CORRELATION BETWEEN INDEX PROPERTIES WITH SHEAR STRENGTH PARAMETERS FOR HAWASSA SOIL

M Sc THESIS

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ADVISORS' APPROVAL SHEET

This is to certify that the thesis entitled "Correlation between Index Properties with Shear Strength Parameters for Hawassa Soil" submitted in partial fulfillment of the requirements for the degree of Master's with specialization in Geotechnics, the Graduate Program of the School of Civil Engineering, and has been carried out by Batamo Belihu Baye Id. No GTPG/ 009/2006, under our supervision. Therefore we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the School.

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DECLARATION

I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisors Dr. Tensay Gebremedhin and Dr. Feto Esmo has not been presented as a thesis for a degree in any other university. All sources of materials used for this thesis have also been duly acknowledged.

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ABBREVIATIONS, ACRONYMS AND SYMBOLS

AASHTO	American Association of State Highway and transportation Officials
ASTM	American Society for Testing and Materials
c	Cohesion
c’	Effective Cohesion
CD	Consolidated drained
CU	Consolidated undrained
D_{90}, D_{85}, D_{75}, D_{60}, D_{50}, D_{30}, D_{25}, D_{15}, and D_{10}	Particle diameter corresponding to 90 %, 85 %, 75 % 60 %, 50 %, 30 %, 25 %, 15 %, and 10 % finer on the Cumulative particle size distribution curve, respectively
ERA	Ethiopian Roads Authority
HU	Hawassa University
LL	Liquid limit
M_1	Mass of the sand used to fill the test hole, funnel and base plate, g
M_2	Mass of the sand used to fill the funnel and base plate, g
M_3	Moist mass of the material from test hole, g
M_4	Dry mass of material from test hole, g
M_5	Weight of sand in cone pouring cylinder
MDD	Maximum Dry Density
NP	None plastic
NMC	Natural Moisture Content
OMC	Optimum Moisture Content
PI	Plasticity index
PL	Plastic limit
SNNPRS	Southern Nations Nationalities and Peoples Regional State
u_a	Pore air Pressure
UC	Unconfined compression

USCS	Unified Soil Classification System
u_w	Pore water Pressure
V	Volume of the test hole
V_1	Internal volume of calibration container
σ	Normal stress
σ’	Effective normal stress
τ	Shear stress
φ	Angle of internal friction
φ’	Effective internal angle of friction
φ_{ds}	Angle of internal friction found by Direct shear test
φ_{tri}	Angle of internal friction found by Triaxial compression
ω	Water content of the material removed from test hole

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ABSTRACT

Shear strength parameters (angle of internal friction and cohesion) are the key engineering properties of soil. In every situation finding these parameters by laboratory testing or by using advanced equipment may be uneconomical for clients during the preliminary design phase. So it is a common practice in geotechnical engineering to estimate these parameters by employing empirical equations from easily found engineering properties of soil. Correlations and empirical equations developed are site specific, and a good result will be found using them for soils from which they are developed. In this study an attempt was made to correlate shear strength parameters (angle of internal friction and cohesion) for Hawassa city soil from index properties. Detailed analysis was carried out by randomly taking thirty four samples from seven sub cities of Hawassa city. The sampling pits were dug manually using hand tools with plan area of 1.50 m by 1.50 m at depth of 3.0 m from the ground surface. From laboratory test results, most samples were found to be with very low values of density and no plasticity. The soil was classified as cohesionless soil of fine to medium sized sand with group name silty sand, sandy silt, silty, clayey sand, silty, clayey sand with gravel and well graded sand with gravel consecutively. Due to the nature of the soil, cohesion was exempted from correlation. The Influence of easily found index properties of soil on its shear strength parameter was studied, and a regression carried out for the selection of most influencing variables. This parameters identified as the grain size distribution, density, and water content of the soil.

Key words: Angle of internal friction, cohesion, index properties, Hawassa.

1. INTRODUCTION

1.1 BACKGROUND

Soil has very important role to support all structures, like buildings, roads, railway lines, pipelines, etc. The geotechnical properties of soil on which a superstructure is to be constructed must be understood well in order to avoid superstructure and foundation failures, and respectively economic and social crises. One of the most important engineering properties of soil is its ability to resist sliding along internal surfaces within a mass. The stability of structures built on soil depends on the shear resistance offered by the soil along probable surfaces of slippage according to Too, (2012) and Omotoso et.al, (2011) as cited in Roy, Surendra and Dass, Gurcharan (2014). The shear failure of a soil mass occurs when the shear stresses and compressive stresses from superstructure loads exceed the shear and compressive resistance of the soil respectively and also it is notified that the failure in soil occurs not by breaking of particles instead by relative movement of the particles.

Shear strength of soil is “the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it” (Das, B.M, 2010). Roy, and Dass, G. (2014), the shear strength of soil is generally represented by the Mohr–Coulomb failure criterion. According to this criterion, the shear strength of c-φ soil varies linearly with the applied stress through two shear strength components, namely, cohesion and angle of internal friction. The tangent to the Mohr–Coulomb failure envelopes is represented by its slope and intercept. The slope expressed in degrees or radians is the angle of internal friction and the intercept is the cohesion.

The angle of internal friction indicates the degree of friction and interlocking existing among soil particles, and the cohesion represents mainly intermolecular bond between the adsorbed water surrounding each grain (ionic attraction and chemical cementation between soil particles). Accurate determination of the soil shear strength parameters (cohesion and angle of internal friction) is a major concern in the design of different geotechnical structures. These key parameters can be determined either on the field or in the laboratory. The triaxial compression test and direct shear test are the two most common tests used for determining the angle of internal friction and cohesion values in the laboratory.

1.2 STATEMENT OF THE PROBLEM

All the tests required for determination of engineering properties are elaborate and time-consuming. Sometimes the geotechnical engineer may be interested to have some rough assessment of the engineering properties without conducting elaborate tests. This is possible if index properties are determined. Determination of the shear strength parameters by using direct shear test, or triaxial compression test are the common practice in the laboratory. However, experimental determination of these parameters is extensive, cumbersome and costly compared to determination of soil index properties. Further, it is not also always possible to conduct the above tests in every new situation. In order to surmount such pragmatic difficulties, empirical correlations shall be developed and used as a tool to estimate the engineering properties of soils. Empirical correlations are widely used in geotechnical engineering practice as a tool to estimate the engineering properties of soils to save money, time, and to avoid inconveniences by developing statistical models and equations Roy S. and Dass G.(2014) For practical purposes the results of routine index tests and correlations can be used as a first approximation of the soil parameters for use in preliminary design of geotechnical structures, and later as a mean to validate the results of laboratory tests. Provided that the engineering behavior of soil varies from place to place and even with time, and other factors, the empirical formulae developed here are site specific and gives better result for soils from where they are formulated. Not to be misled, we have to be aware of this to use them as they are.

1.3 OBJECTIVES

1.3.1 General Objective

The general objective of this study is to predict shear strength parameters from index properties that can be easily determined from laboratory tests for Hawassa soil.

1.3.2 Specific Objectives

In light with the overall objective, the specific objectives of the research are:

- to determine the index properties of soil such as in-situ density, moisture content, specific gravity, particle size distribution, and Atteberg limits to classify the soil.

- to find Soil shear strength parameters: angle of internal friction and cohesion.

1.4 SIGNIFICANCE OF THE STUDY

Understanding the relationship between soil index properties with shear strength parameters will benefit designers for preliminary estimation while determining shear strength parameters for bearing capacity, slope stability and embankment works for Hawassa soil. It could be useful information source for designers and public building officials prior to detail investigation of the geotechnical property of the soil.

1.5 ORGANIZATION OF THE THESIS

This thesis work contains five chapters, references and appendices, each with detail coverage of specific topics. The first chapter contains the background of the thesis, problem statement, objectives, and scope of the thesis and organization of the study. Chapter two covers literature review which contains an introduction to soil shear strength, shear strength properties, some of previous correlations made by different researchers in the world, and an overview of the geology of Hawassa area. Chapter three covers methods and materials. The fourth chapter covers results and discussion. Chapter five presents summary and conclusion. Finally, lists of reference and appendices presented.

1.6 SCOPE OF THE INVESTIGATION

This thesis is limited to determine soil cohesion and internal friction angle from remoulded specimens by direct shear test and all other index property tests. All the transportation and test methods followed are as per American Society for Testing and Materials (ASTM) standards.

2 LITERATURE REVIEW

2.1 GENERAL

Soil is a complex engineering material, and its properties are not unique or constant. Instead, they vary with many environmental factors, like time, stress history, moisture fluctuation, etc.

2.1.1 Soil Formation

Soil is formed from the physical and chemical weathering of rocks. Physical weathering involves reduction of size without any change in original composition of the parent rock. The main agents responsible for this process are exfoliation, unloading, erosion, freezing, and thawing. Chemical weathering causes both reductions in size and chemical alteration of the original parent rock. The main agents responsible for chemical weathering are hydration, carbonation, and oxidation. Often chemical and physical weathering take place in concert.

2.1.2 Shear Strength of Soil

The basic definition of soil shear strength was given in chapter one. Also mentioned these are two important shear strength parameters, the angle of internal friction and cohesion. Shear strength of soil is a function of the normal stress applied, the angle of internal friction, and the cohesion. The angle of internal friction describes the inter-particle friction and the degree of the particle interlocking. This property depends on soil mineral type, soil particle texture, shape, gradation, void ratio, and normal stress. The frictional component of the soil shear strength cannot exist without any normal stress acting on the soil mass. The cohesion describes soil particle bonding caused by electrostatic attractions, covalent link, and chemical cementation. So, with normal stress, the angle of internal friction, and cohesion, different equations were developed.

2.1.3 Shear Strength Parameters (φ and c)

The internal angle of friction (φ') account for three different contribution of energy expanded during shearing. Sliding frictional resistance between grains, rearrangement and dilation, associates with interlocking and rotation of grains, and particle breakage, as described by Bareither et.al. 2008b cited in (Barrios, D.2010). Mitchell and Soga 2005 cited in (Barrios, D. 2010), explained other factors and phenomena such as mineralogy, grain size, grain shape, and grain size distribution, relative density, stress state, type of tests and stress path, and drainage also contribute to the shear strength of granular soils. The total (undrained) shear strength parameters, c and φ, are designated as cohesion and angle of internal friction, respectively. Undrained shear strength applied where there is no change in the volume of pore water which means no consolidation and are measured in the laboratory by shearing without permitting drainage, Guyer et al. (2011).

The angle of shear strength represents the interlocking between the soil particles whereas cohesion is mainly due to the intermolecular bond between the adsorbed water surrounding each grain, especially in fine grained soil. As the soil grain size increases, the soil internal friction angle increases and its cohesion decreases, as (Murthy 2008; El-Maksoud, 2006, Mousavi et.al, 2010, Jain et al. 2010, Lun (2011) cited in Roy S. & Dass G.(2014) Therefore, in a rational manner the main parameters which affect the soil strength parameters are the soil type, soil plasticity, and soil density. The angle of internal friction depends upon dry density, particle size distribution, and shape of particles, surface texture, and water content whereas cohesion depends upon size of clayey particles, types of clay minerals, valence bond between particles, water content, and proportion of the clay. Shear strength is affected by several factors such as grain size distribution, density, water content and others.

2.1.4 Mohr - Coulomb Failure Criterion

Mohr (1900) quoted in Das (2010), theory for rupture in materials contended that, a material fails because of a critical combination of normal stress and shear stress and not from either maximum normal or shear stress alone. “The failure in soil occurs by relative movements of the particles and not by breaking of the particles”, Arora, (2004). Thus, the functional relationship between normal stress and shear stress on a failure plane can be expressed in the following form:

$$\tau_f = f(\sigma) \quad \text{--} \quad 2.1$$

Taking the approximate shear stress on the failure plane as a linear function of the normal stress

$\tau_f =$ c + σtanφ ……………………………………………………………...... 2.2

where, τ_f = shear strength; c = cohesion;σ = normal stress applied on the failure plane; and φ = angle of internal friction.

2.1.5 Pore Water Pressure in Soil

Saturated soil have water filling all of the void spaces. This leads to the concept of effective and normal stress. When a column of saturated soil is subjected to load, the total stress at appoint is carried by both the soil particles and the pore water. So in equation form it is described as:

$\sigma = \sigma' + u$ -- 2.3

where, σ = total stress; σ' = effective stress carried by the soil solids; and

u = pore water pressure.

The effective stress concept can be explained by the soil particles acting as a connected skeleton to support the load. Therefore, the effective stress is often directly proportional to the total stress. Also, the shear failure envelope formula, in equation 2.2 above, can be addressed in terms of effective stresses for saturated soil:

$\tau_f' = c' + \sigma'(\tan\varphi)$-- 2.4

Where c' = effective – stress cohesion; and

φ' = effective – stress angle of internal friction.

In the field, however, soil may be only partially saturated. As Masada T. (2009) cited Bishop et al, (1960) given the following equation to describe the shear strength of unsaturated soil.

$\sigma' = \sigma - u_a - \text{x}(u_a - u_w)$ -- 2.5

Where u_a = pore air pressure; x = degree of saturation; and u_w = pore water pressure.

So going back to equation 2.4 and the shear strength at failure for unsaturated soil can be written:

$\tau_f = c' + [\sigma - u_a + \mathrm{x}(u_a - u_{w)}](\tan\varphi')$ -- 2.6

For soil that is completely dry (x = 0),

$\tau_f = c' + (\sigma - u_a)(\tan\varphi')$ -- 2. 7

For soil that is 50 % saturated (x = 0.5)

$\tau_f = c' + (\sigma - 0.5\, u_a - 0.5\, u_{w)})(\tan\varphi')$ -- 2.8

Finally, for soil that is 100% saturated(x =1):

$\tau_f = c' + (\sigma - 0.5\, u_{w)})(\tan\varphi')$--- 2.9

Typically, u_a is less than 0 and u_w is greater than 0. Experiments done by Casagrande and Hirschfeld (1960) cited in Masada T. (2009) revealed that unsaturated soil has greater shear strength than the same soil in a saturated condition. In some cases the unsaturated state may be temporary, and the soil may become eventually saturated due to surface precipitation and subsurface drainage events. Therefore, it is conservative to design structures using the shear strength of saturated soils. Thus, the above equations are the expressions of shear strength based on total stress and effective stress. The value of cohesion for sand and inorganic silt is zero. For normally consolidated clays, c can be approximated to be zero. Over consolidated clays have values of c that are greater than zero. The angle of friction, φ’ is sometimes referred to as the drained angle of friction.

Table 2.1 Typical values of drained angle of friction for sand and silts, DAS (2010)

Soil type	$\boldsymbol{\varphi'}$(**deg**)
Sand: Rounded grains	
Loose	27 - 30
medium	30 - 35
Dense	35 - 38
Sand: Angular grains	
Loose	30 -35
medium	35 - 40
Dense	40 - 45
Gravel with some sand	34 - 48
Silts	26- 35

2.2 PREVIOUS INVESTIGATIONS ON HAWASSA SOIL

Previously conducted studies have attempted to study the classification, shear strength and the geologic formation of Hawassa city. According to Eyob Teferi (2011), “the soil is more likely to be silt soil of no or very low plasticity with group symbol ML and group name sandy silt” and specific gravity ranging from 2.49 to 2.51.

Ayalew Gashaw (2012), classified the soil, as silt and sandy silt with group symbol ML, the specific gravity range become 2.428 to 2.547, consistent with the Eyob Teferi’s work, and he expressed his observation for lower values of density and specific gravity observed, and he tried to check whether the soil is organic or not and finalized his work on shear modulus and damping ratio values of soil commonly found in Hawassa.

Meseret, G/Wahid (2007) tried to explain, the existence of volcanic tuff and ash intercalated with the soil formations is an indication that volcanism and sedimentation were overlapped in this area at some time in the past. Moreover, the presence of sand size grains indicates that rivers from highland areas were feeding sediments to the lake. Most of the soil in the city are of lacustrine origin that are derived from volcanic rocks. They are of Quaternary age and mainly fine to medium textured. Even though, I have strong criticism and reservation on the classification that soils with no plasticity and cohesion to be classified as red clayey soil. The soil of Hawassa city classified based on the origin, distribution and their stratigraphic positions in to five groups. Alluvium and Flood plain deposits, Colluvium and slope debris, Soil developed over lacustrine deposits and ignimbrites, Soil developed over tuffaceous rock and flat topography, and Red clayey soil, Geological Survey of Ethiopia (2000).

2.3 PREVIOUS CORRELATIONS FOR φ AND c

“Researchers have been compiling and analyzing geotechnical data for many years to provide supporting evidences for new theories, develop new useful empirical correlations, or validate existing theories or relationships. Several different mathematical functions or models were applied to best represent the correlations existing among geotechnical data”, Masada T. (2009). Some of the empirical equations or models developed for different type of soil for particular circumstances of soil type, water content, dry density and other soil properties.

Roopnarine et al. (2012), developed a linear relationship between percentage of clay and sand content with peak and residual friction angles. FA (friction angle) and SS (shear strength). P for peak and R for residual friction angles,

FA – P = 24.5 – 0.159 % clay + 0.357 % sand

FA – R = 92.8 – 0.886 % clay - 0.723 % sand

Roy S. & Dass G. (2014), formulated a linear relationship of cohesion with specific gravity and angle of internal friction with bulk density of soil.

Cohesion = - 0.525+ 0.241*Specific gravity

Angle of internal friction = - 29.604+ 34.220*Bulk density (g/cm^3)

Guyer et al. (2011), given suggestion for sandy soil, cohesion is negligible and relatively its high permeability of sand, the angle of internal friction is usually and solely based on drained tests. The angle of internal friction of sand is primarily affected by the density of the sand and normally varies within the limits of about 28 to 46 degrees:

φ = 30 + 0.15 DR for soils with less than 5percent fines

φ = 25 +0.15 DR for soils with more than 5percent fines.

Brinch Hansen, (1970) and Teferra, (1975) cited in (Braja M. Das (2014), suggested the following empirical correlations for φ' depending based on large database for granular soil:

φ'(deg) = $26^o + 10\,D_r + 0.4\,C_u + 1.6\log(D_{50})$.

φ' (deg) = $\tan^{-1}\left(\frac{1}{ae+b}\right)$

where, D_r = relative density (fraction)

Cu = Uniformity coefficient, e = void ratio

$a = 2.101 + 0.097\left(\frac{D_{85}}{D_{15}}\right)$, $b = 0.845 - 0.398a$

D_{85} and D_{15} = diameters through which, respectively, 85 % and 15 % of soil passes.

D_{50} = mean grain size, in mm (which is the diameter through which 50 % of the soil passes)

Sorensen et al. (2013), suggested simple correlations between plasticity index (PI) and drained peak shear strength parameters in terms of φ'_{OC} and c'_{OC} have been proposed on the basis of a comprehensive database of triaxial compression tests on undisturbed over consolidated Danish clays of very low to extremely high plasticity by the Danish Geotechnical Institute over the decades and proposed the following correlations for the relationship and the interrelation to the effective friction angle and effective cohesion are:

Lower boundary estimate:

4 < PI < 50 φ'_{OC}= 44-14*log PI (deg.)

50 ≤ PI < 150 φ'_{OC}= 30-6*log PI (deg.)

Best estimate:

4 < PI < 50 φ'_{OC} = 45-14*log PI (deg.)

50 ≤ PI< 150 φ'_{OC}= 26-3*log PI (deg.)

For heavily over consolidated clays with 7 % < PI < 80 % the test data indicated a cautious lower bound estimate of the relationship between c'_{OC} and the plasticity index, PI given by the following equations depending on the value of PI:

7 % < PI < 30 % c'_{OC} = 30 (kPa)

30 % ≤PI < 80 c'_{OC} = 48 – 0.6 PI(kPa)

PI > 80 c'_{OC}= 0 (kPa)

Perumal et al. (2015), summarized their investigation on cohesion less soil sample collected from Cauvery river basin in Tamil Nadu. The results were from both triaxial and direct shear tests. ID stands relative density.

Table 1.2 ummary of correlations, Perumal et al. (2015)

Method of correlation	Observation	correlation equation	Strength (%)
Trend line concept	Triaxial test	φ_{tri} = 24.08° + 0.171 ID	99.4894
	direct shear test	φ_{ds} = 23.65° + 0.167 ID	99.6894

Table 2.3 Summary of collected previous correlation equations between different soil properties and shear strength parameters

No	Equation	Applicability (soil type)	Reference
1	FA – P = 24.5 – 0.159 % clay + 0.357 % sand FA – R = 92.8 – 0.886 % clay - 0.723 % sand	clay and sand	Roopnarine et al. (2012)

2	Angle of internal friction = - 29.604+ 34.220*Bulk density (g/cm^3) Cohesion = - 0.525+ 0.241*Specific gravity	Cohesionless soil	Roy & Dass (2014)
3	φ = 30 + 0.15 DR > soil with less than 5 % fines φ = 25 +0.15 DR > Soil with more than 5 % fines	Sandy soil	Guyer et al. (2011)
4	φ'(deg) = 26^o +10 D_r +0.4 C_u +1.6 log (D_{50}).		Das, B.M (2014)
5	φ' (deg) = $\tan^{-1}(\frac{1}{ae+b})$, where a = 2.101 +0.097($\frac{D_{85}}{D_{15}}$), b = 0.845 – 0.398a	Cohesionless soil	"
6	4 < PI < 50 φ'_{OC}= 44-14*log PI (deg.) 50 ≤ PI < 150 φ'_{OC}= 30-6*log PI (deg.)	Overconsolidated clays	Sorensen et al. (2013)
7	7 % < PI < 30 % c'_{OC} = 30 (kPa) 30 % ≤PI < 80 c'_{OC} = 48 – 0.6 I_p (kPa) PI > 80 c'_{OC}= 0 (kpa)	"	"
8	φ_{tri} = 24.08° + 0.171 ID, φ_{ds} = 23.65° + 0.167 ID	Cohesionless soil	Masada, T. (2009)
9	φ = 67.712+0.09*%Gravel*+0.252*%Sand - 0.524*%Compaction	A-6 b	"
10	$\varphi = \frac{53.46\ PI - 660.9}{PI}$	A-6 b	"
11	$\varphi = \frac{53.46\ \omega - 660.9}{\omega}$	A-6 b	"
12	φ = - 0.007(%clay)^2 + 0.981(%clay) - 9.239	A-6 b	"
13	φ = 15.51ln(%clay) - 39.27	A-6 b	"
14	$\varphi = \frac{-560}{\%clay} + 32.31$	A-6 b	"
15	φ = - 0.649*(%compaction)+85.98	A-6 b	"

3 MATERIALS AND METHODS

To accomplish the objective of this study different field and laboratory works were conducted and finally, a computer program (SPSS 20) was used to correlate angle of internal friction and cohesion with index properties. In this chapter, the study area description, sampling and test methodology used were explained.

