04		0	34
49	BH-107+680	UDS2-107+680	4.50	4.73	96	87	63	46		NP		A-4 (2)	SM	9	1159	1066	2.617	16	114	1.45		2.76		0	31
50	BH-108+480	UDS1-108+480	1.50	2.00	98	90	69	58		NP		A-4 (5)	ML	5	946	899	2.639	7	67	1.93	1.64			0	33
51		UDS5-108+480	7.50	7.70	97	97	95	84		NP		A-4 (8)	ML	6	907	854	2.665	8	78	2.12	0.26			0	26
52	BH-109+280	UDS2-109+280	3.00	3.40	100	97	59	27		NP		A-2-4 (0)	SM	8	1085	1006	2.607	13	73	1.59		3.24		0	33
53	BH-110+060	UDS1-110+060	1.50	2.00	97	87	45	31		NP		A-2-4 (0)	SM	7	1048	980	2.573	11	69	1.63	1.78			0	32
54	BH-110+420	UDS2-110+420	4.50	4.85	86	68	34	23		NP		A-1b (0)	SM	7	966	899	2.608	10	70	1.90		0.79		0	30
55	BH-110+600	UDS1-110+600	2.00	2.50	94	75	38	28		NP		A-2-4 (0)	SM	11	936	843	2.703	14	73	2.21	0.18			0	36
56	BH-111+500	UDS2-111+500	3.00	3.50	97	87	50	26		NP		A-2-4 (0)	SM	12	1125	1005	2.532	20	79	1.52		1.01		0	39
57	BH-115+700	UDS1-115+700	1.50	1.65	100	99	77	63		NP		A-4 (6)	ML	3	1028	993	2.585	6	83	1.60		2.55		0	26
58	BH-117+800	UDS2-117+800	3.00	3.25	100	98	86	65		NP		A-4 (6)	ML	20	1282	1068	2.607	36	96	1.44		1.75		10	17
59	BH-137+600	UDS3-137+600	5.00	5.35	91	82	49	33		NP		A-2-4 (0)	SM	10	904	821	2.644	12	80	2.22		2.43		0	28
60	BH-138+000	UDS1-138+000	10.00	10.85	91	88	73	50		NP		A-4 (3)	ML	2	926	907	2.712	3	74	1.99		0.23		0	36
61	BH-138+000	UDS1-138+000L	1.50	1.75	86	73	48	13		NP		A-1b (0)	SM	11	1422	1276	2.635	28	79	1.06	0.35			0	28
62	BH-148+600	UDS1-148+600	3.00	3.35	99	97	91	68		NP		A-4 (7)	ML	4	1103	1066	2.580	6	81	1.42		1.74		0	25
63	BH-148+900	UDS2-148+900R	4.50	4.75	100	99	90	76		NP		A-4 (8)	ML	2	1172	1151	2.588	4	77	1.25		4.97		0	23
64	BH-148+900	UDS1-148+900L	3.00	3.20	87	85	79	74		NP		A-4 (8)	ML	10	1048	954	2.557	15	80	1.68	2.99			0	28
65	BH-148+900	UDS1-148+900C	1.50	1.75	97	96	90	81		NP		A-4 (8)	ML	7	1106	1038	2.594	11	81	1.50	0.75			0	29

Remark

Reported by:- (Lab. Eng.) Basufekad.T

Approved by:- (Mat. Eng.) 13/06/18 Dawit.K

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	Ô"É³" ›=Í='"Ó ®¾}.¼ÓM Ÿ<v"Á **Gondwana Engineering PLC.**	Title	Soil Test Summary		ISO 9001 : 2015 Certified
		Document No.: LAB-F-035	Revision No. 0	Effective Date: 10/01/2013	

Project:- Modjo–Hawasa Express Road, Ziway–Arsi Negele Section (Lot - III) Date: 22/08/18
Client:- SBI, International Holding AG
Sample of:- Undisturbed Structure Samples
Test Specified by:- Gondwana Engineering PLC.

