ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

DEPARTEMTNET OF CIVIL AND ENVIRONMENTAL ENGINEERING

AN ALTERNATIVE FOR STABILIZATION OF ADDIS ABABA EXPANSIVE SOIL BY CRUSHED AND NATURAL SAND

BY

HIWOT SIMON

JUNE 2016

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A Thesis Submitted to the School of Graduates of Addis Ababa University in Partial Fulfillment of the Requirement for the Degree of Master of Science in Civil Engineering

By

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DECLARATION

I hereby declare that this thesis is my original work that was carried out under the supervision of Professor Alemayehu Teferra. Furthermore, this thesis is not presented in any other university or institution for the award of degree or diploma. All sources of materials used for this thesis have been duly acknowledged.

Candidate's

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Date: June 2016

Acknowledgement

First of all, I praise God for granting me with the health and strength. Nothing is possible without him.

I would also like to thank my advisor Professor Alemayehu Teferra for his guidance and support. I extend my gratitude to Dr.Ing. Samuel Tadesse and Dr. Henok for their valuable comments during the progress report presentation.

My heartfelt thanks go to my mentor and boss, Ing. Melles Girmay, General Manager of MGM Consulting Architects and Engineers P.L.C. for encouraging and giving me the time I needed.

Many thanks to Ing. Wondesen Bekele, General Manager of Wonder Construction P.L.C for his great cooperation on site in the period of collection of materials.

I gratefully acknowledge Ethiopian Road Authority for providing me the sponsorship.

Last but not least, my special thank go to my beloved husband, Ato Daniel Aleyu , my daughter, Shalom Daniel, my little twin, Zoe Daniel and Yodahe Daniel for their love that nourish me.

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SYMBOLS AND ABBERVATIONS

AASHTO – American Association of Highway and Transportation Officals

ASTM – American Society for Testing and Materials

c- Cohesion

CBR – California Bearing Ratio

CS – Crushed Sand

ERA – Ethiopian Road Authority

ES – Expansive Soil

ISO – International Organization for Standardization

LL – Liquid Limit

MDD - Maximum Dry Density

NS – Natural Sand

OMC – Optimum Moisture Content

PI – Plastic Index

PL – Plastic Limit

q_u – Unconfined Compressive Strength

SL – Shrinkage Limit

SSD –Saturated Surface Dry

UCS – Unconfined Compression Strength Test

USCS – Unified Soil Classification System

ABSTRACT

In this thesis, an alternative for stabilization of expansive soil by crushed and natural sand were examined separately. The test results revealed that there is a significant improvement on soil consistency. The classification of the mixed soil is altered from CH to MH group for both types of sand blended. Additionally, the swelling potential and swelling pressure reduced considerably. However, the groups of the mixed soil remain under the group of A-5-7 for all proportion of the sand-soil mixture. For higher proportion of mixture a considerable improvement was attained in the value of unconfined compressive strength. Nevertheless, there is no marked progress achieved in the soaked CBR value. Finally, after making cost comparison for both types sands, 40% sand by weight of soil has showed a cost reduction when compared to total soil replacement.

1. INTRODUCTION

1.1 GENERAL

Expansive soils are found in many parts of the world. It is the most problematic soil if it is not treated carefully. Damages in buildings, roads, and railway which are constructed on expansive soils are observed so far. This is due to the swell-shrink characteristics of the expansive soil when subjected to change in moisture. Detail knowledge about the expansive soil will be helpful in devising the method of improving the nature of the soil. The expansive nature of the soil comes from the amount and type of clay particle present in the soil. The worst clay mineral which is known to expand and shrink upon wetting and drying is known as montmorillonite. Montmorillonite is composed of units made of two silica tetrahedral sheets with the center alumina octahedral sheet. The outstanding feature of montmorillonite structure is that water and other polar molecules, such as certain organic molecules, can enter between the unit layers causing to expand. [11] Most light structures that are built on expansive soils are exposed to moisture change due to many reasons. Poor drainage system is one of the reasons for moisture change. This leads to swelling of the expansive soils. On the other hand, the structures are not strong enough to balance the pressure exerted by the swelled soil. Therefore, the structure will be forced to entertain this pressure. Following drying, the soil will shrink. Through this repeated cycle of moisture change, the structures will be damaged early before their design period.