3.1 DESCRIPTION OF THE STUDY AREA

Hawassa city, is located in the Main Ethiopian Rift (MER) and it is the administrative center for the Southern Nations Nationalities and Peoples Regional State (SNNPRS) and Sidama Zone. The city is located within the geographical co-ordinates of 6^0 45’ to 7^0 15’N and 38^0 15’ to 38^0 45’ E.

The city administration has an area of 157.2 sq. kilo meters, divided in to 8 sub-cities and 32 Kebeles, Anasimos T. (2013). The climate in the area is dry to sub-humid.

Table 2.1 Average rainfall for Hawassa in mm, Anasimos, T.(2013)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Now	Dec
Average precipitation	12	27	54	84	75	84	69	90	90	45	15	9
Average rainfall	3	9	15	15	14	13	15	16	19	11	6	4

Table 3.2 Average high or low temperature for Hawassa in ^{o}c, Anasimos, T.(2013).

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Now	Dec
Average High	21	20	23	21	20	19	19	19	21	21	22	21
Average low	10	10	12	13	13	13	13	13	12	11	9	10

3.2 GEOLOGY OF HAWASSA AREA

Geological survey of Ethiopia (2000), explained the Hawassa area in the way that: “the oldest rocks produced by voluminous eruptions of a rhyolitic magma categorized rhyolitic ignimbrites, basaltic lavas of several tens of meters probably overlying the earliest layers of re-sedimented pumice filling the Hawassa Caldera. The later re-sedimented pumice deposits overlay marginal parts of the basaltic ridge, others are Basaltic lapilli-stone (tuff-cone) form the Tabor volcanic cone within the city of

Hawassa, unnamed volcanic crater on the southern skirt of Hawassa city and an unnamed hill representing an erosional remnant of a tuff cone to the west of the road from Hawassa to Shashemene. Finally, basaltic scoria (scoria cones) poorly sorted and clast supported, dominated by fragments of scoria forming two small cones along the Hawassa bypass".

3.3 SAMPLING LOCATIONS

During selecting possible sampling locations, points taken under consideration were: Hawassa city is bounded at West by the Lake, at North and East by Oromia region. It is developing in South direction, which is Tula to Shebedino woreda. Sub cities in this direction are like Menaheria and Tabor, especially Tabor sub city is free of old buildings and Kebele owned houses. Every construction starts from new plot. Therefore, these places need detail survey for construction and decided to have more samples from Tabor and Menahria sub cities and few samples from other sub cities. Thirteen sites from Tabor, ten sites from Menahria, each three sites from Misrak and Addis Ketema, each two places from Mehal ketema and Bahl Adarash and one site from Hayki Dar sub city.

After deciding the sampling sites, the major factor considered during selecting depth of pits was the depth of shallow foundations of buildings, which are placed at 2.0 m to 3.0 m depth. The Hawassa city administration building regulatory department has a principle for soil tests to be taken from 3.0 m for shallow foundations like up to ground plus three.

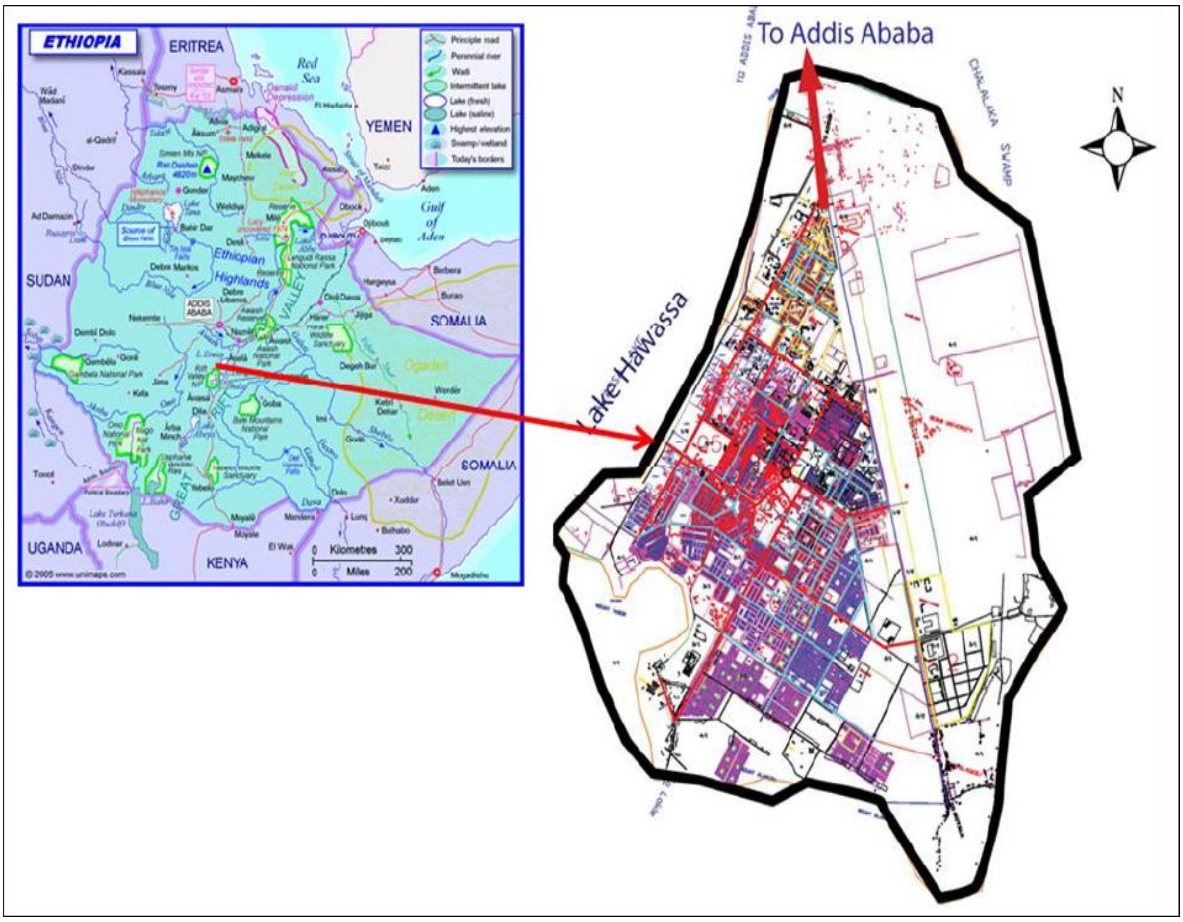

Figure 3.1 Location of the study area, Anasimos T.(2013).

3.4 SAMPLE COLLECTION, PRESERVATION AND LABORATORY PROCEDURES

During the field work, from selected 34 places, disturbed samples were collected at 3.0 m depth from ground surface. The preserving and transporting method of the samples were done according to ASTM D - 4220-95 (Standard Practice for Preserving and Transporting of Soil Samples). All soil samples were properly marked on each sample container to accurately identify the place of the sample and the name of the sampling area. Different tests performed for different soil properties like field density, natural moisture content, specific gravity, grain size analysis, liquid limit, plastic limit, plasticity index and shear strength parameters; cohesion and angle of internal friction. All test locations with their geographic coordinates are shown below.

Table 3.3 Test Locations with their geographic coordinates

Name of test pits	Code	Northing	Easting	Elevation
Etab Soup Factory	Test #01	777035	445281	1728
Bale Wold Church	Test #02	775713	441612	1708
Piazza UN Office	Test #03	778840	441802	1702
Textile Factory	Test #04	778093	445324	1727
New Bus Station	Test #05	775619	444204	1737
Site Six condominium	Test #06	776828	444541	1742
Addisu Gebeya	Test #07	775830	444249	1760
Poultry farm	Test #08	776006	442967	1746
Infolink College	Test #09	780545	443011	1724
Finance Bureau	Test #10	781305	443374	1708
City Administration New	Test #11	776724	444009	1756
Stadium (New)	Test #12	778268	443830	1717
Supreme Court	Test #13	780063	443783	1729
Bureau of Water resource	Test #14	779347	443959	1718
Alamura KHC	Test #15	776894	443526	1733
TTC/polytechnic	Test #16	776993	441605	1704
Membo Condominium	Test #17	777176	442061	1703
Blen Eye clinic	Test #18	776673	441489	1706
Diaspora ‘Chefe’)	Test #19	778748	445807	1716
Diaspora St. mary Church	Test #20	779276	445694	1709
IoT building HU	Test #21	778971	444819	1714
Shell Feul Depot	Test #22	778725	444196	1711
Debub Academy	Test #23	777403	444119	1738
Cathedral Church	Test #24	778410	440881	1705
‘Buna Board’	Test #25	775627	444604	1743
Near Alto School	Test #26	775756	441928	1729
Yeshi Hotel	Test #27	778677	443351	1717
ST.Gorge church	Test #28	775410	444598	1764
SNNPRS Office	Test #29	778426	444631	1720
Gored Gored Hotel	Test #30	678085	442122	1710
Bureau of Education	Test #31	779166	443441	1715
Old AirPort	Test #32	781446	442615	1705
Haile resort	Test #33	782569	442651	1697
South Spring	Test #34	782639	443223	1704

3.5 LABORATORY TEST METHODS

For this study all tests undertaken in laboratory according to American Society for Testing and Materials, ASTM Standards.

3.5.1 Moisture Content Test

Moisture content of a soil has a direct effect on its strength and stability. The moisture content of a soil in its natural state is termed as ‘Natural moisture content’ which characterizes its performance under the action of load and temperature. The test carried out in laboratory as per the procedures of, ASTM (1998). In all works, to be representative the minimum moist mass of material was not less than 20 grams. For all tests conducted, coefficients of variation was found below 2.7 % or below 7.8 % of their mean, and no test was repeated.

3.5.2 Specific Gravity Test

According to ASTM (D 854 - 98), the test method covers determination of the specific gravity of soil that passes the 2.0 mm or No. 10 sieve, in order to use the result in connection with hydrometer portion of the test, by means of a pycnometer. The two procedures used for performing the specific gravity test.

Method A—Procedure for Oven-Dry Specimens and

Method B—Procedure for Moist Specimens,

For specimens of organic soil and highly plastic, fine-grained soil, tropical soil, and soil containing halloysite, Procedure B shall be the preferred method. In this work both methods performed and the results compared.

According to ASTM Designation 854 - 98, there is a criteria for judging the acceptability range of two results obtained by single operator. Precision for cohesive soil is 0.06 and no acceptable range index for non-cohesive soil. But throughout my work the material is non-cohesive and it was set an acceptable range index for two results 0.02. Otherwise the test was repeated.

3.5.3 Direct Shear Test Procedure

Since three or more specimens were prepared for test, the samples were sieved from 2.0 mm sieve and remoulded in a square shear box of 60 mm x 60 mm x 20 mm. The samples were prepared at water content and in-situ density the same as in the field conditions during the test, with respect to the fact, that the conditions in the future might not remain constant due to natural moisture variations. Based on the experiment, the graph between normal stress and shear stress was plotted to determine the cohesion and angle of internal friction. All details of the test was done according to ASTM standards of D 3080 -72.

During the test, recording the data continued till the specimen fails, which is indicated by proving – ring and dial gauge begins to descend after having reached the maximum or peak value. For soil which do not give a peak value, it is assumed to have failure occurrence, when a shear strain of 6 % to 10 % is reached.

3.5.4 Compaction Test

With the development of heavy rollers and their use in field compaction, the standard Proctor test was modified to better represent field conditions. The modified Proctor test is the revised version according to ASTM Test Designation D-1557 and AASHTO Test Designation T-180. To have the maximum possible compactive effort, increased density and decreased water content, in this work, modified Proctor test method 'C ' was used for each thirty four samples.

3.5.5 Grain size analysis Test: mechanical sieving and Hydrometer test

Grain size analysis is a useful identification system for soil grain size, and its gradation. Grain size analysis performed for each thirty four samples which are at a depth of 3.0 m. A hydrometer analysis of the fraction passing sieve # 200 was also conducted (appendix E), following the procedures stipulated in ASTM D 422 (Standard Method for Particle Size Analysis of Soils).

3.6 ANALYSIS METHOD USED

Generally in this analysis procedure the value of shear strength parameters: angle of internal friction, φ and cohesion, c are considered as the dependent variables whereas the in-situ density, natural moisture content, D_{60}, D_{50}, D_{25} D_{15}, D_{10}, percentage fine sized sand particles, percentage silt sized particles, fine percentage passing No. 200

sieve, coefficient of curvature, Cc and clays sized particles in percentage values were the independent (Predictor) variables.

In carrying out the statistical analysis, both the statistical software program called SPSS and MS excel spreadsheet are used to determine the scatter plot, to inspect the predicted value with measured values and the earlier used for regression. The MS excel spread sheet is found to be manageable tool for scatter plot and determination of correlation between few number of variables.

The study of residuals or error is very important in deciding the adequacy of the statistical model. If the error shows any kind of pattern, then it is considered that the model is not taking care for all the systematic information. Residuals should be random, and it should follow the normal distribution with zero mean and constant variance in addition to R square or adjusted R square according to, Goyal et al. 2006; Montgomery et al. 2003. Grivas and Chaloulakou 2006 and Papanastasiou et al. 2007 as cited in Roy Surendra and Dass Gurcharan, (2014). It can be referred from appendices.

4 RESULTS AND DISCUSSION

4.1 LABORATORY TEST RESULTS

Index properties of soil which are not of primary interest to the geotechnical engineer but which are indicative and gives a clear piece of information for the engineering properties of soil. Soil is classified and identified based on index properties. All the tests performed in accordance with the methods described in chapter thee and some of the results are represented below indifferent forms.

4.1.1 Compaction Test Results

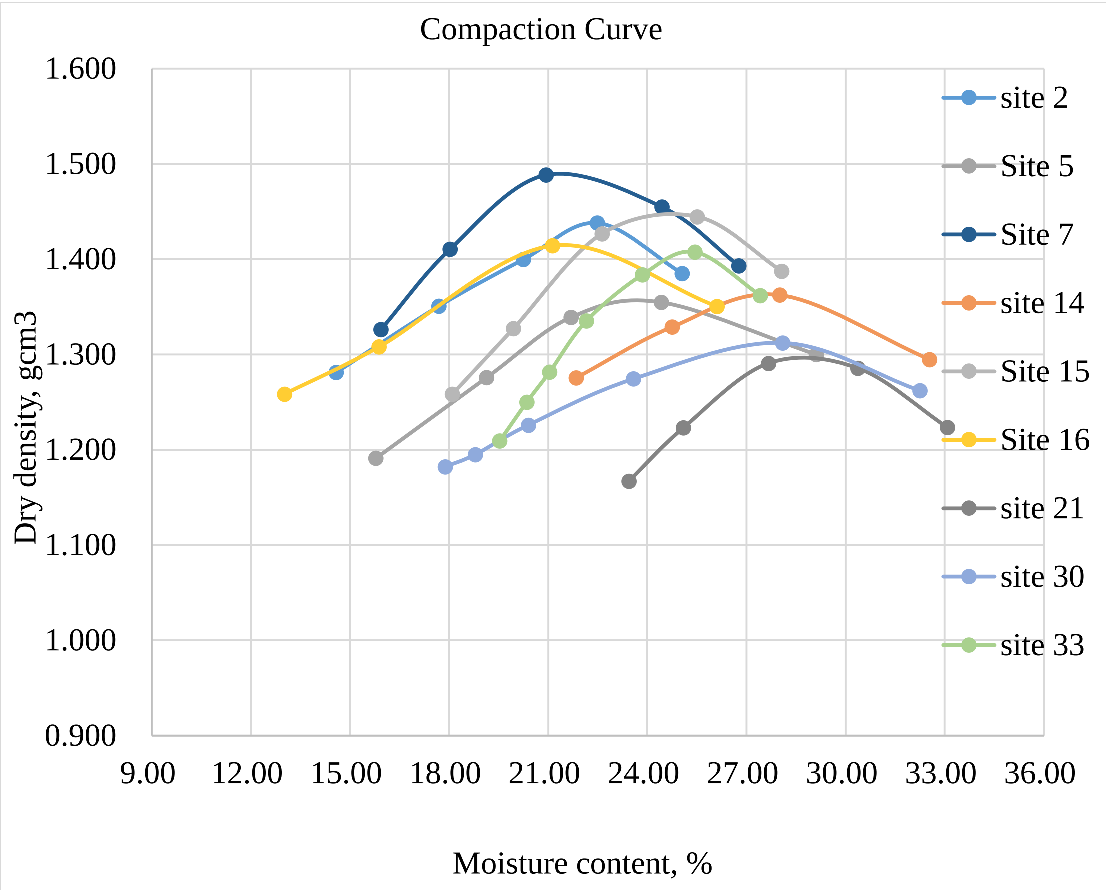

Figure 4.1 Some of Moisture Content vs. Density Relationship

4.1.2 Combined Gradation Test Result

Mechanical Sieving and Hydrometer, combined analysis were performed on all the samples and a plot of percent finer against size of soil particle in millimeter on a semi-log scale was plotted.

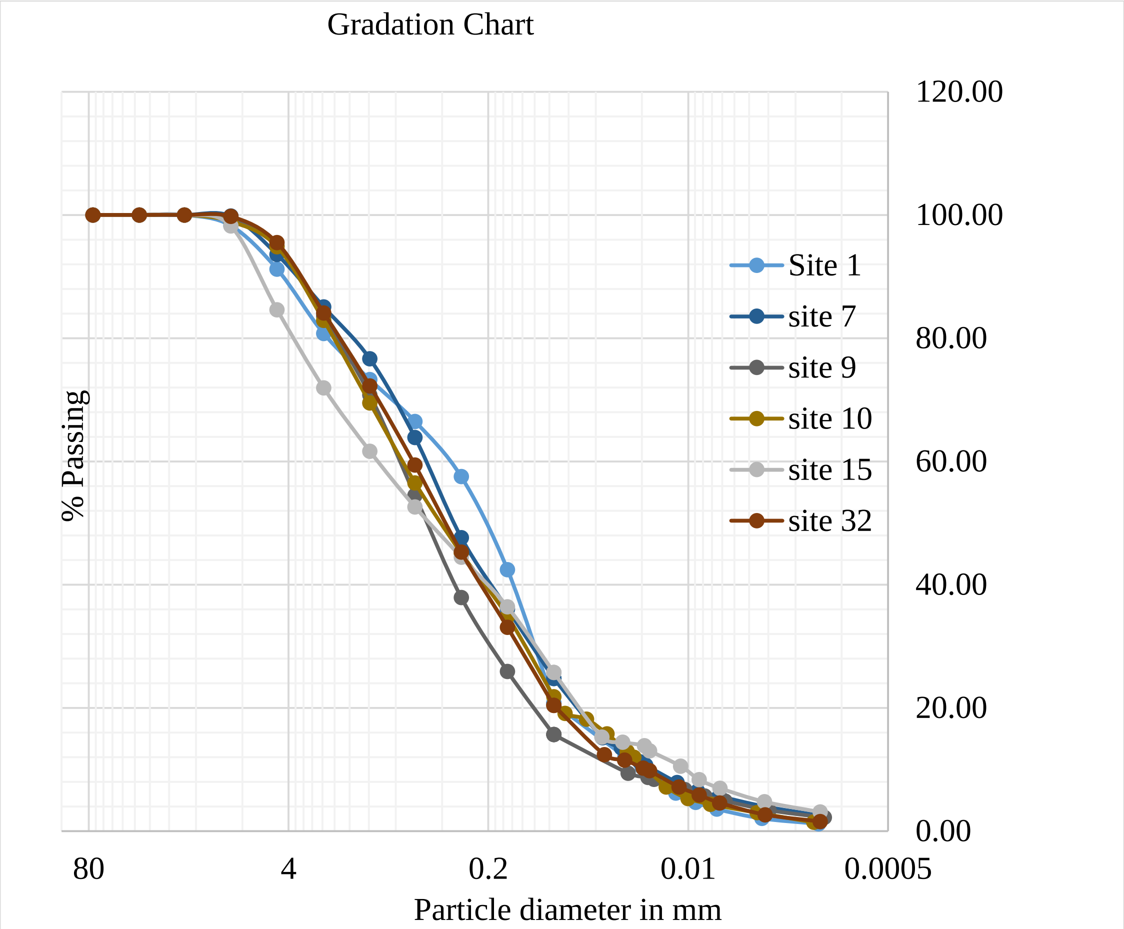

Figure 4.2 some of combined grain size analysis charts

4.1.3 Direct Shear Test

Shear strength parameters; angle of internal friction and cohesion were determined by direct shear test equipment by applying different normal loads of 100 kN, 200 kN, and 300 kN. The test results were ranged 23^o to 35^o. But almost all the results recorded were 27^o to 31^o for angle of internal friction with average of 29^o and nearly no cohesion values recorded.