Sr. No	Station (km)	Sample ID	Depth (m)		Sieve Pass % (mm)				LL %	PL %	PI %	AASHTO Soil Class	USCS	NMC %	Bulk Density kg/m³	Dry Density kg/m³	Sp. Gra (Gs)	e^0	Collapse Potential I_e at Kpa					Direct Shear	
			From	To	4.750	2.000	0.425	0.075											100	200	300	400	800	C (KPa)	Ø Degree
1	BH-ST-25C	ST25C-UD1	4.50	4.70	92	89	81	57		NP		A-4 (4)	ML	14	762	668	2.56	2.84					2.44	0	33
2	BH-ST-27R	ST27R-UD1	2.50	2.75	96	89	60	42		NP		A-4 (1)	SM	6	1163	1101	2.58	1.34					1.14	0	23
3	BH-ST-28R	ST28R-UD1	3.00	3.25	94	90	82	60		NP		A-4 (5)	SM	5	1051	1003	2.51	1.50		0.70				0	32
4		ST28R-UD2	12.00	12.15	80	70	51	36		NP		A-4 (0)	SM	12	1030	918	2.594	1.83		5.65				0	26
5	BH-ST-28L	ST28L-UD1	7.50	7.70	96	89	57	35		NP		A-2-4 (0)	SM	5	892	847	2.541	2.00		1.37				0	33
6		ST28L-UD2	10.50	10.85	95	84	55	36		NP		A-4 (0)	SM	15	845	732	2.509	2.43		2.25				0	29
7	BH-ST-29R	ST29R-UD1	2.50	2.80	98	95	77	48		NP		A-4 (3)	SM	2	951	935	2.538	1.71			0.79			0	29
8	BH-ST-30C	ST-UD1	8.00	8.15	98	97	83	63		NP		A-4 (6)	ML	16	1122	966	2.520	1.61					1.70	0	29
9	BH-ST-30R	ST-UD1	4.50	4.75	100	96	79	44		NP		A-4 (2)	SM	7	867	810	2.594	2.20			0.92			0	30
10		ST-UD2	8.00	8.14	100	97	85	58		NP		A-4 (5)	ML	5	977	927	2.642	1.85				1.77		0	15
11	BH-ST-31C	ST-UD1	3.50	3.75	100	100	95	69		NP		A-4 (7)	ML	17	1348	1157	2.573	1.24		1.06				0	25
12	BH-ST-31R	ST-UD1	3.50	3.70	100	99	95	81		NP		A-4 (8)	ML	21	1427	1181	2.513	1.13		7.26				8	21
13		ST-UD2	6.20	6.55	100	99	81	71		NP		A-4 (7)	ML	17	1435	1226	2.521	1.06		7.41				17	15
14		ST-UD2	10.50	10.70	100	100	98	97		NP		A-4 (8)	ML	21	1163	962	2.510	1.61		0.60				5	20
15	BH-ST-32C	ST-UD1	2.50	3.00	100	100	99	97	46	27	19	A-7-6 (13)	ML	13	1209	910	2.638	1.90		3.16				0	28
16		ST-UD2	4.50	4.65	98	97	92	58		NP		A-4 (5)	ML	13	898	794	2.695	2.39		2.23				0	18
17	BH-ST-32R	ST-UD1	4.50	4.70	100	100	100	14		NP		A-2-4 (0)	SM	5	1118	1066	2.449	1.30		0.48				0	24
18		ST-UD2	6.00	6.20	100	99	95	85	33	22	11	A-6 (8)	ML	21	1324	1097	2.564	1.34		5.41				0	22
19		ST-UD3	10.50	10.70	100	100	82	37		NP		A-4 (0)	SM	18	1310	1106	2.583	1.34		1.98				0	29
20	BH-ST-32L	ST-UD1	2.50	3.00	100	100	98	92		NP		A-4 (8)	ML	21	1089	899	2.561	1.85		1.79				0	27
21		ST-UD2	4.50	4.80	100	100	93	16		NP		A-2-4 (0)	SM	5	1350	1291	2.628	1.04		0.57				0	21
22		ST-UD3	10.50	10.65	100	100	91	25		NP		A-2-4 (0)	SM	2	447	437	2.558	4.86		0.98				0	23
23	BH-ST-33L	ST-UD1	6.00	6.50	73	58	28	13		NP		A-1b (0)	SM	2	909	893	2.559	1.87		0.40				0	37
24	BH-ST-33R	ST-UD1	8.00	8.20	97	95	85	57		NP		A-4 (4)	ML	3	1023	994	2.644	1.66		0.64				0	30
25	BH-ST-33C	ST-UD1	4.00	4.30	100	98	90	82		NP		A-4 (8)	ML	13	990	874	2.532	1.89		2.02				0	31
26		ST-UD2	6.00	6.35	87	73	35	13		NP		A-1b (0)	SM	5	997	947	2.541	1.68		1.67				0	36

Remark

Reported by:- (Lab. Eng.) ______________ Approved by:- (Mat. Eng.) ______________

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	Document No.: LAB-F-035	Revision No. 0	Effective Date: 10/01/2013	

Project :- Modjo–Hawasa Express Road, Ziway–Arsi Negele Section (Lot - III) Date 13/01/19

Client:- SBI, International Holding AG

Sample of:- Disturbed Samples Undisturbed

Test Specified by:- Gondwana Engineering Plc.