1.2 BACKGROUND OF THE PROBLEM

Soils which consist of montmorillonite mineral exhibit volume change when exposed to variation of moisture. As a result of the change in volume, structures built on the expansive soil will be subjected to an uplifting pressure. Light weight structures for instance, residential buildings, runways and railways are damaged due to the shrink and swell characteristics of such soil. The south east and south west part of the Addis Ababa city are covered by expansive soil. These parts of the city are mainly construction sites for most of the civil engineering works. Therefore, the ongoing constructions as well as the existing structures are dealing with the above mentioned problem. Additionally, the damages cause high finical loss to the country.

1.3 OBJECTIVE OF THE STUDY

1.3.1 GENERAL OBJECTIVE

The main aim of the research is to study the effect of natural and crushed sand addition to the stabilization of expansive soil.

1.3.2 SPECIFIC OBJECTIVES

1. To study the effect of natural and crushed sand on the soil consistency.
2. To study the impact of natural and crushed sand addition on the maximum dry density & optimum moisture content of the expansive soil and to recommend the maximum percentage of MDD at OMC that should be achieved in the field for new mix.
3. To study the influence of the addition of natural and crushed sand on the swelling characteristics of expansive soil.
4. To study the effect of natural and crushed sand addition on the unconfined compressive strength of the stabilized soil.
5. To investigate on the variation of CBR value of the new mixture and the original expansive soil.

1.4 METHODOLOGY

In order to achieve the stated objectives, the following methodologies have been employed.

- Review previous studies, books, thesis journals and papers related to properties of expansive soils, sand and its stabilizations by sand.
- Visual identification of the soil on the field.
- Collection of expansive soil and sands had been done.
- Conducting laboratory tests in order to characterize the expansive soil and both types of sands. Moreover, after blending of the sands with the expansive

soils, test that help in classification, in knowing the reduction of swelling and the improvement in strength of the blended soil had been carried out.

The tests conducted in the laboratory include

- Grain size analysis
- Atterberg limit test
- Free swell test
- Moisture-Density relation test
- Swell-consolidation test
- Unconfined Compression Strength test
- California Bearing Ratio test

- Finally, economic analysis had been done in order to evaluate the blending of sand and expansive soil over the conventional way of construction method.

1.5 ORGANAZTION OF THE THESIS

This thesis contains seven chapters. The first Chapter includes introduction, background of the problem, objective and methodology. The second Chapter deals with literature review. In this Chapter, related works to the thesis topic are covered. Laboratory test results and discussion are compiled in the third Chapter. The fourth Chapter covers the cost comparison. Conclusions and Recommendations are presented in Chapter five. Reference materials which are used in this thesis are mentioned in Chapter six.

2. LITERATURE REVIEW

2.1. DEFINITION AND PROPERTIES OF EXPANSIVE SOIL

Clay minerals are known to have a size of 0.002mm or less by most classification systems. However, the most important grain property of fine-grained is their mineralogical composition. Among their mineralogy, montmorillonite, illite and kaolinite are three groups. Montmorillonite is the clay mineral that abundantly present in expansive soil. It is this mineral which is responsible for the swell and shrink character of expansive soil.

At wet state, expansive soil swells. The magnitude of expansion depends on the type and the amount of clay mineral composition, their exchangeable ions, electrolyte content of aqueous phase. When expansive soil is at dry state, it shrinks and this swell –shrink cycle will highly affect structures constructed on it.

The parent of expansive soil is either igeneous rock or sedimentary rock. Basalt, dolerlite sills and dykes, gabros and norites are the basic parent igneous rock of expansive soil. Among sedimentary rocks, shales, marles and limestones are the basic parent materials of expansive soils. [1].