Table 4.1 Typical direct shear test result for test #3

Horizontal displacement Reading	100 kPa		200 kPa		300 kPa	
	proving -ring reading,(Div)	Shear stress (kPa)	Proving -ringreading,(Div)	Shear stress (kPa)	Proving -ringreading ,(Div)	Shear stress (kPa)
0	0	0.00	0	0.00	0	0.00
0.5	0	0.00	45	16.25	125	45.14
1	15	5.42	90	32.50	265	95.69
1.5	35	12.64	135	48.75	365	131.81
2	55	19.86	176	63.56	410	148.06
2.5	80	28.89	225	81.25	450	162.50
3	101	36.47	250	90.28	475	171.53
3.5	117	42.25	275	99.31	493	178.03
4	126	45.50	300	108.33	505	182.36
4.5	140	50.56	330	119.17	515	185.97
5	148	53.44	340	122.78	525	189.58
5.5	155	55.97	360	130.00	533	192.47
6	161	58.14	358	129.28	545	196.81
6.5	167	60.31				
7	170	61.39				
7.5	160	57.78				
8	158	57.06				
8.5						
9						
9.5						
10						

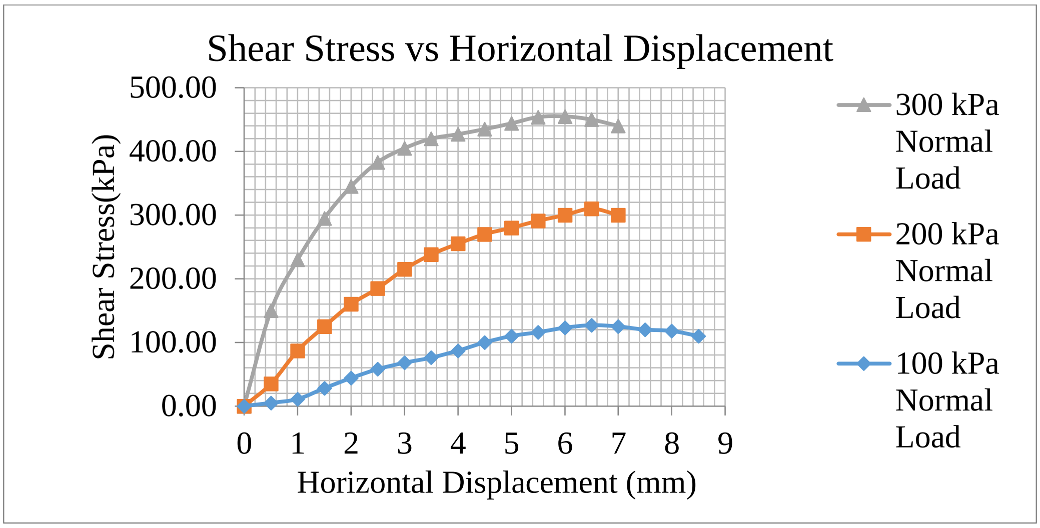

Figure 4.3 Shear stress vs horizontal displacement

Table 4.2 Normal stress and Shear stress

Normal stress(kPa)	Shear stress(kPa)			Failure			unit
0	0	0		at 8.33	φ	32.54	o
100	53.44	61.39		% strain	c	0	kN/m^2
200	122.78	130.00		at peak	φ	33.38	o
300	189.58	196.81			c	0	kN/m^2

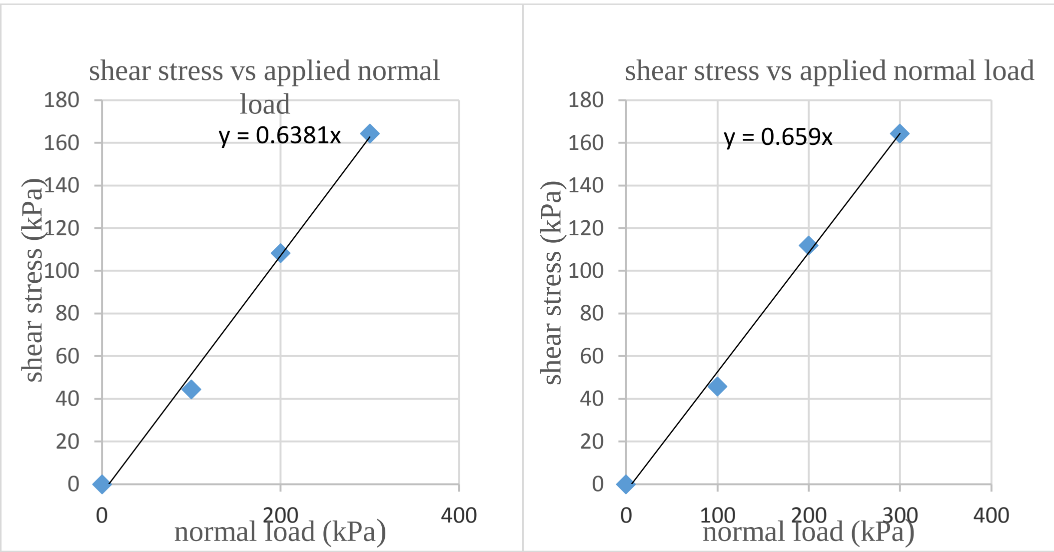

Figure 4.4 Normal stress vs Shear stress

4.1.4 Results for the Specific Gravity Test

A representative test result of specific gravity of study area:

Table 4.3 Specific Gravity oven dried for Test #10

Method A Oven Dry Specimen	A	B	C
Pycnometer ID	2	3	5
Measured temperature of used water, T_x in (T ^{o} C)	26	26	26
Weight of Pycnometer, M_f(g)	130.9	153.62	138.34
Weight of sample, M_o(g)	102.52	100.51	104.28
Weight of pycnometer with full of water, M_a(g)	628.59	652.18	637.14
Weight of pycnometer + sample + water, M_b(g)	685.04	707.48	694.60
Conversion factor, K	0.9986	0.9986	0.9986
Specific gravity(Gs) at 20ºC = $KM_o/(M_o+(M_a-M_b))$	2.222	2.220	2.224
Average Specific gravity(A+B+C)/3	2.20		

Table 4.4 Specific Gravity moist for Test #10

Method B Moist Specimen	A	B	C
Pycnometer ID	2	3	5
Measured temperature of used water, T_x in (T ^{o} C)	25	25	25
Weight of Pycnometer, M_f(g)	130.9	153.62	138.34
Weight of sample, M_o(g)	156.64	157.07	158.7
Weight of pycnometer with full of water, M_a(g)	628.72	652.28	637.32
Weight of pycnometer + sample + water, M_b(g)	715.82	739.48	725.52
Conversion factor, K	0.99880	0.9988	0.9988
Specific gravity(Gs) at 20ºC = $KM_o/(M_o+(M_a-M_b))$	2.250	2.245	2.248
Average Specific gravity(A+B+C)/3	2.25		

Table 4.5 summary of in-situ density, specific gravity and compaction test results

Parameters	Minimum	Maximum
In-situ density	0.998 g/cm^3	1.477 g/cm^3
Natural Moisture Content	7.62 %	38.66 %
Field dry density	0.8 g/cm^3	1.12 g/cm^3
Optimum Moisture Content	19 %	35 %
Maximum Dry density	1.18 g/cm^3	1.49 g/cm^3
Specific Gravity (oven dry)	2.22	2.55
Specific Gravity (moist)	2.25	2.67

The values of in-situ density, field dry density, maximum dry density and specific gravity conducted in the laboratory by both methods were very low. Specially, some field dry densities are below the density of water at standard temperature, which means the soil solid was floating in the water. During specific gravity test it was seen that, most of the soil sample was extremely turbid and never settles after twelve hours soaking.

4.2 SOIL CLASSIFICATION

Soil classification is an important aspect of laboratory test, which tells the characteristic of the soil under interest. There are different methods of classification based on the identification tests performed on the soil. Unified Soil Classification System (USCS) and the American Association of State Highway Transport Officials (AASHTO) method are among the widely used schemes of soil classification. The former method is used in this study for all thirty four samples collected from different parts of the city and summarized below.

Table 4.6 Summary of soil classification according to USCS

Name of test pits	sand category	USCS	
Etab Soup Factory	fine sand	silty sand	SM
‘Bale Wold’ church	fine sand	sandy silt	ML
Piazza UN Office	fine sand	Silty Sand	SM
Textile Factory	medium sand	Silty Sand	SM
New Bus Station	fine sand	Silty Sand	SM
Site Six condominium	fine sand	Silty Sand	SM
‘Addisu Gebeya’	medium sand	Silty Sand	SM
Poultry farm	fine sand	Silty Sand	SM
Infolink College	medium sand	Silty Sand	SM
Finance Bureau	medium sand	Silty Sand	SM
City Administration New	fine sand	Silty Sand	SM
Stadium (New)	fine sand	Silty Sand	SM
Supreme Court	medium sand	Silty Sand	SM
Bureau of Water resource	medium sand	silty, clayey sand	SC - SM
Alamura KHC	medium sand	silty, clayey sand with gravel	SC - SM
TTC/polytechnic	fine sand	sandy silt	ML
Membo Condominium	fine sand	sity sand	SM
Blen Eye clinic	fine sand	sity sand	SM
Diaspora ‘Chefe’)	medium sand	sity sand	SM
Diaspora St. mary Church	fine sand	sity sand	SM
IoT building HU	fine sand	sity sand	SM
Shell Feul Depot	fine sand	sity sand	SM
Debub Academy	fine sand	sity sand	SM
Cathedral Church	fine sand	silty, clayey sand	SC - SM
‘Buna Board’	fine sand	sity sand	SM
Near Alto School	medium sand	silty, clayey sand	SC - SM
Yeshi Hotel	fine sand	silty sand	SM
ST.Gorge church	medium sand	silty, clayey sand	SC - SM
SNNPRS Office	medium sand	well graded sand with silt	SW- SM
Gored Gored Hotel	fine sand	sandy silt	ML
Bureau of Education	fine sand	silty sand	SM
Old AirPort	medium sand	silty sand	SM
Haile resort	fine sand	sandy silt	ML
South Spring	fine sand	sandy silt	ML

4.3 SCATTER PLOT AND BEST-FIT CURVE

The MS excel version 2013 spread sheet is used for scatter plot and best fit curve, which is found to be manageable tool for scatter plot analysis and determination of correlation between small variables. In this work, shear strength parameters: angle of internal friction, φ and cohesion, c are the dependent variables whereas the in-situ density, D_{60}, D_{50}, D_{25} D_{15}, D_{10}, percentage fine sized sand particles, percentage silt sized particles, fine percentage passing No. 200 sieve, coefficient of curvature, Cc and clays sized particles are independent categories.

4.3.1 Natural Moisture Content vs Angle of Internal Friction

The relationship between the natural moisture content and the angle of internal friction for all of the tested samples is shown below. The best fitting trend line for this relationship is φ = - 0.1587x + 33.704. The strength of this equation in predicting an outcome from the natural moisture content is around 43.49 % or $R^2 = 0.43$

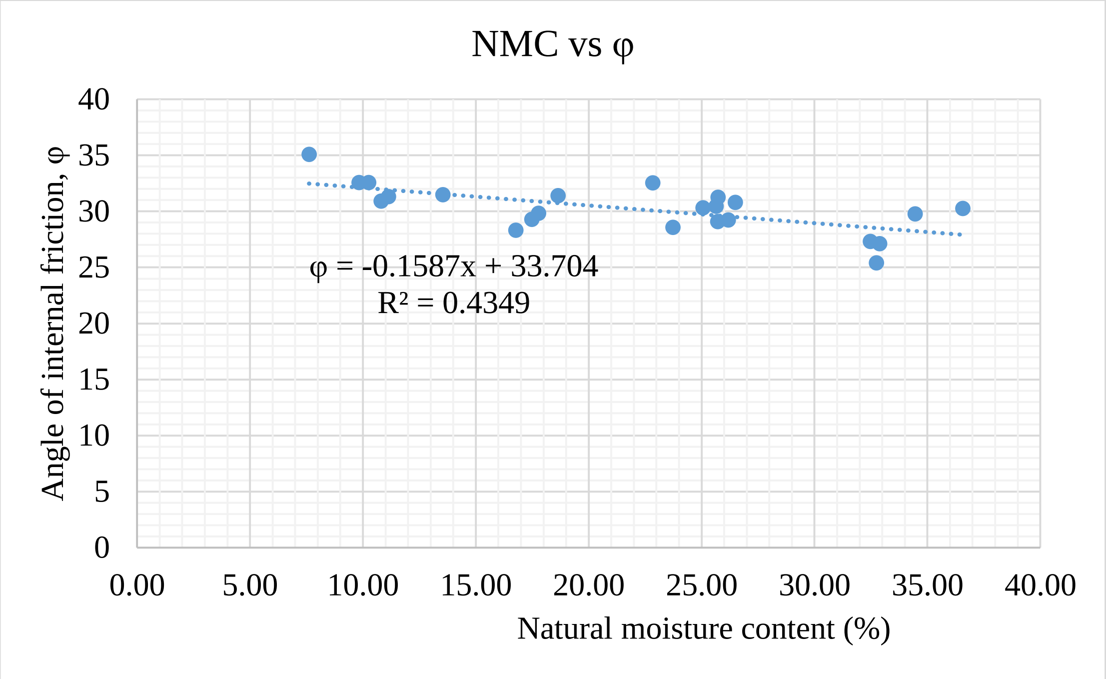

Figure 4.5 NMC vs φ

The angle of internal friction and the natural moisture content of the study area shows that the scattered points from the trend line indicates that the relationship is not very strong. This shows that the determination of the natural moisture content alone cannot satisfactory to predict angle of internal friction the soil of the study area.

4.3.2 D_{50} vs Angle of Internal Friction

The relationship between the D_{50} and the angle of internal friction for the tested samples is shown below. The best fitting trend line for this relationship is φ =-3.8092x + 31.134. The strength of this equation in predicting an outcome from the D_{50} is around 10 % or R^2 = 0.0985

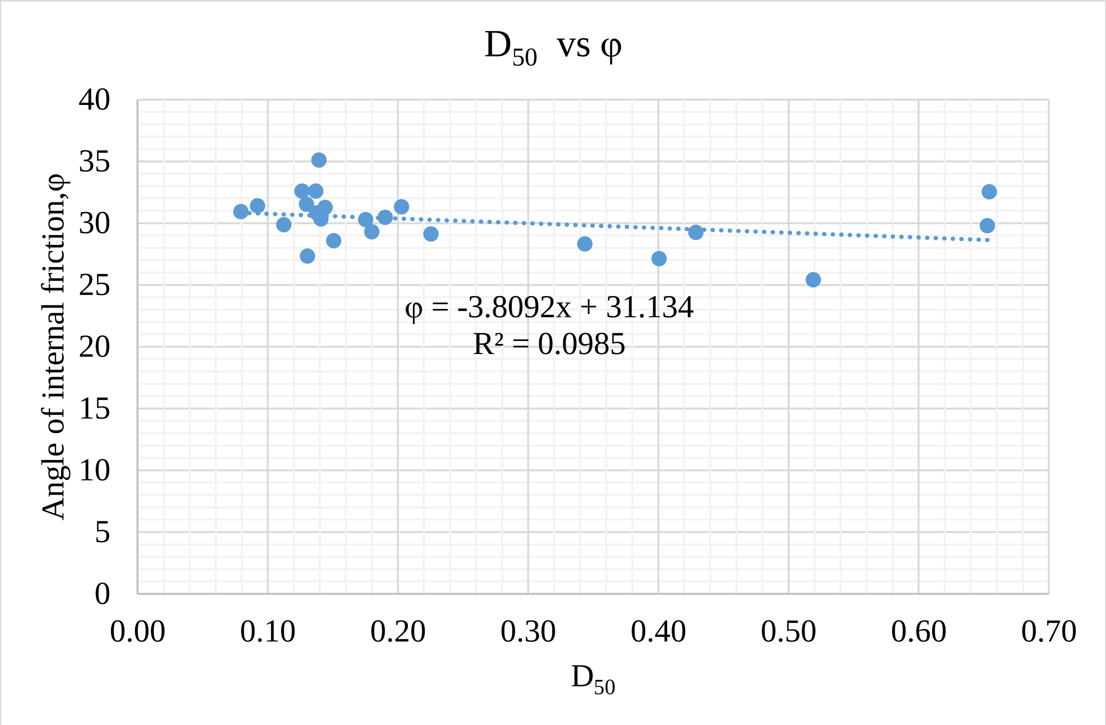

Figure 4.6 D_{50} vs φ

The angle of internal friction and D_{50} of the study area shows that the scattered points from the trend line indicates that the relationship is very weak. This shows that the determination of the D_{50} alone cannot be satisfactory to predict angle of internal friction the soil under study.

4.3.3 D_{30} vs Angle of Internal Friction

The relationship among D_{30} and the angle of internal friction for the tested samples is shown below. The best fitting trend line for this relationship is φ = -16.894x + 31.845. The strength of this equation in predicting an outcome from the D_{30} is around 16.48 % or R^2 = 0.1648.

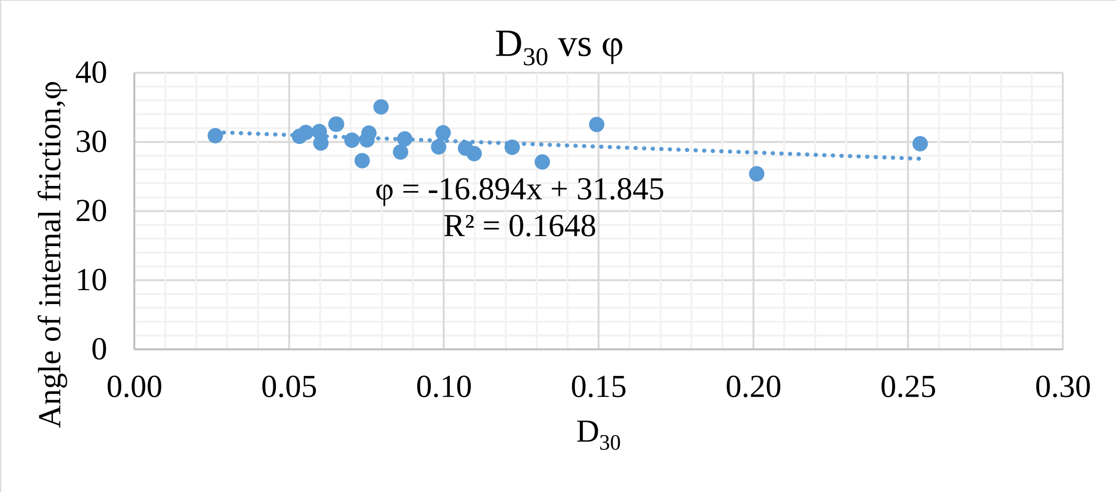

Figure 4.7 D_{30} vs φ

The angle of internal friction and D_{30} of the study area shows that the scattered points from the trend line indicates that the relationship is weak. This shows that the determination of the D_{30} alone cannot be satisfactory to predict angle of internal friction the soil under study.

4.3.4 Percent passing #200 sieve vs Angle of Internal Friction

The relationship between percent passing #200 sieve and the angle of internal friction for the tested samples is shown below. The best fitting trend line for this relationship is $\varphi = 0.0854x + 27.791$. The strength of this equation in predicting an outcome from the percent passing #200 sieve is around 17.77 % or $R^2 = 0.1777$.

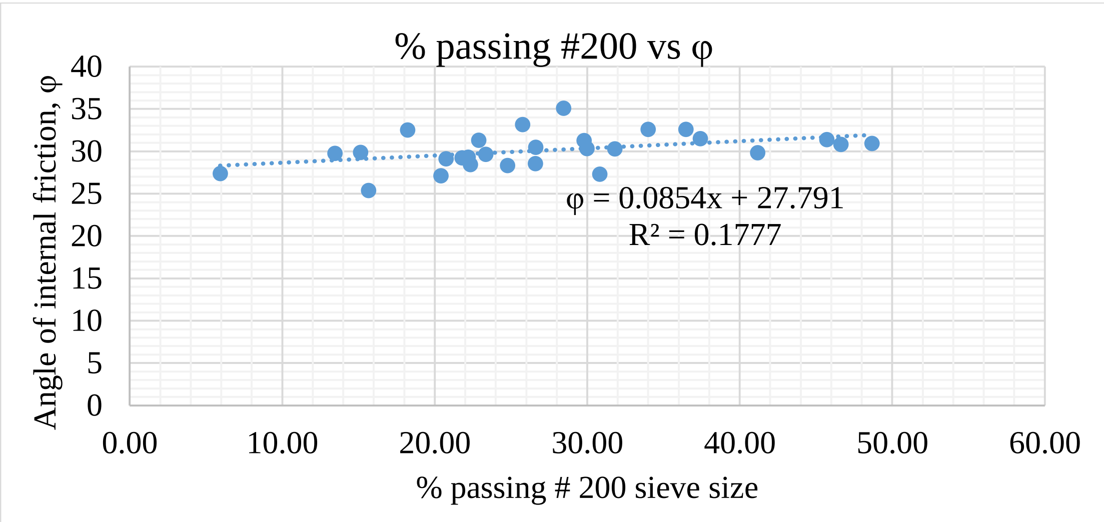

Figure 4.8 % passing #200 vs φ

The angle of internal friction and % passing #200 of the study area shows that the scattered points from the trend line indicates that the relationship is weak. This shows that the determination of the % passing #200 alone cannot be satisfactory to predict angle of internal friction the soil under study.

4.4 REGRESSION ANALYSIS

In this study, it was to develop reliable correlations between shear strength parameters angle of internal friction, φ and cohesion, c with soil index properties. This was done by first collecting data performing direct shear test, and all other index property tests according to ASTM test standards.

Therefore, shear strength parameters: angle of internal friction, φ and cohesion, c are the dependent variables whereas the in-situ density, D_{60}, D_{50}, D_{25} D_{15}, D_{10}, percentage fine sized sand particles, percentage silt sized particles, fine percentage passing No. 200 sieve, coefficient of curvature, Cc and clays sized particles are independent categories. All the thirty four samples classified using grain size analysis and further classified by using Unified Soil Classification System with their group name silty sand, which are twenty three samples at one group, silty sand, silty, clayey sand and silty, clayey sand with gravel all together twenty eight samples at second group, others, whose angle of internal friction value very far from the mean discarded from correlation. Each one Sample with angle of internal friction around mean value taken for control parameter for each group.

Generally, inspecting the above scatter diagram, the points lie scattered randomly around a straight line. The relationship between the variables then can be approximated by the straight-line relationship, in the form of:

$y(x) = b_0 + b_1x_1 + b_2x_2$ ………………………………………………………………...4.1

Nearly in all of the locations, the soil was cohesionless in nature. Only in few locations, cohesive soil were found. Due to fewer values of cohesion data, c collected no correlation was made with index properties.