Sr. No	Station (km)	Field Material Description	Depth (m)		Sieve Pass % (mm)				LL %	PL %	PI %	AASHTO Soil Class	USCS	NMC %	Cp %	SPT N value	Specific Gravity (GS)
			From	To	4.750	2.000	0.43	0.08									
1	126+900	Medium dense Light grayish fine Sand SILT	1.50	1.75											0.11		2.543
2		Medium dense Light grayish fine Sand SILT	3.25	3.70	82	72	54	23		NP		A-2-4 (0)	SM	19		23	2.543
3		Medium dense Light gray fine Sandy Silt	4.50	4.71											0.49		2.500
4		Medium dense Light gray fine Sandy Silt	4.71	5.16	84	72	52	30		NP		A-2-4 (0)	SM	20		25	2.500
5		Medium Light grey fine Sandy SILT	6.16	6.61	90	77	53	32		NP		A-2-4 (0)	SM	23		23	2.589
6	129+000	Loose to medium dense Light gray fine Sandy SILT	1.00	1.20											0.87		2.500
7		Loose to medium dense Light gray fine Sandy SILT	1.20	1.65	99	96	79	52		NP		A-4 (3)	ML	11		9	2.500
8		Loose to medium dense Light gray fine Sandy SILT	2.50	2.75											0.76		2.511
9		Loose to medium dense Light gray fine Sandy SILT	2.75	3.20	97	93	78	42		NP		A-4 (1)	SM	18		16	2.511
10		Loose to medium dense Light gray fine SILT	4.00	4.37											1.61		2.573
11		Loose to medium dense Light gray fine SILT	4.37	4.82	93	88	77	62	36	29	7	A-4 (5)	ML	21		10	2.573
12		Loose to medium dense Light gray fine Sandy SILT	5.50	5.66											2.38		2.531
13		Loose to medium dense Light gray fine Sandy SILT	5.66	6.11	94	86	69	49		NP		A-4 (3)	SM	12		25	2.531
14	131+900	Medium dense Light brown to gray fine Sandy SILT	1.50	1.75											2.38		2.544
15		Medium dense Light brown to gray fine Sandy SILT	1.75	2.20	95	86	65	47	28	23	5	A-4 (2)	SM/SC	23		11	2.544
16		Medium dense Light brown gray fine Sandy SILT	3.00	3.30											0.04		2.638
17		Medium dense Light brown gray fine Sandy SILT	3.30	3.78	93	83	60	35		NP		A-2-4 (0)	SM	15		14	2.638
18		Medium dense light grayish fine Sandy SILT	4.50	4.87											0.14		2.538
19		Medium dense light grayish fine Sandy SILT	4.87	5.32	98	92	74	42		NP		A-4 (1)	SM	23		13	2.538
20		Medium dense light gray fine Sandy SILT	6.00	6.20											0.99		2.525
21		Medium dense light gray fine Sandy SILT	6.20	6.65	98	93	79	57		NP		A-4 (4)	ML	21		25	2.525
22	133+380	Medium dense light grayish Silty fine SAND	1.00	1.33											0.29		2.512
23		Medium dense light grayish Silty fine SAND	1.33	1.78	65	51	39	28	40	33	7	A-2-4 (0)	SM	14		13	2.512
24		Very dense light yellow Silty fine SAND (Weakly welded tuff)	2.50	2.95	47	43	40	37	57	46	11	A-7-5 (1)	GM	24		50	2.557
25		Very dense light yellowish Silty fine SAND (Weakly welded tuff)	4.00	4.45	47	42	29	19		NP		A-1b (0)	GM	21		50	2.525
26		Very dense light yellowish Silty fine SAND (Weakly welded thick tuff)	5.50	5.70	71	64	53	43	42	34	8	A-5 (2)	SM	14		50	2.512
27	136+020	Medium dense light brown Silty fine SAND	1.50	1.95	91	79	38	23		NP		A-1b (0)	SM	9		14	2.596
28		Medium dense light yellowish grey Silty fine SAND	3.30	3.50											0.15		2.518
29		Medium dense light yellowish grey Silty fine SAND	3.50	3.95	76	72	62	51		NP		A-4 (3)	ML	11		21	2.518
30		Medium dense light yellowish grey Silty fine Sand	4.50	4.80											1.32		2.506
31		Medium dense light yellowish grey Silty fine Sand	4.80	5.25	65	57	49	43		NP		A-4 (2)	SM	19		15	2.506
32		Medium dense light yellowish grey Silty fine Sand	6.00	[illegible]											0.33		2.518
33		Medium dense light yellowish grey Silty fine Sand	[illegible]	6.75	[illegible]	53	44	35		NP		A-2-4 (0)	SM	18		22	2.518

Remark: Collapse tests were not been performed on those samples collected [illegible] station [illegible]+900 at depth of 3.00-3.[illegible]3m and 6.00-6.16m because of presence of over sized gravels and very sandy nature of the samples

Reported by:- (Lab. Eng.) Besufekad

GONDWANA ENGINEERING Private Limited Company

Approved by:- (Mat. Eng.)

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Appendix C: Photos







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