Identification and classification of expansive soil is mainly done by mineralogical classification and index tests. For mineralogical identification, X-ray diffraction, Differential Thermal Analysis and Electron microscope resolution. On the other hand, index tests consist of grain size analysis, consistency test, free swell and vertical swell in consolidometer. Based on index properties, different classifications systems are widely used. The classifications systems are described in Table 2.1.1, 2.1.2, Figure 2.1.1 and 2.1.2.

Clay (size)	Silt (size)			Sand			Gravel
	F	M	C	F	M	C	

0.002 (2μ)   0.006   0.02   0.06   0.2   0.6   2.0mm

<u>Legend</u>
F=Fine   M=medium

C=coarse

(a) MIT System

Clay (size)	Silt (size)	Sand				Fine Gravel	Gravel
		Very fine	Fine	Medium	Coarse		

0.005   0.05   0.1   0.25   0.5   1.0   2.0

(b) U.S. Bureau of Soil Classification

Table 2.1.1 Different classification systems [5]

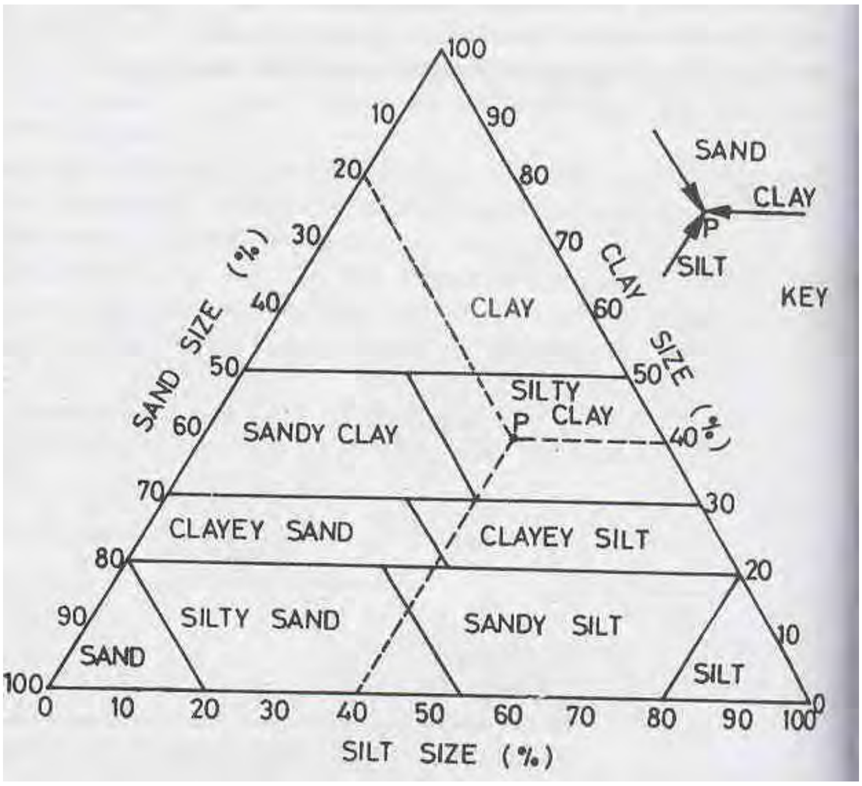


Fig 2.1.1 Modified Triangular Diagram [5]