Table 4.7 using silty sand

No	Equation	Adjusted R square
1	φ = 21.941 – 0.232*NMC+0.335* % passing #200 sieve – 0.612 % clay	76.6 %
2	φ = 22.029 – 0.232 *NMC +171.464 *D_{25}+ 0.332 *% passing #200 sieve – 0.614 % clay	78.90 %
3	φ = 21.931 – 0.238 *NMC +168.926 *D_{25}+ 0.33 *% passing #200 sieve – 0.629 % clay	80.8 %
4	φ = 20.391 – 0.229 *NMC +16.011* D_{60} +185.863 *D_{25}+ 0.351 *% passing #200 sieve – 0.592 % clay	81.10 %
5	φ = 24.897 – 0.229 *NMC – 83.066* D_{30} +198.919 *D_{25} + 0.30* passing #200 sieve – 0.584 % clay	81.0 %
6	φ = 24.588 – 0.236 *NMC + 17.653*D_{60} – 26.471*D_{50} – 72.331* D_{30} +163.303 *D_{25} + 0.271 % passing #200 sieve – 0.540 % clay	80.80 %
7	φ = 24.518 – 0.234 *NMC +17.261*D_{60} – 25.840*D_{50} – 75.394* D_{30} +162.113 *D_{25} + 0.268 % passing #200 sieve – 0.513 % clay	81.80 %

Table 4.8 Measured vs predicted value of angle of internal friction for silty sand

Test pits	measured vale of φ	Predicted value of φ						
		Eqn. 1	Eqn. 2	Eqn. 3	Eqn. 4	Eqn. 5	Eqn. 6	Eqn. 7
Test #01	29	21	36	35	42	32	30	29
Test #03	30	21	31	30	39	29	30	30
Test #04	33	20	40	40	59	34	33	32
Test #05	31	25	39	38	44	35	32	32
Test #06	30	28	37	36	40	35	32	32
Test #07	28	22	35	35	44	30	28	28
Test #08	29	22	37	36	42	33	31	30
Test #09	25	16	41	40	54	30	27	26
Test #10	29	20	36	35	48	31	30	29
Test #11	29	24	36	36	40	33	30	30
Test #12	27	23	34	33	37	31	28	28
Test #13	30	16	48	47	66	35	31	30
Test #17	35	27	38	38	42	36	33	33
Test #18	31	29	32	32	34	32	30	30
Test #19	31	28	33	33	37	31	30	30
Test #20	30	23	35	35	41	32	31	30
Test #21	30	24	36	35	40	33	31	31
Test #22	31	23	35	34	39	32	30	30
Test #23	33	28	39	38	42	37	35	34
Test #25	32	26	35	34	38	33	31	31
Test #27	31	30	36	36	38	34	32	32
Test #31	33	28	38	37	41	36	33	33
Test #32	27	18	36	35	46	30	27	27

Table 4.9 Calculated value of φ, by using newly developed equation for control sample

Test pit	measured value of φ	Predicted Value of φ by using developed equations						
		Eqn. 1	Eqn. 2	Eqn. 3	Eqn. 4	Eqn. 5	Eqn. 6	Eqn. 7
#16	31	33	37	37	39	36	33	32

Table 4. 10 Checking accuracy of the newly developed equations for control sample

Test pit	measured value of φ	Difference from measured value (%)						
		Eqn. 1	Eqn. 2	Eqn. 3	Eqn. 4	Eqn. 5	Eqn. 6	Eqn. 7
#16	31	-6.30	-18.47	-17.39	-23.85	-15.47	-4.92	-4.15

Tbale 4.11 using silty sand, silty clayey sand and silty, clayey sand with gravel

No	Equation	Adjusted R square
1	φ = 24.318 – 0.178*NMC	49 %
2	φ = 24.914 – 0.177*NMC+ 14.267 *D_{60}	52 %
3	φ = 25.392 – 0.178*NMC+14.914* D_{60} +0.241*% passing#200sieve	55 %
4	φ = 25.45 – 0.178*NMC+15.213*D_{60} +0.239*% passing #200 sieve	58 %
5	φ = 26.591– 0.169*NMC+15.327*D_{60} +0.229*% passing #200 sieve	60 %
6	φ = 32.019 –0.179*NMC+13.927*D_{60} +0.164*% passing #200 sieve	60 %
7	φ = 32.291– 0.181*NMC+11.068*D_{60} +0.166*% passing #200 sieve	58 %
8	φ = 31.984 – 0.181*NMC+ 12.391*D_{60} – 15.211*D_{50}	60 %
9	φ = 32.592 – 0.178*NMC+ 12.241*D_{60} – 15.872*D_{50} -0.39* %clay	59 %

Table 4.12 Measured vs predicted value of angle of internal friction for silty sand, silty clayey sand and silty, clayey sand with gravel

Test pits	Measured Value of φ	Predicted value of φ								
		Eqn.1	Eqn.2	Eqn.3	Eqn.4	Eqn.5	Eqn.6	Eqn.7	Eqn.8	Eqn.9
Test #01	29	20	26	32	32	33	36	35	29	28
Test #03	30	18	26	34	34	35	38	37	29	28
Test #04	33	20	37	43	43	44	47	44	32	30
Test #05	31	22	27	33	33	35	38	37	31	30
Test #06	30	21	24	35	35	36	38	38	29	27
Test #07	28	21	29	36	36	38	40	39	30	28
Test #08	29	21	26	32	32	33	37	36	30	28
Test #09	25	18	30	35	35	37	40	38	28	26
Test #10	29	20	31	37	37	39	41	40	30	28
Test #11	29	20	24	31	31	32	35	35	28	28
Test #12	27	19	22	30	30	31	34	33	26	25
Test #13	30	18	33	38	38	39	42	40	28	27
Test #14	30	20	34	38	39	40	43	41	30	28
Test #15	33	21	37	44	44	46	48	45	34	31
Test #17	35	23	27	34	34	35	38	38	31	30
Test #18	31	22	25	37	37	38	40	40	30	27
Test #19	31	20	24	36	36	37	38	38	28	27
Test #20	30	20	26	33	33	34	37	36	29	28
Test #21	30	20	24	32	32	33	36	35	28	28
Test #22	31	20	24	32	32	33	36	35	28	27
Test #23	33	22	26	35	35	36	39	38	31	29
Test #24	30	17	24	30	30	32	35	34	26	24
Test #25	32	22	26	35	35	36	39	38	30	28
Test #26	27	18	40	43	43	45	47	44	28	27
Test #27	31	21	23	35	35	36	38	38	29	27
Test #28	28	18	29	36	36	37	40	38	29	26
Test #31	33	23	26	35	35	36	39	39	31	29
Test #32	27	18	28	34	34	35	38	37	28	26

Table 4.13 Calculated value of φ, by using newly developed equation for control sample

Test pit	measured value of φ	Predicted Value of φ by using developed equations								
		Eqn. 1	Eqn.2	Eqn.3	Eqn.4	Eqn.5	Eqn.6	Eqn.7	Eqn.8	Eqn.9
# 29	31	22	32	35	36	37	41	39	31	31

Table 4. 14 Checking accuracy of the newly developed equations for control sample

Test pit	measured value of φ	Difference from measured value (%)								
		Eqn.1	Eqn.2	Eqn.3	Eqn.4	Eqn.5	Eqn.6	Eqn.7	Eqn.8	Eqn.9
#29	31	29.78	-1.36	-12.86	-13.58	-17.45	-29.11	-24.12	1.82	2.74

4.3.1 Graphical Representation of Measured and Calculated Values

To inspect the prediction accuracy of newly developed empirical equations, the results of the best equations are compared with the actual measured value.

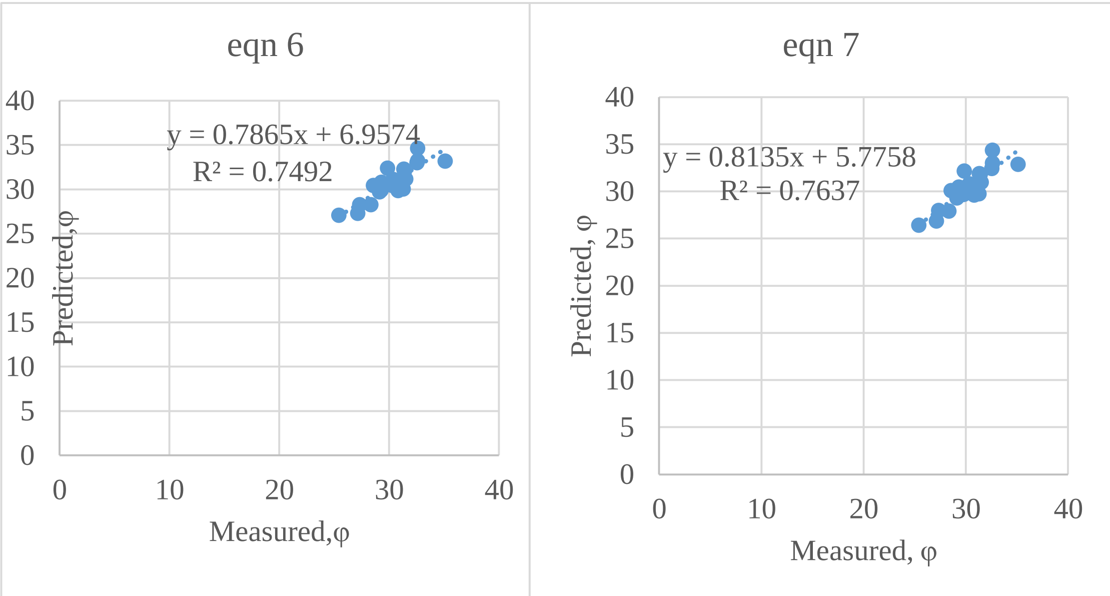

Figure 4.9 Measured vale of φ compared with predicted value of φ

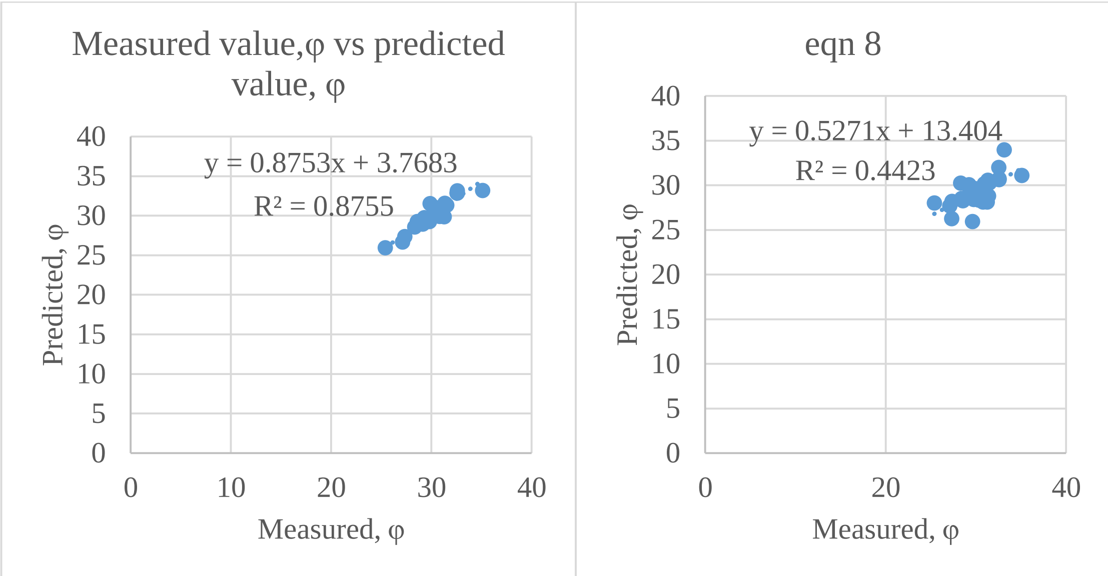

Figure 4.10 Measured vale of φ compared with predicted value of φ

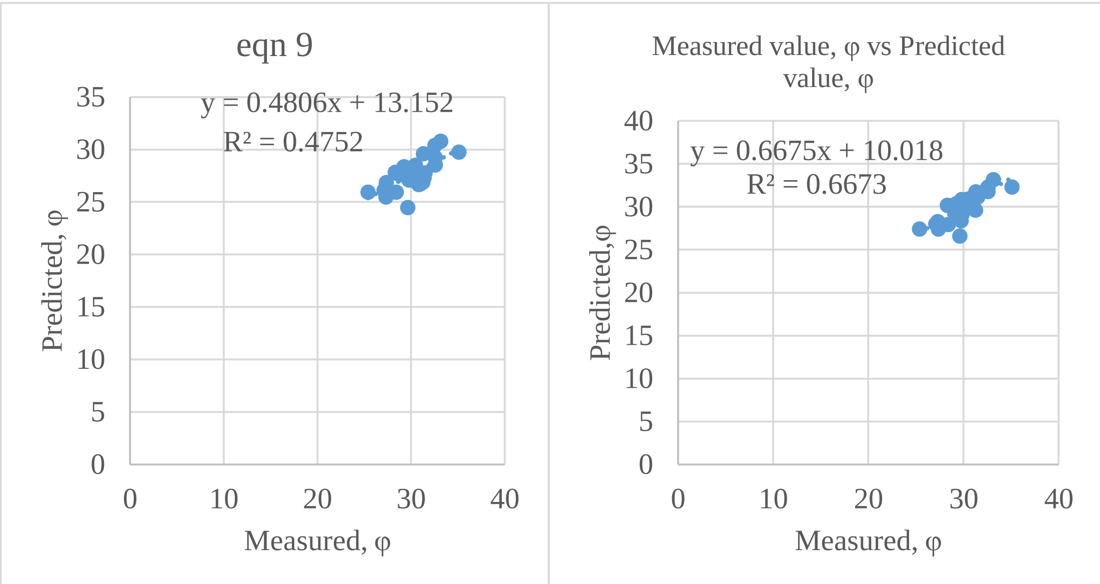

Figure 4.11 Measured vale of φ compared with predicted value of φ

5 SUMMARY AND CONCLUSION

5.1 SUMMARY

According to USCS the soil samples from East side of Hawassa city; near Etab Soup Factory, new Stadium, Saint Merry Church, IOT building of HU, Shell Depot, Debub Academy, Yeshi Hotel, Textile Factory, Diaspora ‘Chefe’ area, and around SNNPR State office are classified and grouped as medium to fine sand, and fine to medium sand, with group name silty sand, and well graded sand with silt, with their corresponding group symbol SM and SW-SM. Samples from South direction of the city are identified and classified as medium to fine sand and fine to medium sand, with group name sandy silt, ML silty sand, SM silty, clayey sand, SC-SM. Samples taken from North direction of the city are classified as medium to fine sand, with group name silty sand and sandy silt with group symbol SM and ML. Samples from North East direction of the city classified as medium and fine sand with group name silty, clayey sand, SC - SM, and sity sand, SM respectively. Samples collected from Central part of the city near Infolink College, and Finance Bureau, are medium sand and classified as silty sand, SM. Samples from West side of the city near ‘Gored Gored’ Hotel and samples from South East of the city are identified fine sand classified as silty sand, SM, and silty, clayey sand, SC- SM.

The identification of the most influencing index properties made by considering the following points in to account:

The value of adjusted R^2 for the regression analysis with relatively higher value as the first point, the slope of the line for the measured verses calculated angle of internal friction graph with relatively higher value as the second and finally, the newly developed equation should give almost the same angle of internal friction value compared with the measured one for the control samples.

5.2 CONCLUSIONS

From field and laboratory investigation of the soil, correlation and statistical regression analyses made, the following conclusions are drawn:

From the model developed, it is possible to predict shear strength parameters from index properties of soil. Again it is seen that natural moisture content, particle diameter corresponding to 60 %, 50 % 25 % finer on the cumulative particle size distribution curve, percentage fine passing No.200 sieve and percentage clay sized particles become the common and the most influencing independent variables to predict angle of internal friction for the specific soil investigated.

All the soil index properties studied and the soil classified by using Unified Soil Classification System (USCS) as cohesionless, fine to medium sized sand particles with group name and group symbols silty sand, SM, which is the most dominant, sandy silt, ML and few samples are silty, clayey sand, SC - ML. Almost the soil studied can be generalized as fine-grained soil.

Shear strength parameters - angle of internal friction and cohesion - were determined by direct shear test and ranged from about 23^o to 35^o. But almost all the results recorded were from about 27^o to 31^o for angle of friction with average of 29^o and nearly no cohesion value recorded.

This correlation can be used for preliminary design works and random checks of designs. With the help of which cost and time required for the test can be saved.

5.3 RECOMMENDATION

This research work is based on the direct shear test results, alternatively, the equation developed may be further improved by increasing the number of samples and using triaxial compression test equipment by considering the effect of pore water pressure.

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APPENDICES

Appendix – A Some of Field Density and Natural Moisture Content

Project	M.sc Thesis work	
Location	Hawassa, Tabor sub city, Near New Bus Station	
Test pit number	05	
Depth	3.0 m	Test method
Type of test	Natural Moisture content	ASTM D 2216
Type of test	Field density	ASTM D 1556

Table A.1 Field density for Test #5

Apparatus Calibration	unit	
Weight of sand in cone pouring cylinder = (M_5)	g	1450
Internal volume of calibration container(V_1)	cm^3	944
Bulk density of the sand, (ρ_1) =($\frac{M_5}{V_1}$)	$\frac{g}{cm^3}$	1369
moist mass of the material from test hole, (M_3)	g	2,092
mass of the sand used to fill the test hole, funnel and base plate, (M_1)	g	8,550
mass of the sand used to fill the funnel and base plate,(M_2)	g	4,800
Mass of sand in hole, (xx = M1 – M2 – M_5)	g	2,300
wet density of the tested material, ($\rho_m = \frac{M_3}{V}$)	$\frac{g}{cm^3}$	1.25

Table A.2 Natural moisture content (NMC) for test #05

Can code	Weight of can	Weight of can + moist soil	Weight of can + oven dry soil	Weight of water	Weight of dry soil	Moisture content (%)
	(g), 1	(g), 2	(g),3	(g),4 = (2-3)	(g), 5 = (3-1)	(4/5)*100
1	21.50	121.89	112.26	9.63	90.76	10.61
2	20.34	106.50	97.76	8.74	77.42	11.29
3	20.97	126.52	116.05	10.47	95.08	11.01
4	20.32	121.22	110.13	11.09	89.81	12.35
5	18.84	114.29	105.58	8.71	86.74	10.04
6	21.25	122.40	111.95	10.45	90.70	11.52
Average Moisture Content						11.14

Project	M.sc Thesis work	
Location	Hawassa, Mehal sub city, Near Bureau of Finance	
Test pit number	10	
Depth	3.0 m	Test method
Type of test	Natural Moisture content	ASTM D 2216
Type of test	Field density	ASTM D 1556

Table A.3 Field density for Test #10

Apparatus Calibration	unit	
Weight of sand in cone pouring cylinder = (M_5)	g	1450
Internal volume of calibration container(V_1)	cm^3	944
Bulk density of the sand, (ρ_1) =($\frac{M_5}{V_1}$)	$\frac{g}{cm^3}$	1369
moist mass of the material from test hole, (M_3)	g	1,960
mass of the sand used to fill the test hole, funnel and base plate, (M_1)	g	8,355
mass of the sand used to fill the funnel and base plate,(M_2)	g	4,569
Mass of sand in hole, (xx = M1 – M2 – M_5)	g	2,336
wet density of the tested material, ($\rho_m = \frac{M_3}{V}$)	$\frac{g}{cm^3}$	1.15

Table A.4 Natural moisture content (NMC) for Test #10

Can code	Weight of can	Weight of can + moist soil	Weight of can + oven dry soil	Weight of water	Weight of dry soil	Moisture content (%)
	(g), 1	(g), 2	(g),3	(g),4 = (2-3)	(g), 5 = (3-1)	(4/5)*100
1	77.76	130.93	120.09	10.84	42.33	25.61
2	78.76	144.50	130.75	13.75	51.99	26.45
3	78.99	131.50	120.12	11.38	41.13	27.67
4	78.60	133.92	122.55	11.37	43.95	25.87
5	78.28	123.42	114.19	9.23	35.91	25.70
6	79.23	122.28	113.46	8.82	34.23	25.77
Average Moisture Content						26.18

Project	M.sc Thesis work	
Location	Hawassa, Tabor sub city, In front of Hawassa Alamra KHC	
Test pit number	15	
Depth	3.0 m	Test method
Type of test	Natural Moisture content	ASTM D 2216
Type of test	Field density	ASTM D 1556

Table A.5 Field density for Test #15

Apparatus Calibration	unit	
Weight of sand in cone pouring cylinder = (M_5)	g	1450
Internal volume of calibration container(V_1)	cm^3	944
Bulk density of the sand, (ρ_1) =($\frac{M_5}{V_1}$)	$\frac{g}{cm^3}$	1369
moist mass of the material from test hole, (M_3)	g	3,080
mass of the sand used to fill the test hole, funnel and base plate, (M_1)	g	7,803
mass of the sand used to fill the funnel and base plate,(M_2)	g	2,880
Mass of sand in hole, (xx = M1 – M2 – M_5)	g	3,474
wet density of the tested material, ($\rho_m = \frac{M_3}{V}$)	$\frac{g}{cm^3}$	1.21

Table A.6 Natural moisture content (NMC) for Test #15

Can code	Weight of can	Weight of can + moist soil	Weight of can + oven dry soil	Weight of water	Weight of dry soil	Moisture content (%)
	(g), 1	(g), 2	(g),3	(g),4 = (2-3)	(g), 5 = (3-1)	(4/5)*100
1	22.38	120.31	104.76	15.55	82.38	18.88
2	19.93	119.81	103.30	16.51	83.37	19.80
3	20.31	135.52	116.75	18.77	96.44	19.46
4	19.94	130.84	112.18	18.66	92.24	20.23
5	18.80	120.48	102.93	17.55	84.13	20.86
6	20.27	119.33	101.96	17.37	81.69	21.26
Average Moisture Content						20.08

Project	M.sc Thesis work	
Location	Hawassa, Menaharia sub city, Diaspora – Mariam Church'	
Test pit number	20	
Depth	3.0 m	Test method
Type of test	Natural Moisture content	ASTM D 2216
Type of test	Field density	ASTM D 1556

Table A.7 Field density for Test #20

Apparatus Calibration	unit	
Weight of sand in cone pouring cylinder = (M_5)	g	1450
Internal volume of calibration container(V_1)	cm^3	944
Bulk density of the sand, (ρ_1) =($\frac{M_5}{V_1}$)	$\frac{g}{cm^3}$	1369
moist mass of the material from test hole, (M_3)	g	1,911
mass of the sand used to fill the test hole, funnel and base plate, (M_1)	g	7,451
mass of the sand used to fill the funnel and base plate,(M_2)	g	3,519
Mass of sand in hole, (xx = M1 – M2 – M_5)	g	2,482
wet density of the tested material, ($\rho_m = \frac{M_3}{V}$)	$\frac{g}{cm^3}$	1.05

Table A.8 Natural moisture content (NMC) for Test #20

Can code	Weight of can	Weight of can + moist soil	Weight of can + oven dry soil	Weight of water	Weight of dry soil	Moisture content (%)
	(g), 1	(g), 2	(g),3	(g),4 = (2-3)	(g), 5 = (3-1)	(4/5)*100
1	21.75	94.58	79.84	14.74	58.09	25.37
2	20.71	88.36	74.28	14.08	53.57	26.28
3	20.30	87.40	73.57	13.83	53.27	25.96
4	23.14	93.67	79.00	14.67	55.86	26.26
5	21.49	81.45	68.99	12.46	47.50	26.23
6	18.79	77.21	65.99	11.22	47.20	23.77
Average Moisture Content						25.65

Project	M.sc Thesis work	
Location	Hawassa, Tabor sub city, Near ‘Buna Board’	
Test pit number	25	
Depth	3.0 m	Test method
Type of test	Natural Moisture content	ASTM D 2216
Type of test	Field density	ASTM D 1556

Table A.9 Field density for Test #25

Apparatus Calibration	unit	
Weight of sand in cone pouring cylinder = (M_5)	g	1450
Internal volume of calibration container(V_1)	cm^3	944
Bulk density of the sand, (ρ_1) =($\frac{M_5}{V_1}$)	$\frac{g}{cm^3}$	1369
moist mass of the material from test hole, (M_3)	g	2,032
mass of the sand used to fill the test hole, funnel and base plate, (M_1)	g	8,810
mass of the sand used to fill the funnel and base plate,(M_2)	g	4,680
Mass of sand in hole, (xx = M1 – M2 – M_5)	g	2,680
wet density of the tested material, ($\rho_m = \frac{M_3}{V}$)	$\frac{g}{cm^3}$	1.04

Table A.10 Natural moisture content (NMC) for Test #25

Can code	Weight of can	Weight of can + moist soil	Weight of can + oven dry soil	Weight of water	Weight of dry soil	Moisture content (%)
	(g), 1	(g), 2	(g),3	(g),4 = (2-3)	(g), 5 = (3-1)	(4/5)*100
1	21.75	107.93	97.57	10.36	75.82	13.66
2	21.43	128.28	115.70	12.58	94.27	13.34
3	23.10	108.92	98.68	10.24	75.58	13.55
4	19.94	108.02	97.44	10.58	77.50	13.65
5	22.39	111.43	100.65	10.78	78.26	13.77
6	21.07	125.79	113.51	12.28	92.44	13.28
Average Moisture Content						13.54

Appendix – B Specific Gravity both oven dry and moist

Project	M.sc Thesis work
Location	Hawassa, Tabor sub city, Near New Bus Station
Test pit number	05
Depth	3.0 m
Type of test	Specific Gravity
Test method	ASTM D 854 - 98

Table B.1 Specific Gravity oven dried for Test #5

Method A Oven Dry Specimen	A	B	C
Pycnometer ID	1	2	3
Measured temperature of used water, T_x in (T ^{o} C)	22	22	22
Weight of Pycnometer, M_f(g)	160.3	160.53	160.49
Weight of sample, M_o)g)	106.26	103.44	103.91
Weight of pycnometer with full of water, M_a(g)	658.02	629.1	652.58
Weight of pycnometer + sample + water, M_b(g)	719.93	689.42	713.22
Conversion fator, K	0.9996	0.9996	0.9996
Specific gravity(Gs) at 20^{o}C = $KM_o/(M_o+(M_a-M_b))$	2.395	2.398	2.400
Average Specific gravity(A+B+C)/3	2.4		

Table B.2 Specific Gravity moist for Test #5

Method B Moist Specimen	A	B	C
Pycnometer ID	2	3	5
Measured temperature of used water, T_x in (T ^{o} C)	26	26	26
Weight of Pycnometer, M_f(g)	130.9	153.62	138.34
Weight of sample, M_o)g)	152.46	152.05	154.7
Weight of pycnometer with full of water, M_a(g)	628.59	652.18	637.14
Weight of pycnometer + sample + water, M_b(g)	719.17	742.61	729.06
Conversion fator, K	0.99860	0.99860	0.99860
Specific gravity(Gs) at 20^{o}C = $KM_o/(M_o+(M_a-M_b))$	2.460	2.464	2.461
Average Specific gravity(A+B+C)/3	2.46		

Project	M.sc Thesis work
Location	Hawassa, Tabor sub city, In front of Hawassa Alamra KHC
Test pit number	15
Depth	3.0 m
Type of test	Specific Gravity
Test method	ASTM D 854 - 98

Table B.5 Specific Gravity oven dried for Test #15

Method A Oven Dry Specimen	A	B	C
Pycnometer ID	2	3	5
Measured temperature of used water, T_x in ($T\ ^oC$)	26	26	26
Weight of Pycnometer, M_f(g)	130.9	153.62	138.34
Weight of sample, M_o)g)	101.12	101.92	102.08
Weight of pycnometer with full of water, M_a(g)	628.59	652.18	637.14
Weight of pycnometer + sample + water, M_b(g)	686.00	710.06	695.12
Conversion fator, K	0.9986	0.9986	0.9986
Specific gravity(Gs) at $20^oC = KM_o/(M_o+(M_a-M_b))$	2.310	2.311	2.311
Average Specific gravity(A+B+C)/3	2.31		

Table B.6 Specific Gravity moist for Test #15

Method B Moist Specimen	A	B	C
Pycnometer ID	2	3	5
Measured temperature of used water, T_x in ($T\ ^oC$)	24.5	24.5	24.5
Weight of Pycnometer, M_f(g)	130.9	153.62	138.34
Weight of sample, M_o)g)	156.61	151.62	157.43
Weight of pycnometer with full of water, M_a(g)	628.78	652.33	637.4
Weight of pycnometer + sample + water, M_b(g)	720.26	741.1	729.48
Conversion fator, K	0.9990	0.999	0.999
Specific gravity(Gs) at $20^oC = KM_o/(M_o+(M_a-M_b))$	2.402	2.410	2.407
Average Specific gravity(A+B+C)/3	2.41		

Project	M.sc Thesis work
Location	Hawassa, Menaharia sub city, Diaspora – Mariam Church’
Test pit number	20
Depth	3.0 m
Type of test	Specific Gravity
Test method	ASTM D 854 - 98

Table B.7 Specific Gravity oven dried for Test #20

Method A Oven Dry Specimen	A	B	C
Pycnometer ID	2	3	6
Measured temperature of used water, T_x in (T ^{o} C)	23	23	23
Weight of Pycnometer, M_f(g)	130.9	153.62	180.89
Weight of sample, M_o)g)	104.88	106.06	108.22
Weight of pycnometer with full of water, M_a(g)	628.97	652.48	679.82
Weight of pycnometer + sample + water, M_b(g)	688.50	712.89	741.25
Conversion fator, K	0.9993	0.9993	0.9993
Specific gravity(Gs) at 20^{o}C = $KM_o/(M_o+(M_a-M_b))$	2.311	2.322	2.311
Average Specific gravity(A+B+C)/3	2.31		

Table B.8 Specific Gravity moist for Test #20

Method B Moist Specimen	A	B	C
Pycnometer ID	1	4	6
Measured temperature of used water, T_x in (T ^{o} C)	26	26	26
Weight of Pycnometer, M_f(g)	160.51	148.15	180.89
Weight of sample, M_o)g)	158.61	157.19	159.48
Weight of pycnometer with full of water, M_a(g)	657.74	645.86	679.49
Weight of pycnometer + sample + water, M_b(g)	751.17	738.65	773.36
Conversion fator, K	0.9986	0.9986	0.9986
Specific gravity(Gs) at 20^{o}C = $KM_o/(M_o+(M_a-M_b))$	2.430	2.437	2.427
Average Specific gravity(A+B+C)/3	2.43		

Project	M.sc Thesis work
Location	Hawassa, Tabor sub city, Near 'Buna Board'
Test pit number	25
Depth	3.0 m
Type of test	Specific Gravity
Test method	ASTM D 854 - 98

Table B.9 Specific Gravity oven dried for Test #25

Method A Oven Dry Specimen	A	B	C
Pycnometer ID	2	3	5
Measured temperature of used water, T_x in ($T\ ^oC$)	26	26	26
Weight of Pycnometer, M_f(g)	130.9	153.62	138.34
Weight of sample, M_o)g)	107.17	106.08	101.08
Weight of pycnometer with full of water, M_a(g)	628.59	652.18	637.14
Weight of pycnometer + sample + water, M_b(g)	690.28	713.34	695.20
Conversion fator, K	0.9986	0.9986	0.9986
Specific gravity(Gs) at 20^{o}C = $KM_o/(M_o+(M_a-M_b))$	2.353	2.358	2.346
Average Specific gravity(A+B+C)/3	2.35		

Table B.10 Specific Gravity moist for Test #25

Method B Moist Specimen	A	B	C
Pycnometer ID	1	4	6
Measured temperature of used water, T_x in ($T\ ^oC$)	26	26	26
Weight of Pycnometer, M_f(g)	160.51	148.15	180.89
Weight of sample, M_o)g)	164.15	162.68	162.05
Weight of pycnometer with full of water, M_a(g)	657.74	645.86	679.49
Weight of pycnometer + sample + water, M_b(g)	754.47	741.62	774.74
Conversion fator, K	0.9986	0.9986	0.9986
Specific gravity(Gs) at 20^{o}C = $KM_o/(M_o+(M_a-M_b))$	2.431	2.428	2.423
Average Specific gravity(A+B+C)/3	2.43		

Project	M.sc Thesis work
Location	Hawassa, Hayki dar sub city, Near 'Gordi Gored' Hotel
Test pit number	30
Depth	3.0 m
Type of test	Specific Gravity
Test method	ASTM D 854 - 98

Table B.11 Specific Gravity oven dried for Test #30

Method A Oven Dry Specimen	A	B	C
Pycnometer ID	1	4	6
Measured temperature of used water, T_x in (T ^{o} C)	26	26	26
Weight of Pycnometer, M_f(g)	160.51	153.62	180.89
Weight of sample, M_o)g)	100	100	100.4
Weight of pycnometer with full of water, M_a(g)	657.78	645.86	679.49
Weight of pycnometer + sample + water, M_b(g)	717.77	705.98	739.80
Conversion fator, K	0.9986	0.9986	0.9986
Specific gravity(Gs) at 20^{o}C = $KM_o/(M_o+(M_a-M_b))$	2.496	2.504	2.501
Average Specific gravity(A+B+C)/3	2.50		

Table B.12 Specific Gravity moist for Test #30

Method B Moist Specimen	A	B	C
Pycnometer ID	1	4	6
Measured temperature of used water, T_x in (T ^{o} C)	25	25	25
Weight of Pycnometer, M_f(g)	160.51	153.62	180.89
Weight of sample, M_o)g)	145.64	180.15	173.22
Weight of pycnometer with full of water, M_a(g)	657.81	646.03	679.6
Weight of pycnometer + sample + water, M_b(g)	743.51	752.28	781.79
Conversion fator, K	0.9988	0.9988	0.9988
Specific gravity(Gs) at 20^{o}C = $KM_o/(M_o+(M_a-M_b))$	2.427	2.435	2.436
Average Specific gravity(A+B+C)/3	2.43		

Appendix – C   Moisture – Density Relation Modified Proctor

Project	M.sc Thesis work
Location	Hawassa, Tabor sub city, Near New Bus Station
Test pit	#05
Depth	3.0 m

Table C -1 Moisture – Density Relationship test #5

Modified ASTM D - 1557 - 00			Weight of hammer, kg			4.5
No. of blows - 56			No. of layers			5
trials	unit	1	2	3	4	5
Weight of (mold + wet soil)	g	7368	7667	7900	8019	8004
Weight of mold	g	4439				
Weight of wet soil	g	2929	3228	3461	3580	3565
Volume of mold	cm^3	2124				
Wet density	cm^3	1.379	1.520	1.629	1.685	1.678
Container No		1	2	3	4	5
Weight of container +wet soil		251.74	286.26	328.34	484.13	381.97
Weight of container + dry soil		228.21	252.85	283.66	410.38	321.19
Weight of water		23.53	33.41	44.68	73.75	60.78
Weight of container		79.15	78.25	77.72	108.44	112.39
Weight of dry soil		149	175	206	302	209
Moisture content	%	15.79	19.14	21.70	24.43	29.11
Dry density	g/cm^3	1.191	1.276	1.339	1.355	1.300

Maximum Dry Density (MDD) = 1.36 g/cm^3	Optimum Moisture Content (OMC) = 24 %

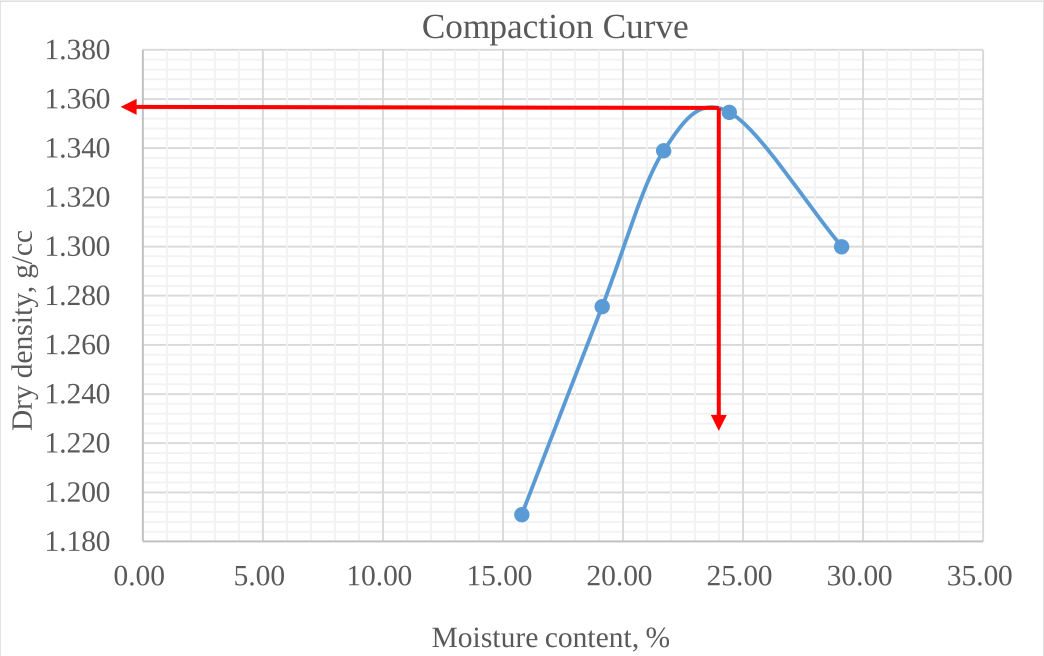


Figure C -1 OMC vs. MDD for #05

Project	M.sc Thesis work
Location	Hawassa, Tabor sub city, Near Poultry Farm
Test pit	#08
Depth	3.0 m

Table C -2 Moisture – Density Relationship test #8

Modified ASTM D - 1557 - 00			Weight of hammer, kg			4.5
No. of blows - 56			No. of layers			5
trials	unit	1	2	3	4	5
Weight of (mold + wet soil)	g	7955	8125	8250	8263	8242.0
Weight of mold	g	4439				
Weight of wet soil	g	3516	3686	3811	3824	3803
Volume of mold	cm^3	2124				
Wet density	cm^3	1.655	1.735	1.794	1.800	1.790
Container No		1	2	3	4	5
Weight of container +wet soil		42.47	269.71	229.08	304.94	386.23
Weight of container + dry soil		216.53	237.57	202.06	261.18	329.75
Weight of water		26	32	27	44	56
Weight of container		79.17	78.73	77.77	78.24	110.46
Weight of dry soil		137	159	124	183	219
Moisture content	%	18.88	20.23	21.74	23.92	25.76
Dry density	g/cm^3	1.392	1.443	1.474	1.453	1.424

Maximum Dry Density (MDD) = 1.48 g/cm^3	Optimum Moisture Content (OMC) = 22 %

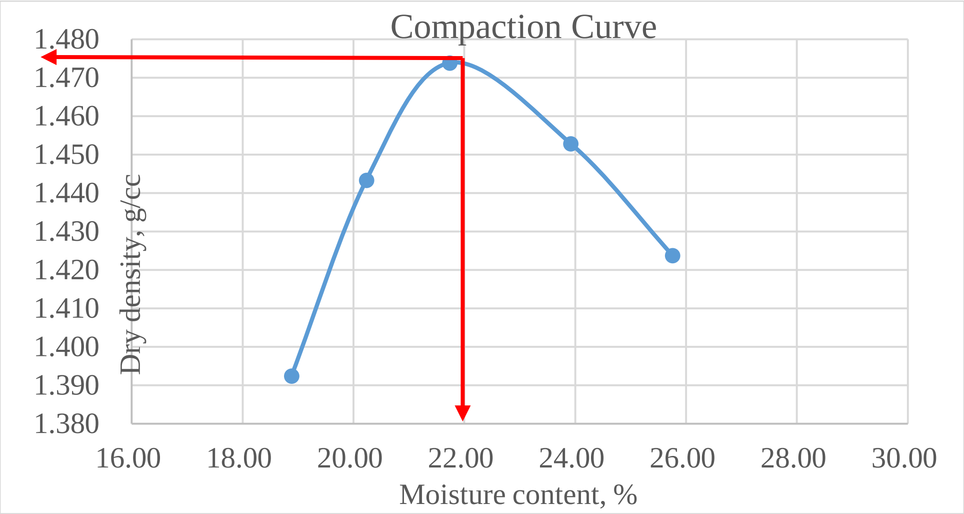


Figure C -2 OMC vs. MDD for Test #08

Appendix - D Some Direct Shear Test Results

Project	Msc. Thesis work
Location	Hawassa Tabor, sub city near 'Bale Wold' Church
Pit number	#02
Depth	3.0 m
Test method	ASTM D 3080
Sample size	60 mm X 60 mm X 20 mm
Area of the sample	60 mm X 60 mm
In-situ density of sample	1.21
Rate of horizontal displacement	0.01 mm per min.
Proving-ring reading calibration factor	1.3 N/div

Table D.1 Direct shear test result for test #2

Horizontal displacement Reading	100 kPa		200 kPpa		300 kPa	
	proving -ring reading,(Div)	Shear stress (kPa)	proving -ringreading,(Div)	Shear stress (kPa)	proving -ring reading,(Div)	Shear stress (kPa)
0	0	0.00	0	0.00	0	0.00
0.5	34	12.28	120	43.33	210	75.83
1	36	13.00	165	59.58	258	93.17
1.5	37	13.36	195	70.42	283	102.19
2	43	15.53	225	81.25	293	105.81
2.5	52	18.78	250	90.28	311	112.31
3	62	22.39	272	98.22	325	117.36
3.5	75	27.08	295	106.53	345	124.58
4	86	31.06	316	114.11	362	130.72
4.5	96	34.67	325	117.36	376	135.78
5	105	37.92	330	119.17	388	140.11
5.5	109	39.36	333	120.25	404	145.89
6	113	40.81	330	119.17	410	148.06
6.5	116	41.89			412	148.78
7	119	42.97			400	144.44
7.5	121	43.69				
8						
8.5						
9						
9.5						
10						

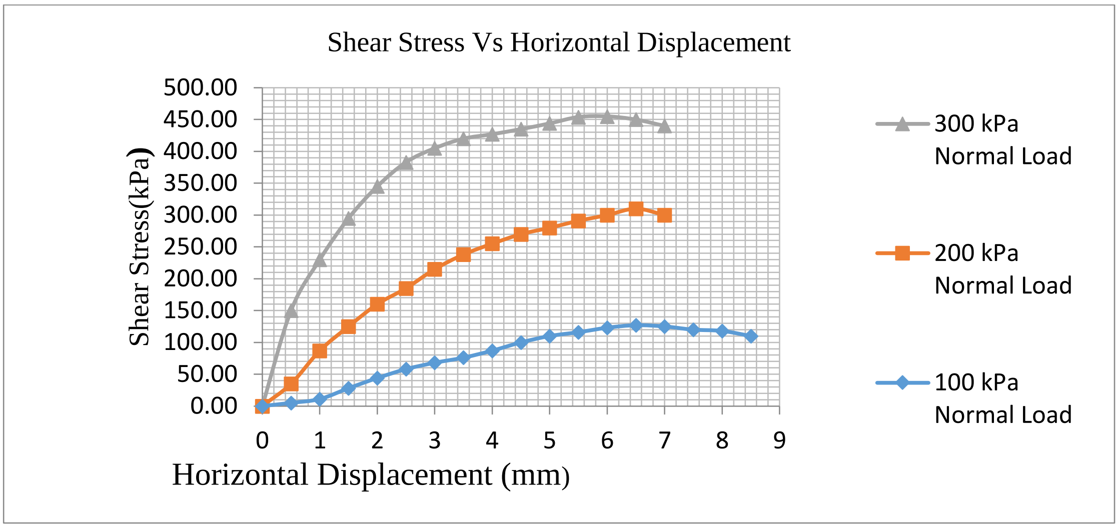


Figure D.1 Shear stress vs horizontal displacement

Table D.2 Normal stress and Shear stress

Normal stress(kPa)	Shear stress(kPa)			Failure criteria			unit
0	0	0		At 8.33 % strain	φ	26.64	o
100	37.92	43.69			c	0	kN/m^2
200	119.17	120.25		At peak	φ	27.61	o
300	140.11	148.78			c	0	kN/m^2

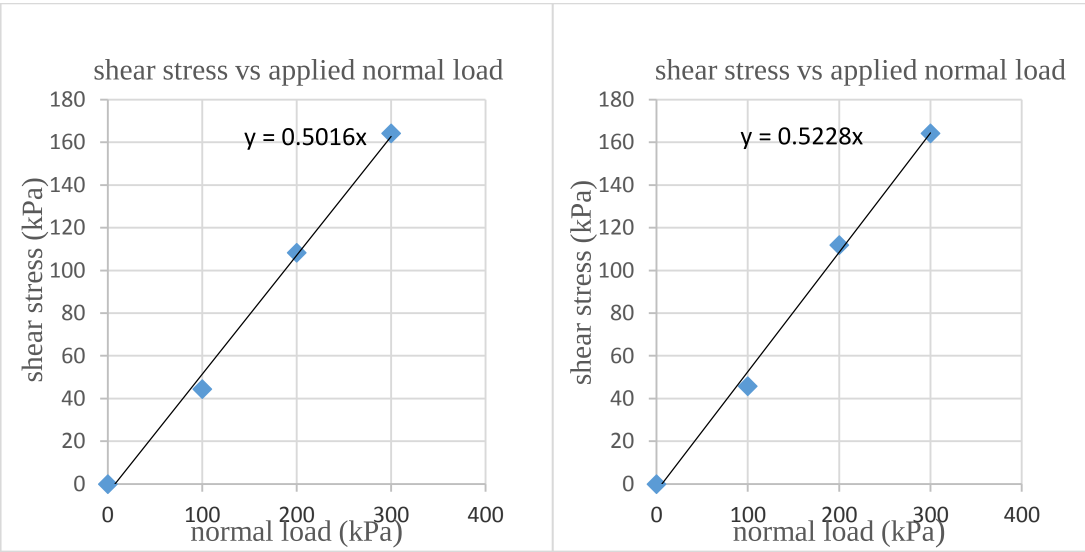


Figure D.2 Normal stress vs Shear *stress*

Project	Msc. Thesis work
Location	Hawassa , Piazza near 'UN office
Pit number	#03
Depth	3.0 m
Test method	ASTM D 3080
Sample size	60 mm X 60 mm X 20 mm
Area of the sample	60 mm X 60 mm
In-situ density of sample	1.30
Rate of horizontal displacement	0.01 mm per min.
Proving-ring reading calibration factor	1.3 N/div

Table D.3 Direct shear test result for test #3

Horizontal displacement Reading	100 kPa		200 kPpa		300 kPa	
	Proving- ring reading,(Div)	Shear stress (kPa)	Proving -ringreading (Div)	Shear stress (kPa)	Proving -ringreading (Div)	Shear stress (kPa)
0	0	0.00	0	0.00	0	0.00
0.5	5	1.81	45	16.25	110	39.72
1	30	10.83	110	39.72	155	55.97
1.5	50	18.06	160	57.78	225	81.25
2	65	23.47	195	70.42	270	97.50
2.5	80	28.89	230	83.06	325	117.36
3	95	34.31	255	92.08	355	128.19
3.5	100	36.11	283	102.19	390	140.83
4	123	44.42	305	110.14	430	155.28
4.5	136	49.11	320	115.56	460	166.11
5	147	53.08	332	119.89	480	173.33
5.5	155	55.97	332	119.89	480	173.33
6	160	57.78	345	124.58	480	173.33
6.5	163	58.86	355	128.19		
7	162	58.50	360	130.00		
7.5	160	57.78	358	129.28		
8	158	57.06	353	127.47		
8.5			350	126.39		
9						
9.5						
10						