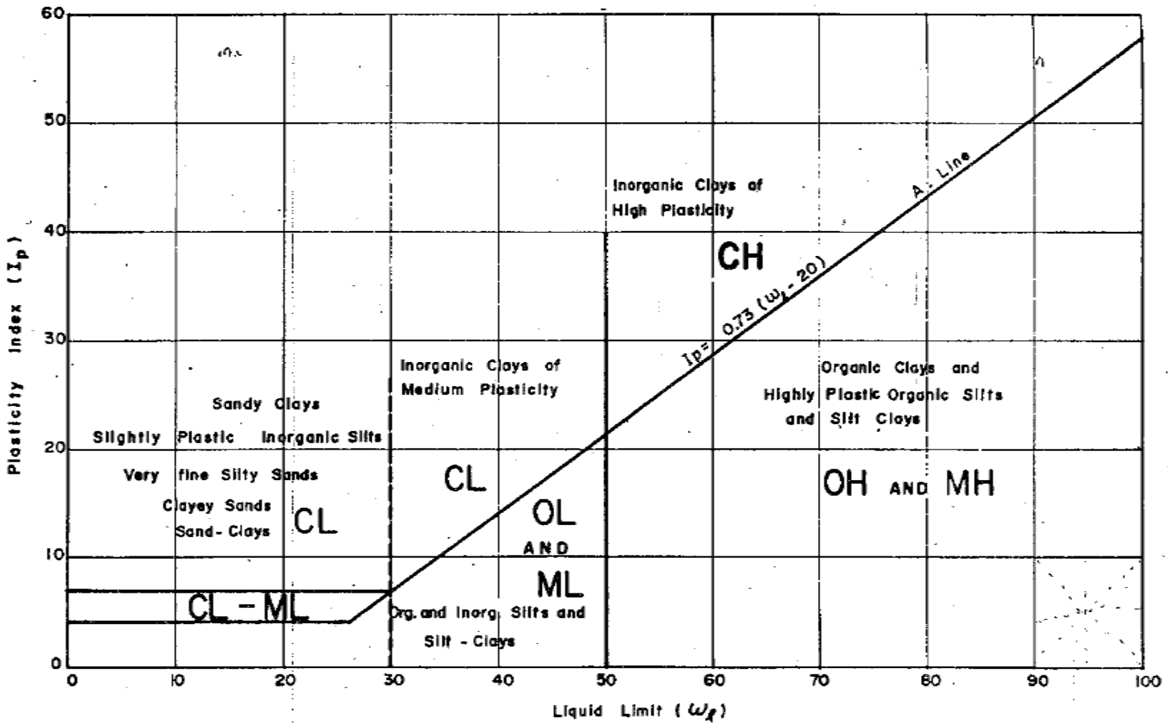


Fig 2.1.2 Plasticity chart [1]

General Classification	Granular Materials (35 per cent or less passing No. 200)							Silt-clay Materials (More than 35 percent passing No. 200)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7 A-7-5 A-7-6
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				
Sieve analysis per cent passing No. 10 No. 40 No. 200	50 max 30 max 15 max	50 max 10 max	51 min 10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40 sieve Liquid limit Plasticity Index	6 (max)		N.P	40max 10max	41 min 10max	40max 11 min	41 min 11min	40 max 10 max	41 min 10 max	40 max 11 min	41 min 11 min
Group index	0		0	0		4 max		8 max	12 max	16max	20 max
Usual types of significant constituent materials	Stone fragments gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as sub-grade	Excellent to good							Fair to poor			

The A-7 group is subdivided into A-7-5 or A-7-6 depending on the plastic limit. For P.L.<30, the classification is A-7-6; for P.L ≥30, it is A-7-5.

Table 2.1.2 AASHTO Classification System

2.2. Location of expansive soil in Addis Ababa

Expansive soils are found in many parts of the world. Ethiopia is one of the countries known to have expansive soils. Since many constructions are undergoing in the capital city of Ethiopia, Addis Ababa, expansive soils are being encountered. The distribution of expansive soil in Africa is shown in Figure 2.2.

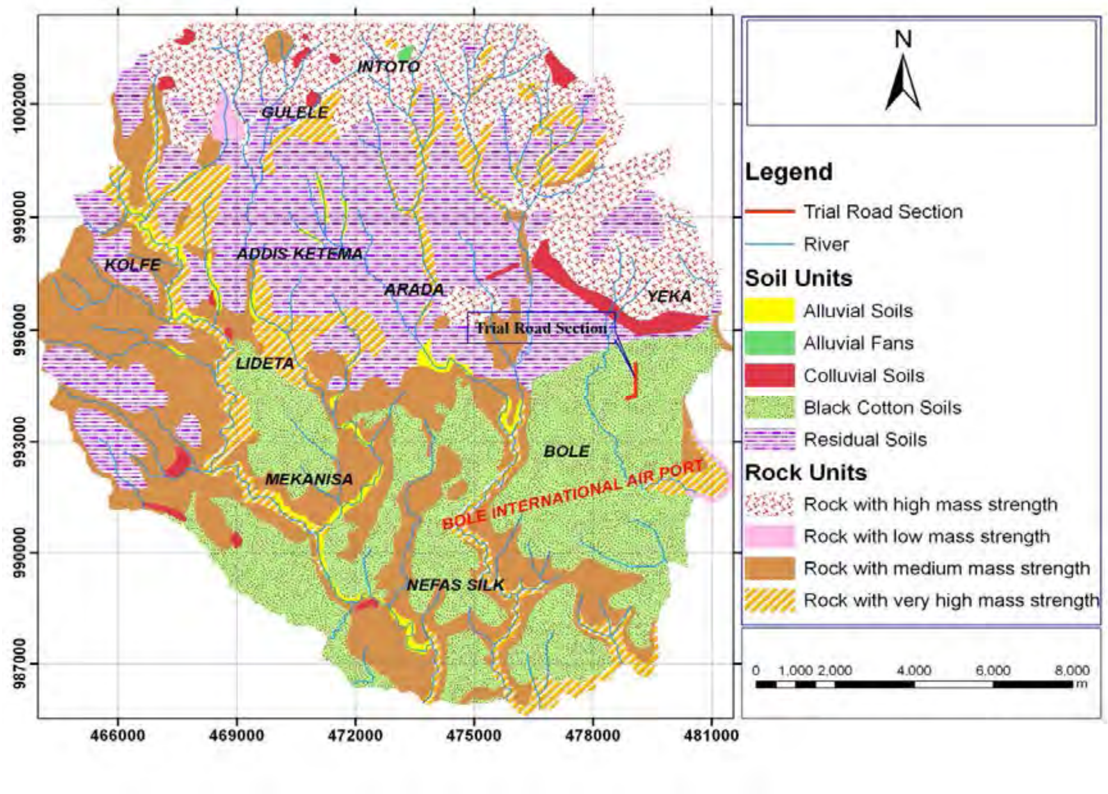


Figure 2.2 Engineering Geological Map of Addis Ababa [13]

2.3. Damage caused by expansive soil

Due to the swell and shrink characteristics of expansive soil when exposed to fluctuation of moisture, most structures built on it are highly damaged to extent of hindering to give their intended service. A study by [3] on ninety six buildings in different localities of Addis Ababa such as Bole, Olympia, Nifas silk Lafto, Old-airport, Mekanisa, Gerji and Bolebulebula showed that sixty nine of them are damaged. Lack of proper drainage systems and hence percolation of surface water to the foundation which leads to disturbance of soil equilibrium moisture contributes mostly to the damage caused. This factor contributes to 84% of the cause for the damages. However, other factors like leaking of pipes and new adjacent construction contribute to 16% of the causes for the damages. Figure 2.3.1, 2.3.2 and 2.3.3 show some of the failures observed in Addis Ababa.

Figure 2.3.1 Detachment of Tie beam and column in Bole senior secondary school [3]

Figure 2.3.2 Failure of fence [2]

Figure 2.3.3 Crack developed on wall [2]

2.4 Methods of soil stabilization

According to [4], different techniques are practiced in order to improve the properties of expansive soils. Some of them are discussed as follows.

1. Pre-wetting

   This technique is based on the assumption that the soil volume change can be constant if the soil is pre-wetted before construction of foundation. Hence, structural damage can be stopped. Pre-wetting involves direct flooding or ponding of the building area. The foundation and floor area is flooded by constructing a small berm around the outside of the foundation trenches to impound the water. However, this method is time consuming and due to the excess moisture, moisture migration to underlying soil can result.