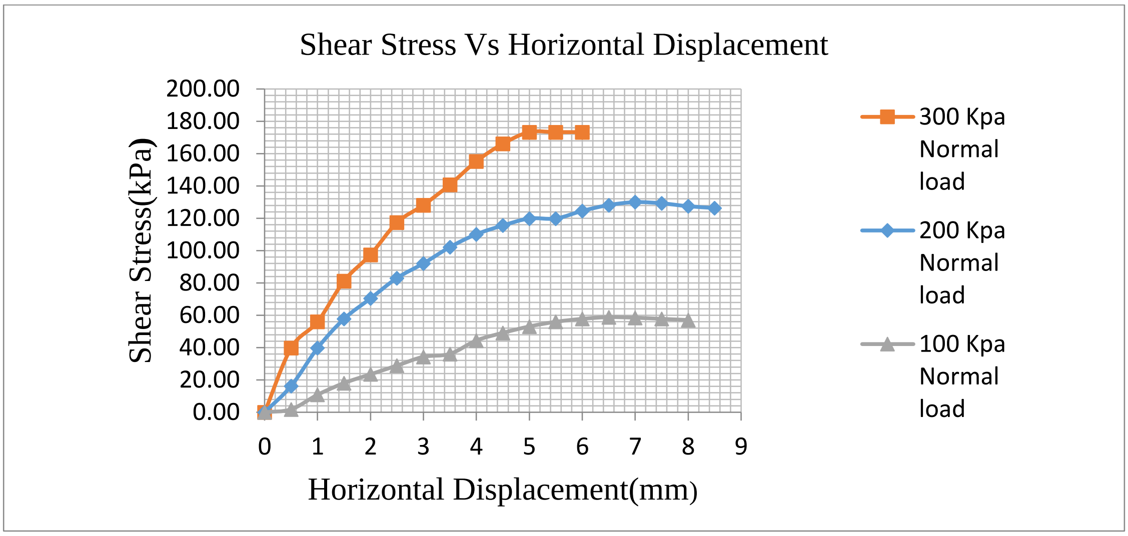


Figure D.3 Shear stress vs horizontal displacement

Table D.4 Normal stress and Shear stress

Normal stress(kPa)	Shear stress(kPa)			Failure criteria			unit
0	0	6.2593		At 10 % strain	φ	30.28	o
100	53.08	58.86			c	0.00	kN/m^2
200	119.89	130.00		At peak	φ	29.79	o
300	173.33	173.33			c	6.26	kN/m^2

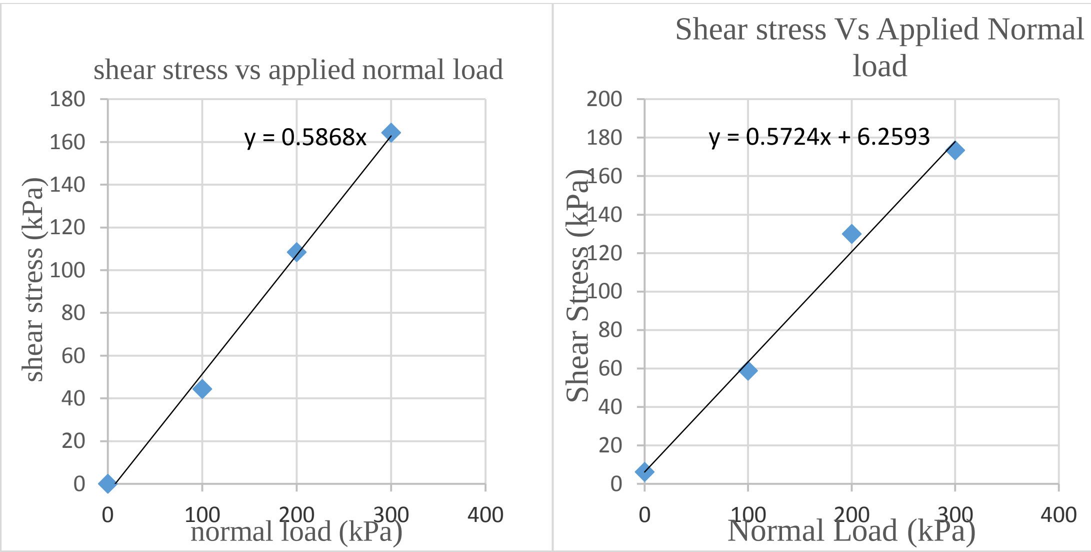


Figure D.4 Normal stress vs Shear stress

Appendix - E Some of Gradation Test Results Sieve Analysis with Hydrometer Combined

Sieve analysis

Table E.1 Typical sieve analysis

Sieve No	Sieve Opening (mm)	Weight of Sieve (g)	Wt of sieve + Retained soil (g)	Weight of Retained soil (g)	Percentage Retained (g)	Cumulative Percentage (g)	Percentage Passing (g)
1	2	3	4	5=[4]-[3]	6=[5] %		
3"	75	0	0	0	0.00	0.00	100.00
1 1/2"	37.5	0	0	0	0.00	0.00	100.00
3/4"	19	0	0	0	0.00	0.00	100.00
3/8"	9.5	544.3	544.9	0.6	0.12	0.12	99.88
No 4	4.75	573.72	585.77	12.05	2.42	2.54	97.46
No 8	2.36	548.57	566.77	18.2	3.66	6.20	93.80
No 16	1.18	506.89	524	17.11	3.44	9.64	90.36
No 30	0.6	476.4	505.8	29.4	5.91	15.55	84.45
No 50	0.3	426.11	469.05	42.94	8.63	24.18	75.82
No 100	0.15	416.51	472.16	55.65	11.18	35.36	64.64
No 200	0.075	397.72	491.91	94.19	18.93	54.29	45.71
pan	-		227.42	227.42			
Total weight retained in sieves(column = 5)				497.56			
Weight sieved through #200 (weight in pan)				227.42			
Error (original weight - weight retained)				2.44			
Error in (%)				0.488			
Percent gravel (%) 2.54			Percent sand (%) 51.75			Percent fines (%) 45.71	

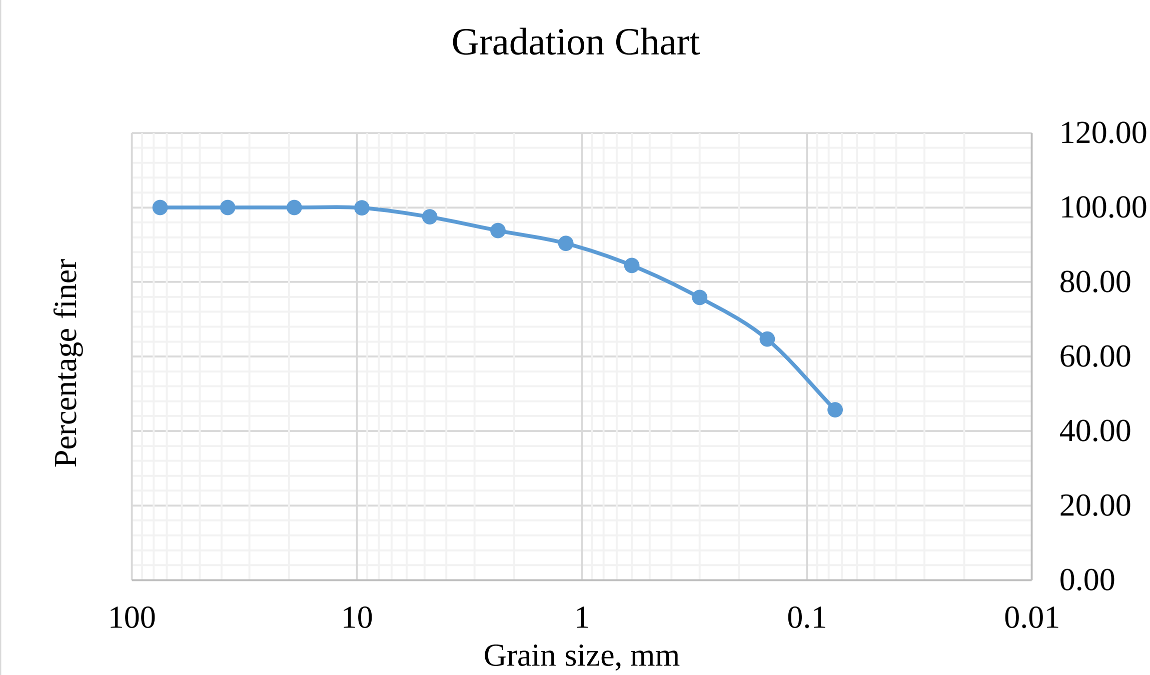


Figure E-1 sieve analysis gradation chart

Procedure for Hydrometer test

Since the soil is mostly of sand size, 75 g to 100 g air-dry sample was prepared. And dispersing agent of sodium hexametaphosphate of 40 g/L with tape water, which means 5 g or 125 ml per test sample. The sample placed and covered in a 250 ml beaker for at least 16 hrs.

Corrections

1 **Hygroscopic moisture correction:**

When the sample was weighed for the hydrometer test, an auxiliary portion of 10 g taken and oven dried at temperature of 110 ± 5°C, and weighed again. Therefore,

$$\text{Hygroscopic moisture correction} = \frac{\text{Mass of Oven dried soil}}{\text{Mass of air dried soil}} \leq 1.0$$

Table E- 2 Typical Hygroscopic moisture correction

Can Code	Weight of can	Weight of Can + Soil	Weight of Can + Oven dried Soil	Wt. of Water (g)	Wt. of Dry Soil (g)	Moisture Content (%)	weight of air dried soil (g)
0	(g) 1	(g) 2	(g) 3	4= 2-3	5=3-1	6 = (4/5)%	7= (2 - 1)
1	9.68	19.68	19.50	0.18	9.82	1.83	10.00
Hygroscopic Moisture Correction, % (= 5/7)							98.20

For specific test number #27, weight of sample taken was, W = 100 g, hygroscopic moisture correction was 98.20 %. So dry soil used in calculation to find soil particles suspended in hydrometer analysis become W_{corr} = 100 g * 89.20 % = 98.20 g.

2 Composite Correction for Hydrometer Readings

The net amount of the corrections for the three items specific gravity change, temperature variation, and meniscus correction is designated as the composite correction. The value is negative and is added to the hydrometer readings. The composite correction considers the following conditions:

- The hydrometer is graduated by the manufacturer to be read at the bottom of the meniscus formed by the liquid on the stem. Since it is impossible to read the hydrometer through the soil suspension, it becomes necessary to read the top of the meniscus. Reading the top of the meniscus results in a lower value than actual.

- Hydrometers are calibrated to read 1.00 in distilled water at 20° C. However, the addition of sodium metaphosphate (the dispersing agent) increases the specific gravity of the solution, thereby altering the readings. This increase must be determined in order to correct the readings. A composite correction has to be determined for each individual hydrometer reading. For convenience, a graph of composite corrections for a series of 0.1°C temperature differences for the range of expected test temperatures was prepared. Measurement of the composite corrections made at temperatures spanning the range of expected test temperatures, and corrections for the intermediate temperatures calculated assuming a straight-line relationship between the two observed values, using ASTM D 422, section 7.2. All other procedures followed as section 7.3 and the rest of ASTM D 422.

Table E- 3 Sample composite correction for a hydrometer bulb 152 H readings taken from laboratory for graph preparation

5 g/L solution temperature, ^{o}C	25.30	25.90	26.20	26.9
Composite correction factor	3	3	2.5	2.5

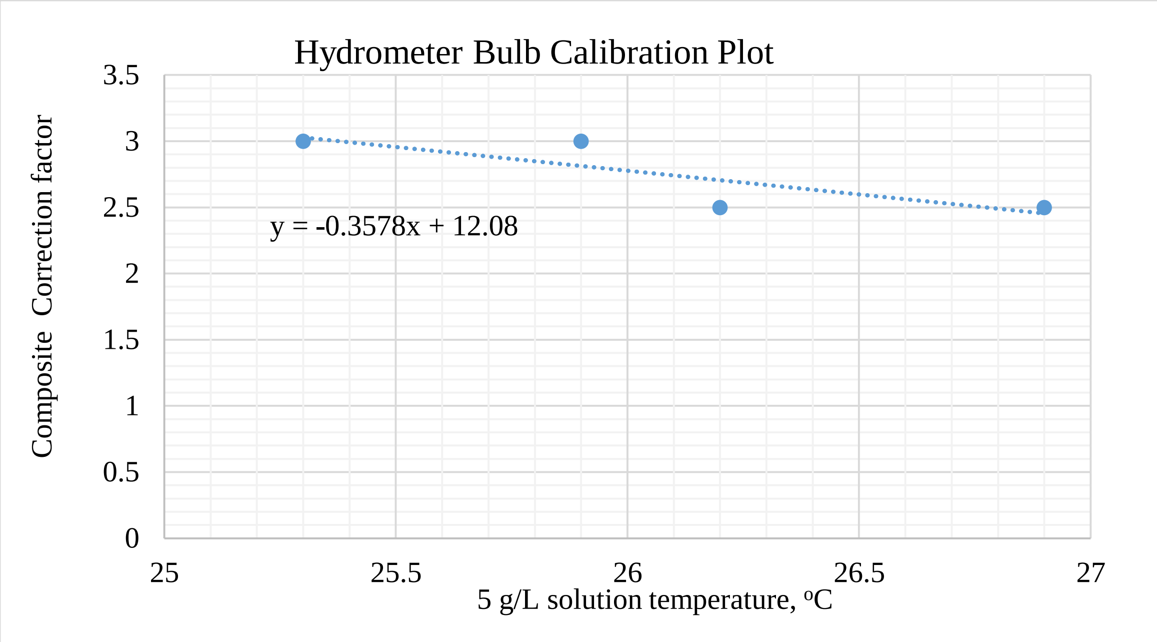


Figure E-2 Hydrometer Bulb Calibration Plot for Composite correction made at SNNPRS Bureau of Construction Regulatory department Laboratory

Table E -4 adjusted values for composite Correction factor hydrometer 152 H, for different temperature

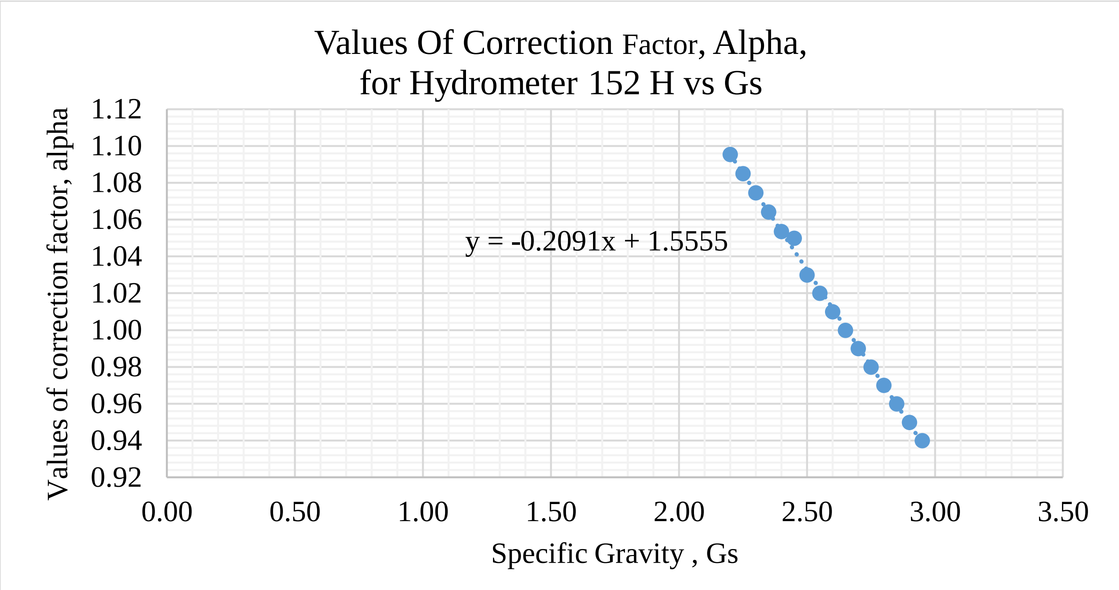
5 g/L solution temperature, ºC	22.0	22.10	22.20	.....	26.90	27.0
Composite correction factor	4.208	4.173	4.137		2.455	2.419

3 Correction for α,

Correction factor to be applied to the reading of hydrometer 152 H. The Values shown on the scale are computed using a specific gravity of 2.65 on ASTM D 422 in Table 1, but in this case the specific gravity is below this value in the table given and the table adjusted as per data available.

Table E- 5 adjusted values for Correction factor alpha, to be applied to the readings of Hydrometer 152 H, when the computed values of specific gravity is below 2.45

Specific Gravity	2.95	2.9	2.85	…	…	2.45	2.4	2.35	2.3	2.25	2.2
Correction for, α	0.94	0.95	0.96			1.05	1.05	1.06	1.07	1.09	1.10



Figur E- 3 Values of Correction Factor, Alpha, for Hydrometer 152 H Vs values of Specific gravity below 2.45

To find D which is the square root of [0.3*h*L_e/g(Gs-1)*r_w*t), it is better to use the K-value from tables than finding from the dynamic viscosity, h in poise(dyne = sec/cm^2) of water at specific temperature and calculating D.

Table E- 6 for hydrometer analysis for test number #27

Reading No	Time (1)	Elapsed Time, min (2)	Hydrometer reading, Actual (3)	Temperature °C (4)	composite correction, C (5)	Corrected hdrometer reading (6) = (3 - 5)	K- value (7)	Effective depth(L),cm (8)	$\sqrt{\frac{L}{T}}$	Particle diameter, mm (10) = (8*9)	Persent of soil in suspension (%) (11) = (6*a/w_{cor})
1	10:15:30	0.5	56	26.9	2.46436	53.53564	0.01344413	7.1	3.768289	0.0506613	57.24
2	10:16	1	53	26.9	2.46436	50.53564	0.01344413	7.6	2.75681	0.0370629	54.04
3	10:17	2	48	26.9	2.46436	45.53564	0.01344413	8.4	2.04939	0.0275523	48.69
4	10:19	4	41	26.9	2.46436	38.53564	0.01344413	9.6	1.549193	0.0208275	41.20
5	10:20	5	39	26.9	2.46436	36.53564	0.01344413	9.9	1.407125	0.0189176	39.07
6	10:30	15	25	27	2.4308	22.5692	0.01342843	12.2	0.90185	0.0121104	24.13
7	10:45	30	19	27	2.4308	16.5692	0.01342843	13.2	0.663325	0.0089074	17.72
8	11:15	60	14	27	2.4308	11.5692	0.01342843	14	0.483046	0.0064865	12.37
9	14:25	250	8	27	2.4308	5.5692	0.01342843	15	0.244949	0.0032893	5.95
10	10:15	1440	5	27.5	2.263	2.737	0.01335362	15.5	0.103749	0.0013854	2.93

4 Combined analysis

From the test made by mechnical sieving and and sedimentation by hydrometer method combined for the proper analysis of fine grained soil. For Sieve analysis the mass taken was 500 grams and for hydrometer some portion of it was taken, which is

100 grams. Therefore, a correction for mass was made to combine both test. Percent fine or percent passing No.200 sieve was 45.71 % and percent suspended at 30 second was 57.24 %. Therefore, 57.24 % of 45.71 % is taken and become(57.24/100)*45.71 % = 26.164 % and continued for all successive steps upto 1440 minutes (2.93/100)*45.71 % = 1.34 %.