2. Compaction control

   Expansive soil expands highly when compacted at higher density and low moisture. On the other hand, expansive soil expands less when compacted at lower density and higher moisture. The major advantage of this method is that swelling pressure of the soil can be reduced without the effect caused by the excess water introduced in to the soil. Migration of moisture into the underlying soil and elongated time prior to construction can be eliminated through compaction control. Nevertheless, achieving the required density can be difficult on site.

3. Soil replacement

   This method is a simple one. It mainly depends on the type of replacement material, depth of replacement and extent of replacement. The cost is relatively low when compared to chemical stabilization.

4. Chemical stabilizations

   Through different chemical stabilization technique, swelling pressure can be reduced and strength can be enhanced. Among the chemical stabilizations, lime, cement, asphalt and fly ash are known and practiced so far. Even though, most of them result in reducing swelling pressure of the expansive soil, their cost when purchasing and applying on the expansive soil is higher.

5. Mechanical stabilizations

   Mechanical stabilization is the process of improving the properties of soil by changing its gradation. [5]. It is also the simplest stabilization method. In order to achieve the proper grading, soils with coarse particles are mixed or the soils with fine particles are removed. Therefore, soils are divided into two groups.

      a) Aggregates : these are soils which have granular bearing skeleton and having particles of the size larger than 75µ
      b) Binders: These are the soils which have particles smaller than 75µ size. They do not possess a bearing skeleton.

The aggregates consist of strong, well-graded, angular particles of sand and gravel particles which provide internal friction and incompressibility to the soil. The binder which is mostly silt and clay provides cohesion and imperviousness to the soil. The presence of binder should provide plasticity to an extent not exceeding in creating swelling. Therefore, the blended soil will have both internal friction and cohesion. Mechanical stability of blended soils depends on the following factors

a) Mechanical strength of the aggregate: Aggregates which have high strength will provide a mechanically stable mix.
b) Mineral composition: the minerals in the mixed soil should be weather resistance.
c) Gradation: as much as possible the blended soil should be a well graded resulting in higher density.
d) Compaction: The mechanical stability of the mixed soil depends on the degree of compaction attained on site.

2.5 Properties of Natural and Crushed Sand

Sand is a naturally occurring granular material composed of finely divided rock and mineral particles. It has particles size finer than gravel and coarser than silt. The most common constituent of sand is silica (silicon dioxide, SiO_2) usually in the form of quartz. Because of its chemical inertness and considerable hardness, it is the most common mineral resistant to weathering. The other most common type of sand is calcium carbonate which has been created over the past half billion years by various forms of life like coral and shellfish in addition to organic and organically derived fragmental material. On the other hand, some sand contains magnetite, chlorite, glauconitic or gypsum.

AASHTO [1953] set the minimum sand size at 0.074mm and ISO 14688 grades sands as fine, medium and coarse with ranges 0.063mm to 0.2mm to 0.63mm to 2.0mm.

Crushed/manufactured sand is aggregate material having dimension less than 5mm that are processed from crushed rock or gravel. The particles of manufactured sands are cubic, angular and their surface texture is rough. The particle size distribution curve for crushed sand is high in proportion of fines as opposed to natural sand. [6]. Table 2.5 shows the difference between natural sand and crushed/manufactured sand.

Table 2.5 Comparison between natural and crushed sand [6]

Natural sand	Manufactured sand
• Has enough fines	• Has lots of fines
• Has smooth surface	• Provide stable grain distribution
• Surface is smooth and weathered	• Surface is rough
• Rounded to sub angular in shape	• Particles are angular

2.5.1 Availability of sand in Ethiopia

The source of sand and gravel in all parts of Ethiopia are streambeds and lacustrine deposits. These deposits are derived from quartizo-feldspatic basement rocks, sandy marine sediments, reworked pyroclastic material and alluvial deposits. Large quantity of sand and gravel are found along the escarpments in the northern and central Ethiopia. Awash basin (located 70-120km southeast of Addis Ababa) is the major supply of sand for Addis Ababa city. Rocks which are used in construction of buildings and roads are located within or close to rift valley [12]

2.6 Previous works

A number of researches have been carried out worldwide so far in order to improve the properties of expansive soil by using natural sand. Some of them are pointed out here.