Table E - 7 typical combined gradation test

Sieve No	Sieve Opening (mm)	Weight of Sieve (g)	Weight of sieve + Retained soil (g)	Weight of Retained soil(g)	Percentage Retained (g)	Cumulative Percentage (g)	Percentage Passing (g)
1	2	3	4	5=[4]-[3]	6=([5]/W_f) %		
3"	75	0	0	0	0.00	0.00	100.00
1 1/2"	37.5	0	0	0	0.00	0.00	100.00
3/4"	19	0	0	0	0.00	0.00	100.00
3/8"	9.5	544.3	544.9	0.6	0.12	0.12	99.88
No 4	4.75	573.72	585.77	12.05	2.42	2.54	97.46
No 8	2.36	548.57	566.77	18.2	3.66	6.2	93.80
No 16	1.18	506.89	524	17.11	3.44	9.64	90.36
No 30	0.6	476.4	505.8	29.4	5.91	15.55	84.45
No 50	0.3	426.11	469.05	42.94	8.63	24.18	75.82
No 100	0.15	416.51	472.16	55.65	11.18	35.36	64.64
No 200	0.075	397.72	491.91	94.19	18.93	54.29	45.71
	0.050661						26.17
	0.037063						24.70
	0.027552						22.26
	0.020828						18.83
	0.018918		C_u = D_{60}/D_{10} = 11.99				17.86
	0.01211						11.03
	0.008907		C_c = $(D30)^{2}/D_{60}*D_{10}$		= 2.13		8.10
	0.006487						5.65
	0.003289						2.72
	0.001385						1.34

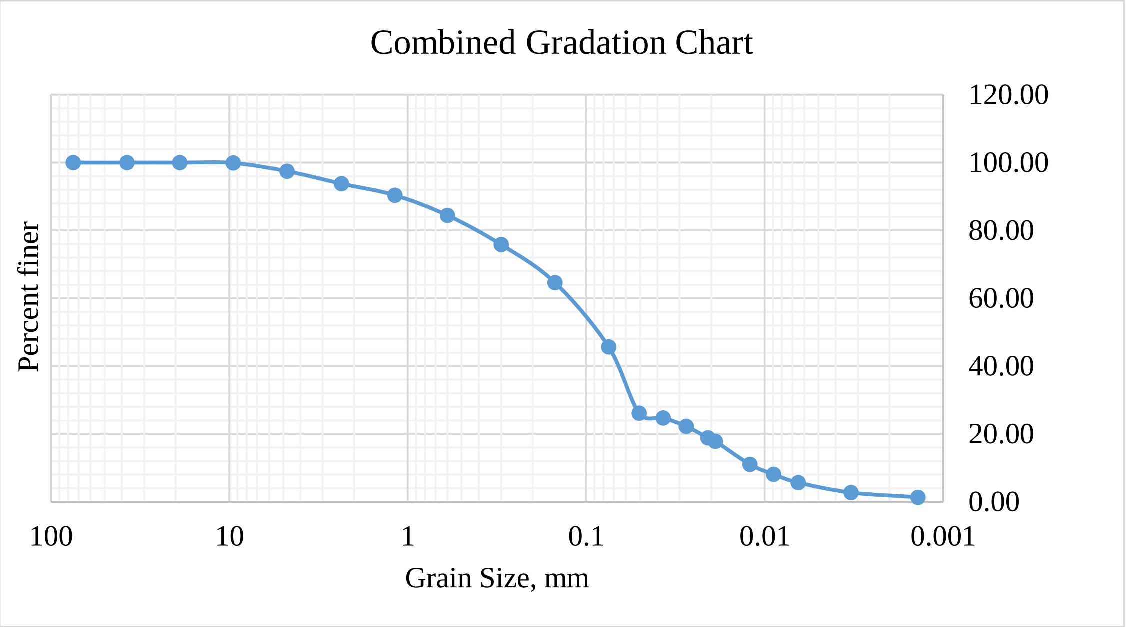


Figure E-4 Combined Gradation Chart (sieve analysis and hydrometer test)

Project	M.sc Thesis work
Location	Hawassa, Bahl Adrash sub city, Near UN Office
Test pit number	03
Depth	3.0 m
Type of test	Specific Gravity
Test method	ASTM D 854 - 98

Table E.8 Sieve analysis and Hydrometer combined

Sieve No	Sieve Opening (mm)	Weight of Sieve (g)	Weight of sieve + Retained soil (g)	Weight of Retained soil (g)	Percentage Retained (g)	Cumulative Percentage (g)	Percentage Passing (g)
1	2	3	4	5=[4]-[3]	6=([5]/W_f)%		
3"	75	0	0	0	0.00	0.00	100.00
1 1/2"	37.5	0	0	0	0.00	0.00	100.00
3/4"	19	0	0	0	0.00	0.00	100.00
3/8"	9.5	544.32	553.42	9.1	1.84	1.84	98.16
No 4	4.75	573.45	619.55	46.1	9.30	11.14	88.86
No 8	2.36	548.66	603.87	55.21	11.14	22.28	77.72
No 16	1.18	506.69	550.22	43.53	8.78	31.06	68.94
No 30	0.6	476.25	511.84	35.59	7.18	38.24	61.76
No 50	0.3	425.3	453.8	28.5	5.75	43.99	56.01
No 100	0.15	419.69	455.47	35.78	7.22	51.21	48.79
No 200	0.075	397.78	481.93	84.15	16.98	68.19	31.81
	0.05363						16.05
	0.039104						15.05
	0.02846						14.05
	0.02101						12.38
	0.01918	$C_u = D_{60}/D_{10} = 33.07$					11.71
	0.01186						8.42
	0.00873	$C_c = (D30)^{\wedge 2}/D_{60*}D_{10} = 0.63$					6.44
	0.00631						5.11
	0.00323	$S_o = (D_{75}/D_{25})^{\wedge 1/2} = 5.91$					2.78
	0.00139						1.32

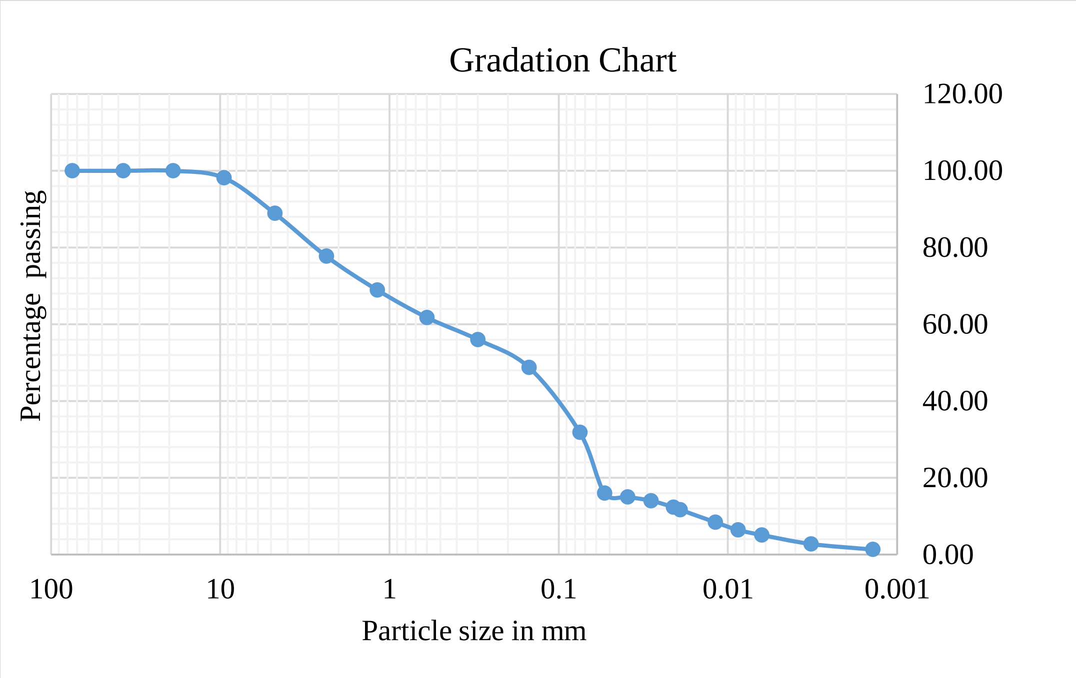


Figure E.5 Graduation Chart

Project	M.sc Thesis work
Location	Hawassa, Menahria sub city, Textile Factory
Test pit number	04
Depth	3.0 m
Type of test	Specific Gravity
Test method	ASTM D 854 - 98

Table E.9 Sieve analysis and Hydrometer combined

Sieve No	Sieve Opening (mm)	Weight of Sieve (g)	Weight of sieve + Retained soil (g)	Weight of Retained soil (g)	Percentage Retained (g)	Cumulative Percentage (g)	Percentage Passing (g)
1	2	3	4	=[4] -[3]	6=([5]/W_f)%		
3"	75	0	0	0	0.00	0.00	100.00
1 1/2"	37.5	0	0	0	0.00	0.00	100.00
3/4"	19	0	0	0	0.00	0.00	100.00
3/8"	9.5	544.32	554.67	10.35	2.08	2.08	97.92
No 4	4.75	573.47	618.3	44.83	9.02	11.10	88.90
No 8	2.36	548.47	619.79	71.32	14.35	25.45	74.55
No 16	1.18	506.64	574.64	68	13.68	39.13	60.87
No 30	0.6	476.31	535.87	59.56	11.98	51.12	48.88
No 50	0.3	425.27	480.12	54.85	11.04	62.15	37.85
No 100	0.15	420.15	458.61	38.46	7.74	69.89	30.11
No 200	0.075	397.91	457.06	59.15	11.90	81.79	18.21
	0.048714						10.35
	0.035198						9.97
	0.02609						9.22
	0.01937						8.28
	0.01762	$C_u = D_{60}/D_{10} = 31.21$					7.91
	0.01112						6.06
	0.00820	$C_c = (D30)^{\wedge 2}/D_{60*}D_{10} = 0.54$					4.94
	0.00602						4.01
	0.00309	$S_o = (D_{75}/D_{25})^{\wedge 1/2} = 4.55$					2.50
	0.00135						1.31

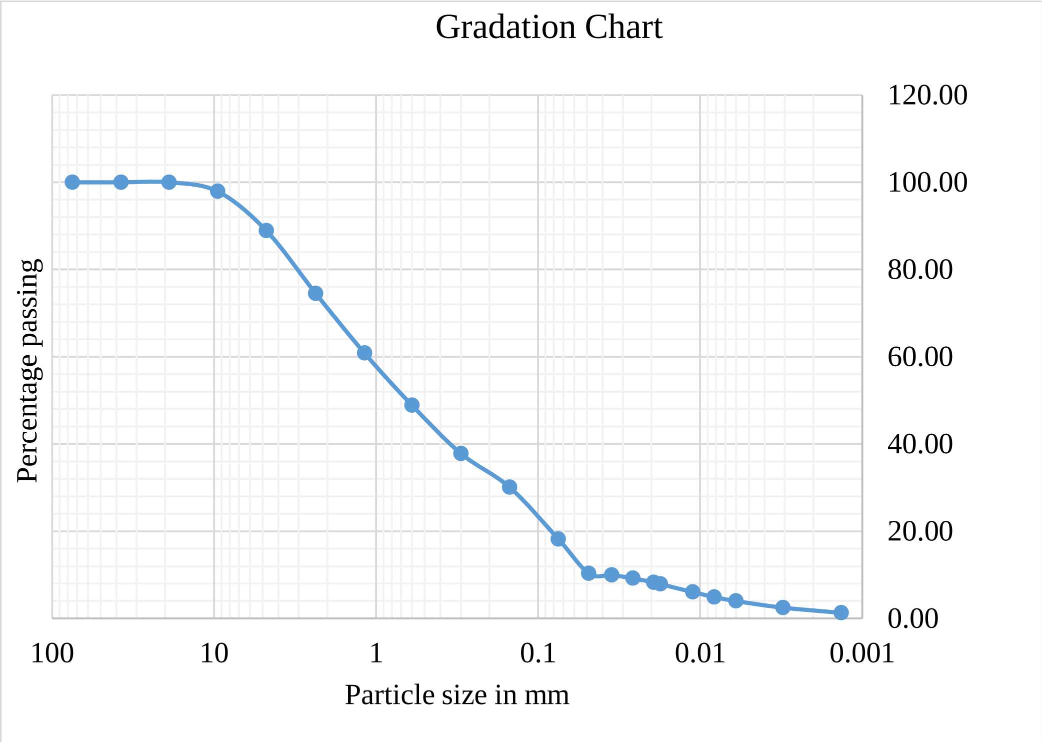


Figure E.6 Graduation Chart

APPENDIX F SUMMARY OF ALL TEST RESULTS

Table F. 1 summary of all test results tabulated

Code	φ (degree)	Cohesion (kN/m^2)	In-situ density (g/cm^3)	NMC (%)	OMC (%)	MDD (g/cm^3)
Test #01	29.11	0	1.14	25.71	24.50	1.44
Test #02	26.64	0	1.21	7.73	22.49	1.44
Test #03	30.28	0	1.30	36.57	19.00	1.33
Test #04	32.54	0	1.14	22.84	28.00	1.34
Test #05	31.32	8.55	1.25	11.14	24.75	1.36
Test #06	29.85	0	1.08	17.78	25.90	1.34
Test #07	28.32	0	1.09	16.77	21.30	1.49
Test #08	29.3	0	1.25	17.48	21.80	1.48
Test #09	25.4	26.48	1.41	32.75	25.90	1.41
Test #10	29.25	0	1.15	26.18	33.00	1.18
Test #11	28.57	19.02	1.09	23.73	23.00	1.34
Test #12	27.32	21.06	1.35	32.48	25.00	1.36
Test #13	29.78	3.77	1.31	34.46	33.60	1.32
Test #14	29.87	0.03	1.29	25.92	27.50	1.37
Test #15	33.16	0	1.21	20.08	21.80	1.45
Test #16	31.12	0.9	1.14	9.06	19.50	1.37
Test #17	35.1	0	1.16	7.62	21.50	1.42
Test #18	30.93	0	1.09	10.81	23.90	1.39
Test #19	30.81	0	1.02	26.49	31.30	1.18
Test #20	30.47	0	1.05	25.65	29.40	1.39
Test #21	30.34	0	1.12	25.06	28.90	1.29
Test #22	31.27	0	1.12	25.73	35.00	1.21
Test #23	32.59	0	1.18	10.25	24.50	1.42
Test #24	29.64	0	1.33	38.66	31.70	1.35
Test #25	31.51	0	1.04	13.54	24.90	1.42
Test #26	27.37	9.57	1.37	33.10	33.00	1.38
Test #27	31.4	0	1.00	18.64	26.35	1.36
Test #28	28.43	8.65	1.28	36.61	32.00	1.34
Test #29	31.42	0	1.20	12.67	30.00	1.33
Test #30	29.09	8.45	1.16	12.67	27.00	1.32
Test #31	32.59	0	1.09	9.83	23.60	1.44
Test #32	27.12	0	1.29	32.88	26.00	1.43
Test #33	23.13	11.38	1.24	20.50	25.40	1.41
Test #34	27.05	18.24	1.13	21.54	25.70	1.44

Code	Gs (o.dry)	Gs (wet)	LL (%)	PL (%)	PI (%)	D_{60} (mm)	D_{50} (mm)	D_{30} (mm)
Test #01	2.38	2.45	0	0	0	0.38	0.23	0.11
Test #02	2.38	2.40	0	0	0	0.11	0.07	0.05
Test #03	2.39	2.48	0	0	0	0.51	0.18	0.07
Test #04	2.42	2.53	0	0	0	1.14	0.65	0.15
Test #05	2.40	2.46	0	0	0	0.30	0.20	0.10
Test #06	2.39	2.39	0	0	0	0.16	0.11	0.06
Test #07	2.40	2.43	0	0	0	0.53	0.34	0.11
Test #08	2.45	2.45	0	0	0	0.29	0.18	0.10
Test #09	2.43	2.57	0	0	0	0.80	0.52	0.20
Test #10	2.22	2.25	0	0	0	0.76	0.43	0.12
Test #11	2.34	2.45	0	0	0	0.23	0.15	0.09
Test #12	2.39	2.45	0	0	0	0.17	0.13	0.07
Test #13	2.37	2.49	37	0	0	1.02	0.65	0.25
Test #14	2.43	2.26	38	31	6	0.94	0.61	0.25
Test #15	2.31	2.41	34	29	6	1.07	0.50	0.10
Test #16	2.34	2.37	33	0	0	0.11	0.07	0.04
Test #17	2.37	2.49	0	0	0	0.21	0.14	0.08
Test #18	2.39	2.44	0	0	0	0.11	0.08	0.03
Test #19	2.26	2.45	0	0	0	0.25	0.14	0.05
Test #20	2.31	2.43	0	0	0	0.36	0.19	0.09
Test #21	2.32	2.43	0	0	0	0.25	0.14	0.08
Test #22	2.30	2.41	0	0	0	0.24	0.14	0.08
Test #23	2.43	2.44	0	0	0	0.22	0.14	0.07
Test #24	2.54	2.67	36	30	6	0.41	0.27	0.11
Test #25	2.35	2.43	0	0	0	0.22	0.13	0.06
Test #26	2.55	2.63	40	35	5	1.47	1.05	0.51
Test #27	2.33	2.45	0	0	0	0.13	0.09	0.06
Test #28	2.35	2.36	42	36	6	0.77	0.42	0.12
Test #29	2.41	2.49	0	0	0	0.64	0.45	0.21
Test #30	2.50	2.43	0	0	0	0.08	0.07	0.03
Test #31	2.36	2.52	0	0	0	0.19	0.13	0.07
Test #32	2.36	2.51	0	0	0	0.63	0.40	0.13
Test #33	2.37	2.52	0	0	0	0.09	0.07	0.06
Test #34	2.38	2.52	0	0	0	0.09	0.05	0.03

Code	D_{25} (mm)	D_{15} (mm)	D_{10} (mm)	Cu	Cc	Gravel (%)	Total Sand (%)
Test #01	0.09	0.04	0.02	20.05	1.57	8.77	70.48
Test #02	0.02	0.01	0.01	13.21	2.23	4.10	44.41
Test #03	0.06	0.03	0.02	33.07	0.63	11.14	57.05
Test #04	0.12	0.06	0.04	31.48	0.54	11.10	70.69
Test #05	0.08	0.06	0.03	10.16	1.14	3.91	73.21
Test #06	0.05	0.02	0.01	16.28	2.19	4.55	54.27
Test #07	0.08	0.03	0.02	30.76	1.33	6.72	68.82
Test #08	0.08	0.06	0.03	10.34	1.19	4.84	72.97
Test #09	0.14	0.07	0.03	27.14	1.73	4.98	79.36
Test #10	0.09	0.03	0.02	39.76	1.04	5.16	73.05
Test #11	0.07	0.06	0.03	9.34	1.26	0.29	73.12
Test #12	0.06	0.03	0.02	11.12	1.97	2.24	65.82
Test #13	0.18	0.09	0.06	17.21	1.07	5.70	80.84
Test #14	0.18	0.07	0.06	17.08	1.18	5.34	79.52
Test #15	0.07	0.03	0.01	102.03	0.97	15.37	58.87
Test #16	0.02	0.01	0.01	14.30	1.53	1.80	43.33
Test #17	0.07	0.03	0.01	14.69	2.08	3.86	67.69
Test #18	0.02	0.01	0.01	18.26	1.01	2.18	49.15
Test #19	0.03	0.02	0.01	21.85	1.03	3.76	49.60
Test #20	0.07	0.04	0.02	19.36	1.12	5.56	67.82
Test #21	0.07	0.03	0.02	14.35	1.33	1.89	68.15
Test #22	0.07	0.03	0.02	15.11	1.46	2.86	67.37
Test #23	0.06	0.03	0.01	16.59	1.47	1.10	64.90
Test #24	0.08	0.04	0.02	24.23	1.74	1.76	74.89
Test #25	0.05	0.02	0.01	18.79	1.37	7.53	55.04
Test #26	0.40	0.21	0.13	11.18	1.34	5.85	88.22
Test #27	0.04	0.02	0.01	11.99	2.13	2.54	51.75
Test #28	0.09	0.06	0.02	41.23	0.94	3.06	74.62
Test #29	0.16	0.10	0.07	8.96	0.99	0.69	87.97
Test #30	0.02	0.01	0.01	10.41	1.72	2.70	39.41
Test #31	0.06	0.02	0.01	17.10	1.99	1.81	61.72
Test #32	0.10	0.05	0.02	33.28	1.47	4.51	75.09
Test #33	0.04	0.02	0.01	7.92	2.94	1.63	42.75
Test #34	0.02	0.01	0.01	13.16	1.27	1.34	41.08

	Sand					
Code	Coarse sand 4.75mm to 2.0mm (%)	Medium sand 2.0mm to 0.425 mm (%)	Fine sand 0.425 mm to 0.075mm (%)	Passing #200 (%)	Silt size (%)	Clay size (%)
Test #01	13.83	18.42	38.23	20.75	35.51	3.95
Test #02	6.18	11.06	27.17	51.49	81.47	7.40
Test #03	15.35	16.22	25.49	31.81	44.74	5.09
Test #04	17.82	30.79	22.08	18.21	31.62	4.34
Test #05	9.78	24.12	39.32	22.88	28.44	3.70
Test #06	9.62	15.13	29.52	41.17	64.68	6.52
Test #07	13.07	30.47	25.28	24.78	43.52	7.18
Test #08	8.78	23.60	40.59	22.19	30.47	4.70
Test #09	18.68	35.60	25.08	15.65	34.74	5.99
Test #10	19.54	27.99	25.52	21.79	43.75	5.24
Test #11	3.84	25.87	43.41	26.59	31.58	2.90
Test #12	1.08	17.69	47.05	30.82	43.25	3.58
Test #13	23.03	35.25	22.56	13.46	27.77	3.79
Test #14	21.17	36.76	21.59	15.14	32.85	5.31
Test #15	17.71	20.82	20.34	25.76	52.43	8.71
Test #16	4.70	13.04	25.58	54.87	85.49	8.40
Test #17	9.17	16.44	42.08	28.44	45.01	4.83
Test #18	4.23	11.14	33.78	48.67	94.41	10.46
Test #19	6.67	22.53	20.40	46.63	67.63	5.23
Test #20	12.32	22.64	32.86	26.62	35.76	2.49
Test #21	7.90	24.33	35.92	29.97	40.40	3.24
Test #22	7.88	22.56	36.93	29.78	43.20	4.74
Test #23	8.20	21.21	35.50	34.00	48.73	5.28
Test #24	10.55	31.90	32.45	23.35	23.35	5.15
Test #25	10.77	14.63	29.65	37.41	37.42	7.90
Test #26	30.53	40.43	17.26	5.93	15.78	2.96
Test #27	5.47	14.98	31.30	45.71	63.53	5.42
Test #28	20.88	27.66	26.08	22.33	40.67	7.54
Test #29	13.05	44.76	30.16	11.34	13.85	1.39
Test #30	5.07	10.09	24.25	57.90	81.10	6.73
Test #31	6.80	20.09	34.82	36.47	58.33	6.82
Test #32	17.90	29.95	27.24	20.40	38.19	4.99
Test #33	3.77	11.30	27.68	55.62	62.50	5.08
Test #34	3.81	10.77	26.50	56.82	81.99	7.55

Table F. 2 a case wise diagnosis by using the relation equation for silty sand

Case Number	Std. Residual	Measured (φ)	Predicted Value of (φ)	Residual
1	0.17	29.11	28.95	0.16
2	-0.22	30.28	30.48	-0.20
3	-0.35	32.54	32.86	-0.32
4	-0.30	31.32	31.59	-0.27
5	-1.90	29.85	31.56	-1.71
6	-0.31	28.32	28.60	-0.28
7	-0.50	29.30	29.75	-0.45
8	-0.63	25.40	25.97	-0.57
9	0.17	29.25	29.10	0.15
10	-0.77	28.57	29.27	-0.70
11	-0.06	27.32	27.37	-0.05
12	0.58	29.78	29.26	0.52
13	2.10	35.10	33.21	1.89
14	0.49	30.93	30.49	0.44
15	0.97	30.81	29.94	0.87
16	0.25	30.47	30.25	0.22
17	-0.78	30.34	31.04	-0.70
18	1.53	31.27	29.89	1.38
19	-0.64	32.59	33.16	-0.57
20	0.17	31.51	31.35	0.16
21	-0.13	31.40	31.52	-0.12
22	-0.32	32.59	32.87	-0.28
23	0.49	27.12	26.68	0.44

Table F. 3 a case wise diagnosis by using the relation equation for silty sand, silty clayey sand and silty, clayey sand with gravel

Case Number	Std. Residual	Measured value of φ	Predicted Value of φ	Residual
1	-0.20	29.11	29.37	-0.26
2	-0.08	30.28	30.39	-0.11
3	0.30	32.54	32.14	0.40
4	-0.32	31.32	31.75	-0.43
5	-0.75	29.85	30.85	-1.00
6	-1.38	28.32	30.16	-1.84
7	-0.80	29.30	30.37	-1.07
8	-1.50	25.40	27.40	-2.00
9	-0.83	29.25	30.36	-1.11
10	-1.15	28.57	30.10	-1.53
11	-0.69	27.32	28.25	-0.93
12	1.05	29.78	28.38	1.40
13	0.56	29.87	29.12	0.75
14	0.02	33.16	33.14	0.02
15	2.10	35.10	32.29	2.81
16	-0.13	30.93	31.10	-0.17
17	-0.09	30.81	30.93	-0.12
18	-0.30	30.47	30.88	-0.41
19	-0.08	30.34	30.44	-0.10
20	1.25	31.27	29.60	1.67
21	0.23	32.59	32.28	0.31
22	2.30	29.64	26.57	3.07
23	0.28	31.51	31.14	0.37
24	-0.04	27.37	27.42	-0.05
25	-0.07	31.40	31.49	-0.09
26	0.37	28.43	27.94	0.49
27	0.61	32.59	31.77	0.82
28	-0.68	27.12	28.02	-0.90

Appendix G regression analysis for silty sand

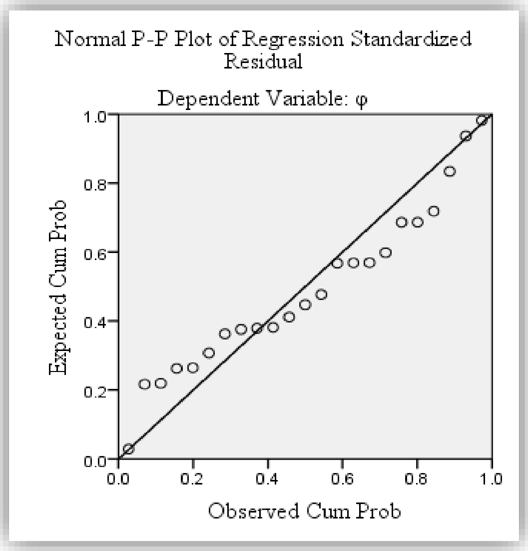


Figure G -1 Normal P-P plot of regression

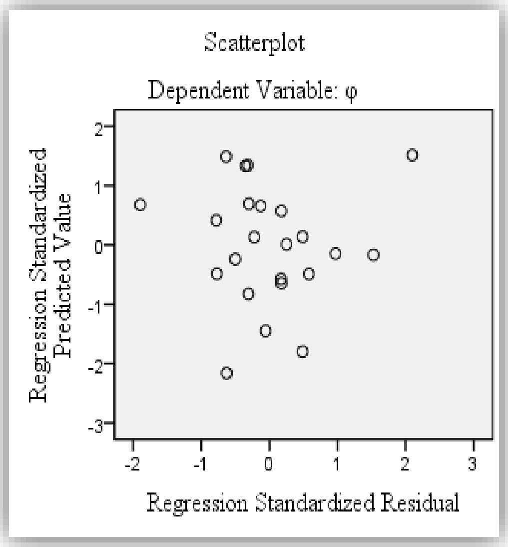
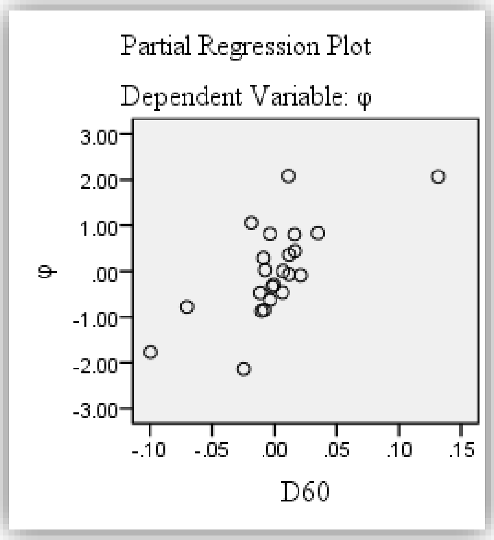


Figure G -2 Scatter plotstandardized Residual



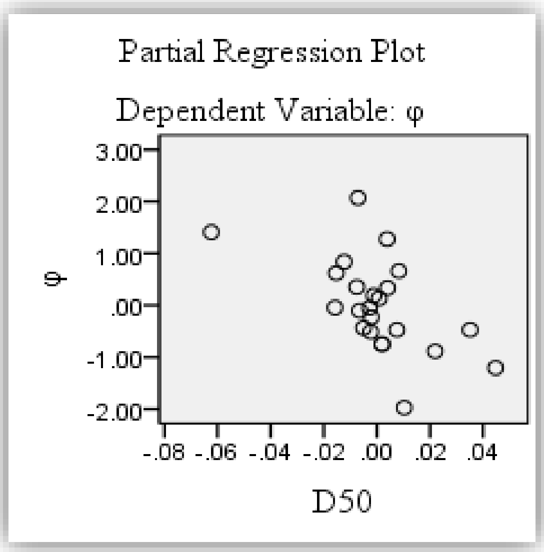


Figure G-3 D_{60} vs φ

Figure G. 4 D50 vs φ

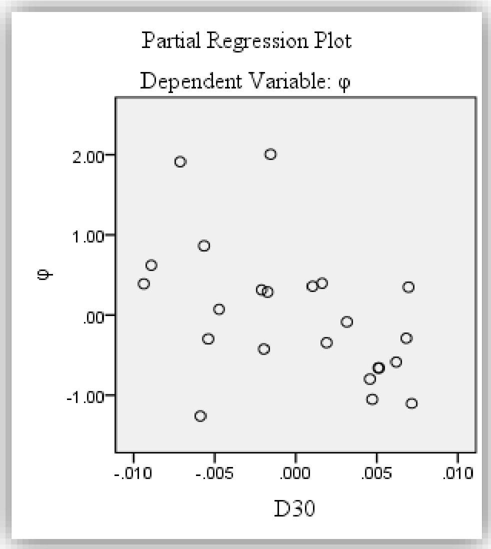


Figure G-5 D_{30} vs φ

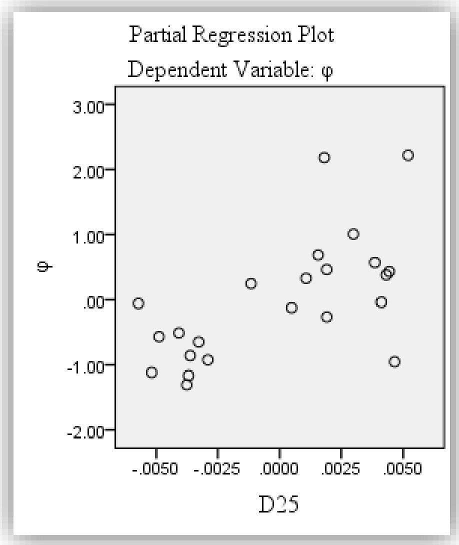


Figure G. 6 D_{25} vs φ

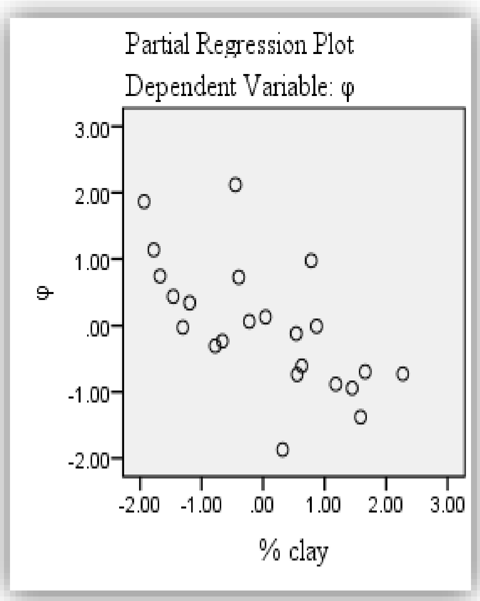


Figure G-7percentage clay vs φ

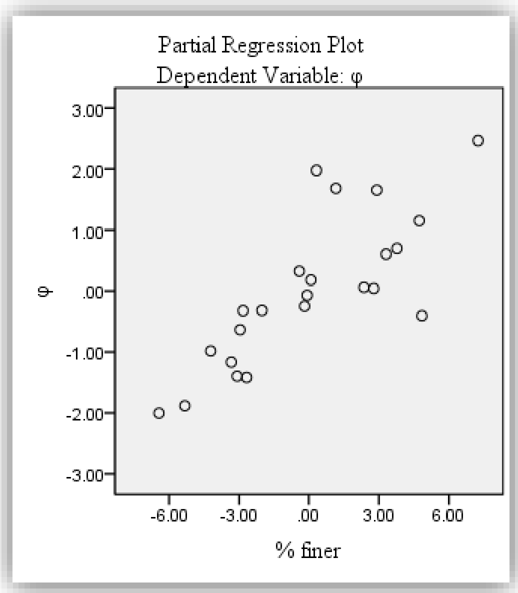


Figure G- 8percent finer vs φ

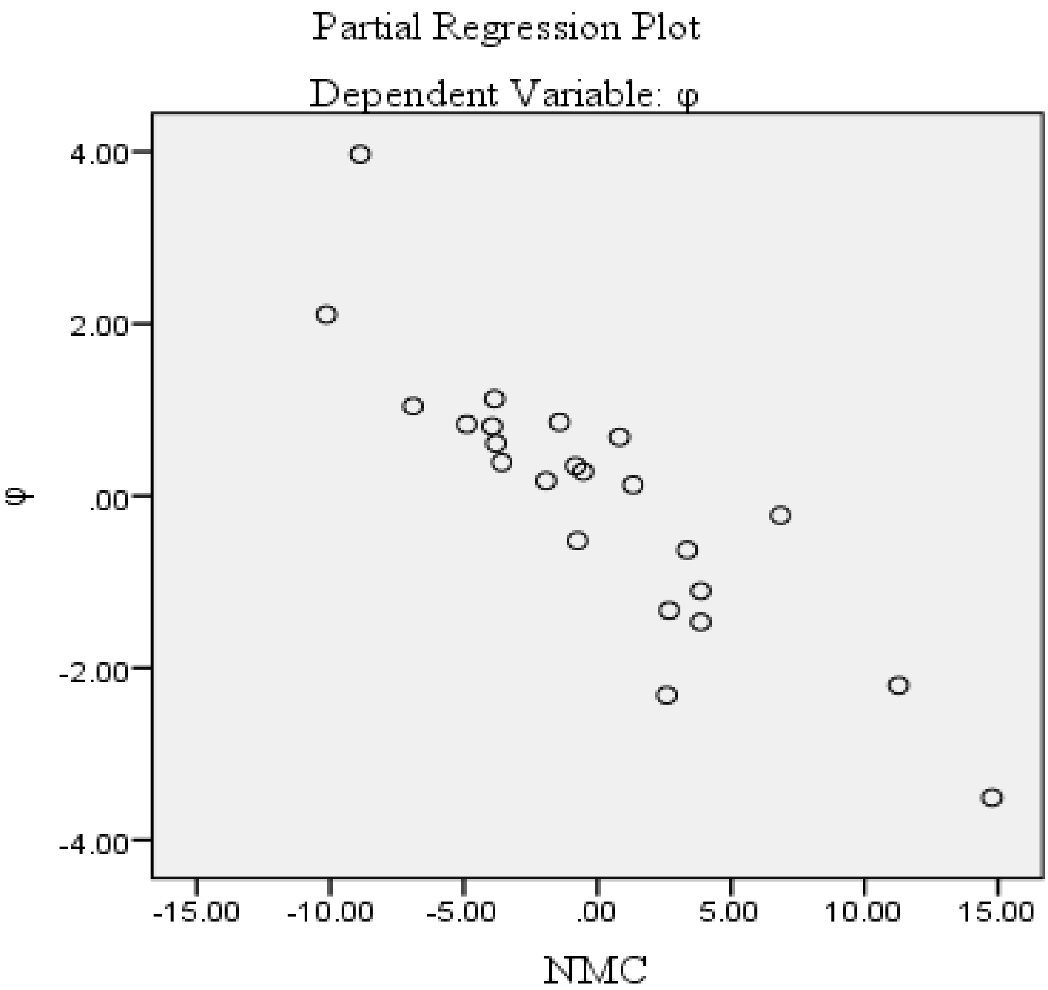
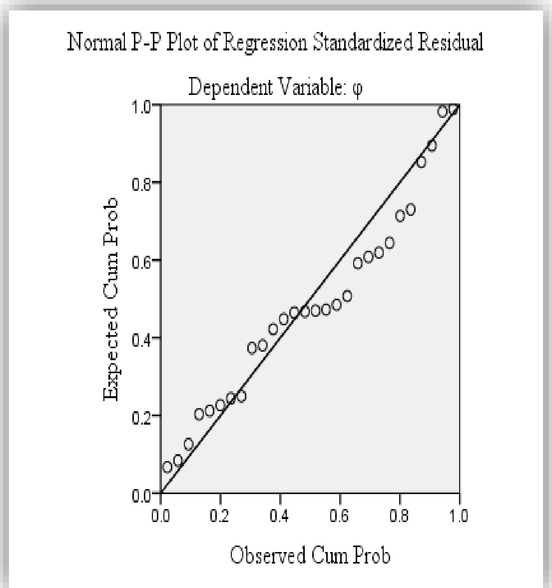


Figure G- 9 NMC vs φ

Appendix H regression analysis for silty sand, silty, clayey sand and silty, clayey sand with gravel



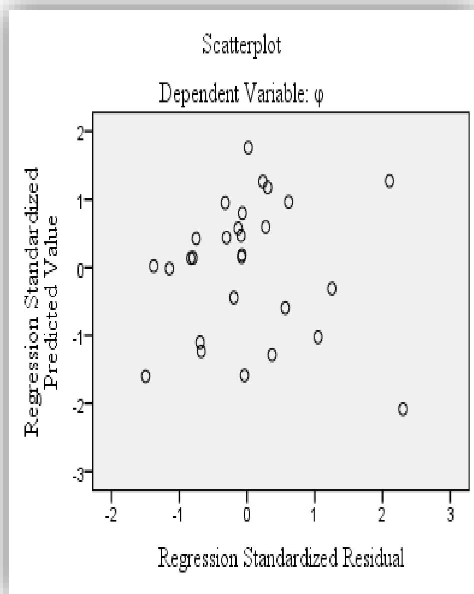
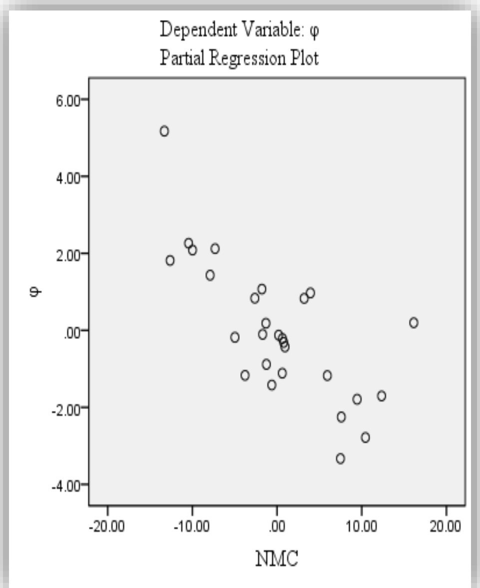


Figure H -1Normal P-P plot of regression standardized Residual

Figure H-2 Scatter plot



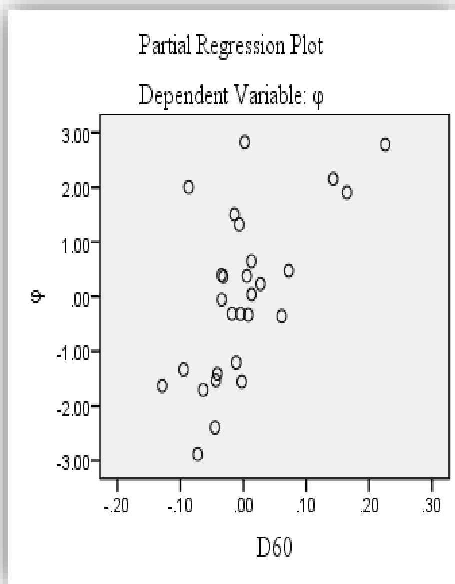


Figure H-3 NMC vs φ

Figure H-4 D_{60} vs φ

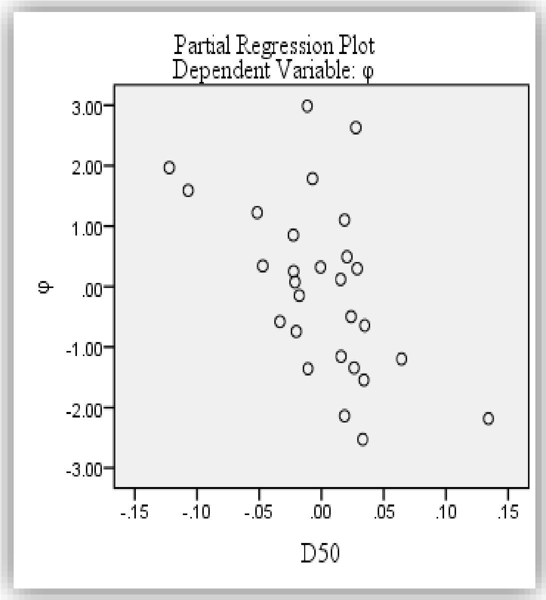


Figure H- 5D_{50} vs φ

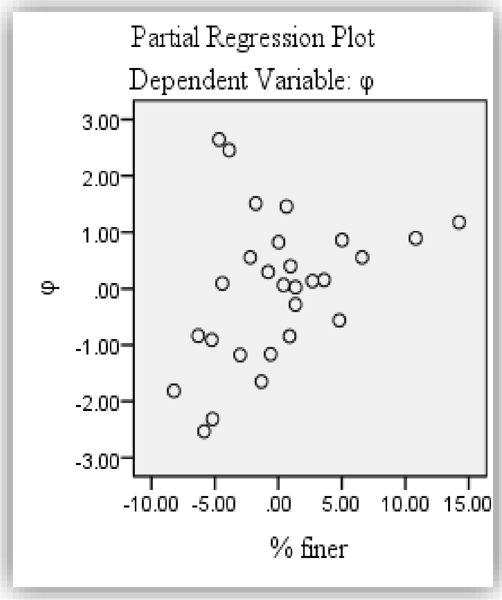


Figure H-6 percent finer vs φ

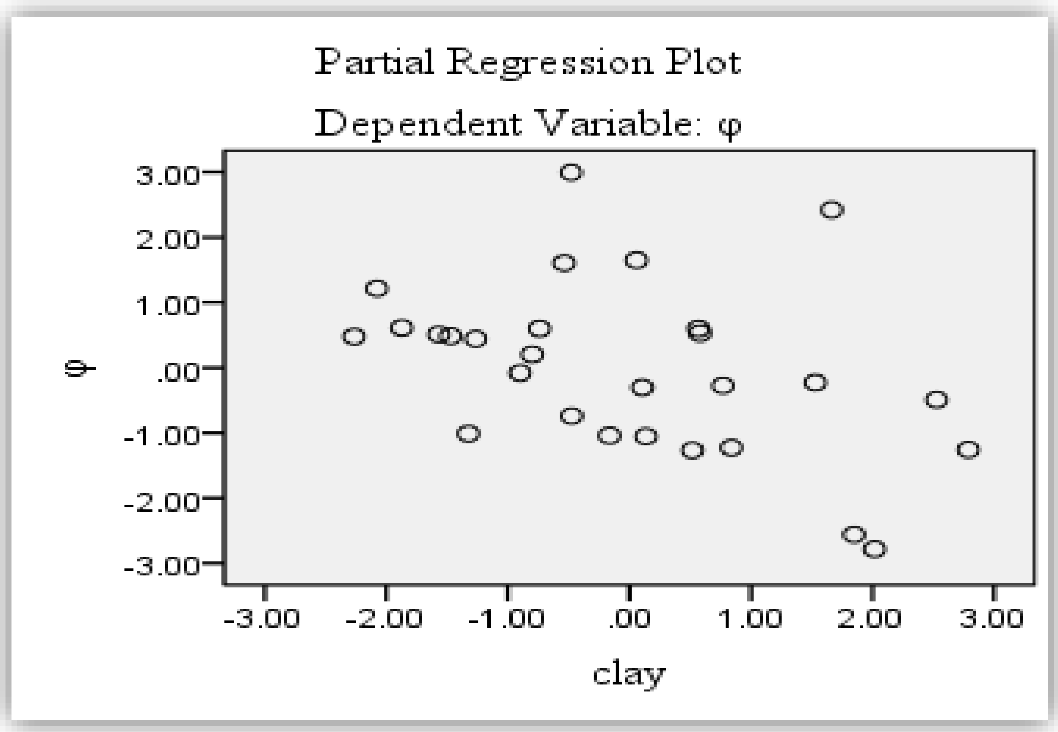


Figure H – 7 percent clay vs φ

Appendix – I Certificate of Calibration for Direct Shear Test Equipment

NATIONAL METROLOGY INSTITUTE OF ETHIOPIA
(NMIE)

CERTIFICATE OF CALIBRATION

This calibration certificate documents the traceability to national standards, which realize the units of measurement according to the international system of units

PO BOX 5722
Addis Ababa, Ethiopia
Tel: 251-11- 6517985
Fax: 251-11-6459312
e-mail: info@ nmie.net
website: http://www.nmie.net

Date of Issue: 2016-11-10 Certificate number: OFS-143 Page 1 of 3

Object		**Force Proving Ring For Shear Testing Machine**
Manufacturer		**Techno test**
Type		**BB003**
Serial Number	Gauge	**10044103**
	Machine	**BA713627**
Customer		**S/N/N/P/ Regional State Construction Bureau (Hawasa)**
Registration no.:		**09-0084**
Number of pages of the certificate		**3**
Date of Calibration:		**2016-10-25**

This calibration certificate may not be reproduced other than in full except with the permission of the issuing authority. Calibration certificates without signature and seal are not valid.

Seal Date	Approved By	Calibrated By
2016-11-02	Miliyon Yohans	Afework Gemeda

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Date of Issue: 2016-11-10   Certificate number: OFS-143   Page 2 of 3

Calibrated Object

Force Proving Ring For Shear Testing Machine

Identification No.:	---
Measuring range:	**0 to 2.5 KN**
Calibration range:	**0 to 1.27 KN**
Scale interval:	**0.001 mm (Dial Gauge Resolution)**

Standards **100 KN Load Cell**

Calibration procedure

The calibration was performed in accordance to procedures
Vertical compression force had been applied axially on both the ring under test and the reference-standard Load Cell.
The applied force is set by the certificate value of the reference and three sets of readings are taken from the dial gauge of the ring under test at each data point.

Location of Calibration Client's Material Testing Laboratory (Hawasa)

Environmental conditions On site calibration

Validity of calibration

The values in this certificate are correct at the time of calibration. Subsequently the accuracy will depend on such factors as the care exercised in handling and use of the instrument and the frequency of use. Recalibration should be performed after a period, which has been chosen to ensure that the instrument's accuracy remains within the desired limits.

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Measuring Results

Average Dial reading [Divisions]	Applied Force [KN]	Ring factor C_R [N/div]
0	0.00	...
100	0.15	1.45
200	0.27	1.37
300	0.39	1.31
400	0.52	1.30
500	0.66	1.31
600	0.77	1.28
700	0.90	1.29
800	1.02	1.28
900	1.15	1.27
1000	1.27	1.27
Average ring factor (N/div)		1.30

Uncertainty:

Estimated uncertainty of measurement: ± 160.3 N (per reading)
The uncertainties are based on root sum square of the contributions with a confidence interval not less than 95% and coverage factor k=2.

End of Certificate