A study in Algeria had been performed to understand the physical mechanism of stabilization of an expansive soil by adding sand at various forms: mixing and intercalation layer of sand. [8]. The aim of this study was to analyze the effect of sand on the soil consistency and on reduction of swelling pressure of the expansive soil. The materials used in this test had been artificial soil clay known as bentonite and beach sand having particle size of 0.1mm up to 2.0mm.

By varying the sand content from 10% up to 70% with an increment of 10% sand mixed with the bentonite, its effect on the consistency and swelling pressure of the mix had been checked.

The final results showed that as the percentage of sand increases the values of liquid and plastic limit had been reduced. This is due to the decrease in the content of fines that contribute to plasticity. An influence of the granularity of sand was also observed. It is due to the total specific area of water adsorption of the sample which depends on the grain size of the sand. When the sand is finer, it will allow the mixture to absorb more water resulting in relatively higher plasticity. Their findings in the variation of liquid limit and plastic index for different percentage of sand are shown in Figure 2.6.1 and 2.6.2 respectively.

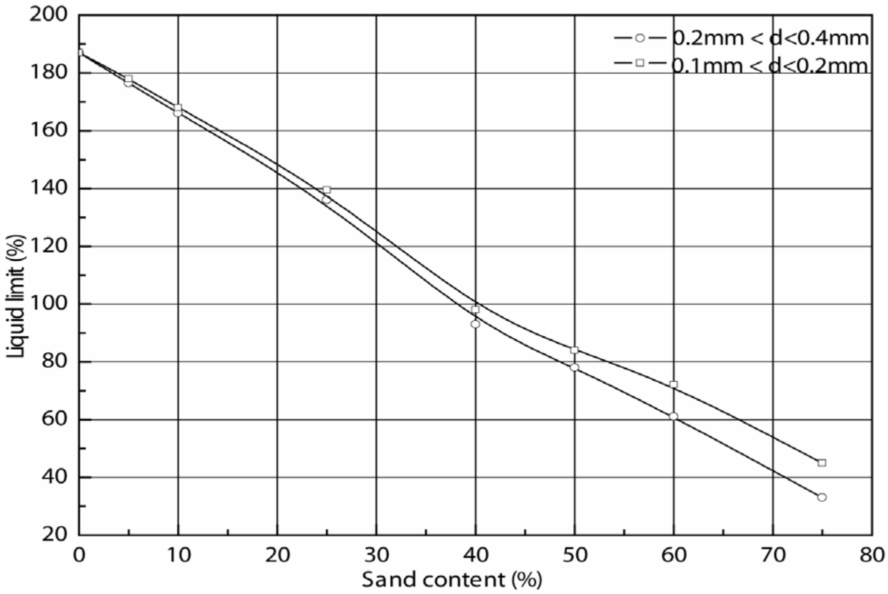


Figure 2.6.1 Variation of liquid limit **[8]**

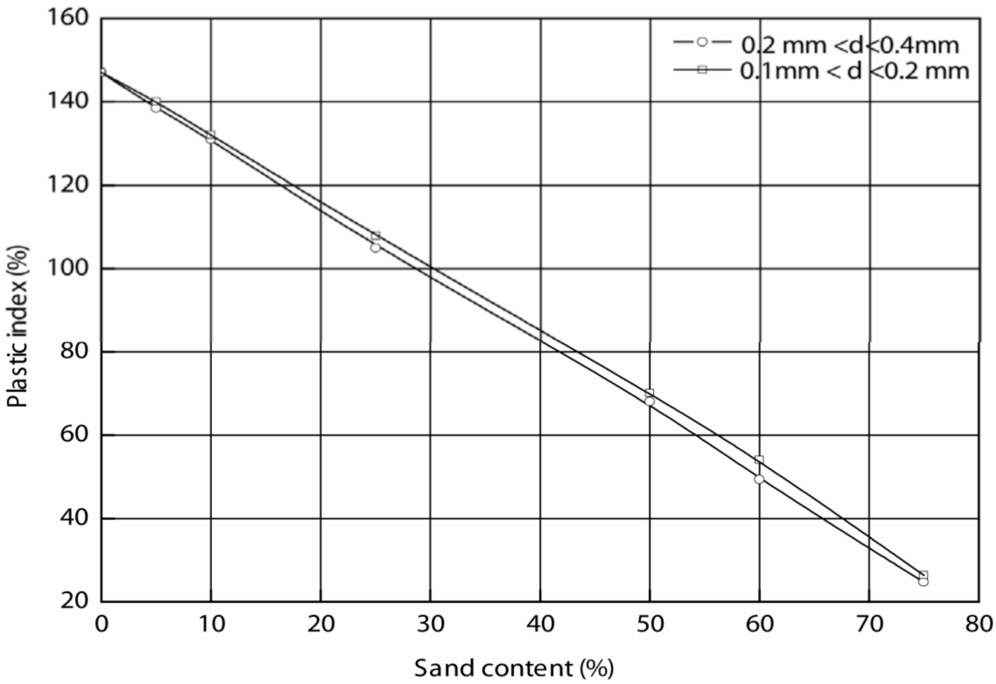


Figure 2.6.2 Variation of plastic index [8]

The study also revealed that as the addition of sand increases the swelling potential as well as swelling pressure decrease. This is due to the reduction of clay content in the mix. Additionally, much of the swelling is absorbed by the voids in the sand grains. When the sand particles are coarser the voids will be larger resulting in the decrease of swelling potential and pressure. The outcomes of their test results with regard to variations of the swelling potential and swelling pressure with sand content are presented in Figure 2.6.3 and 2.6.4 respectively.

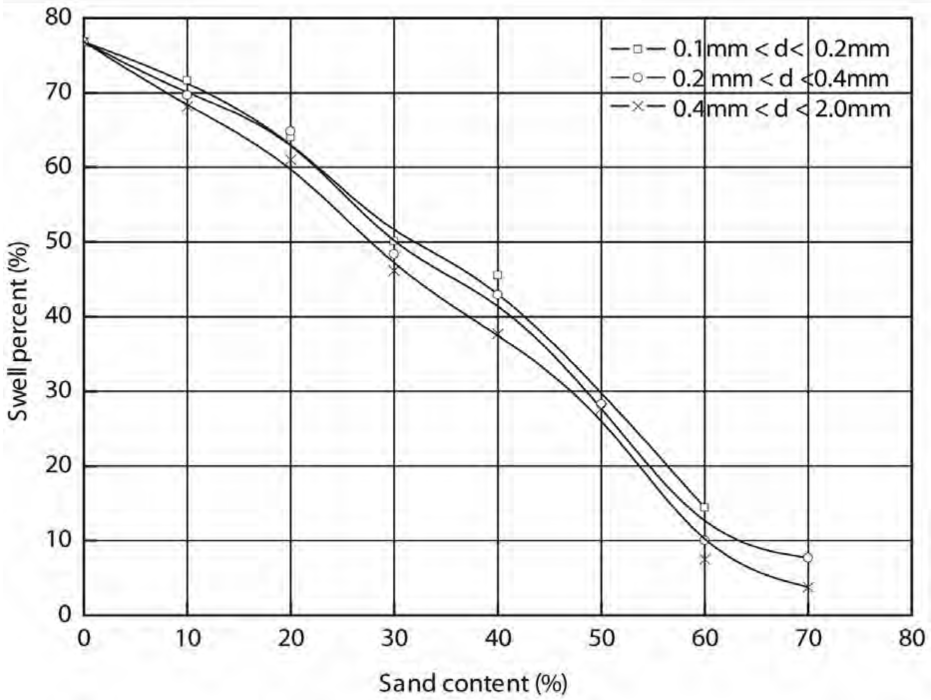


Figure 2.6.3 Variation of swelling potential with sand content [8]

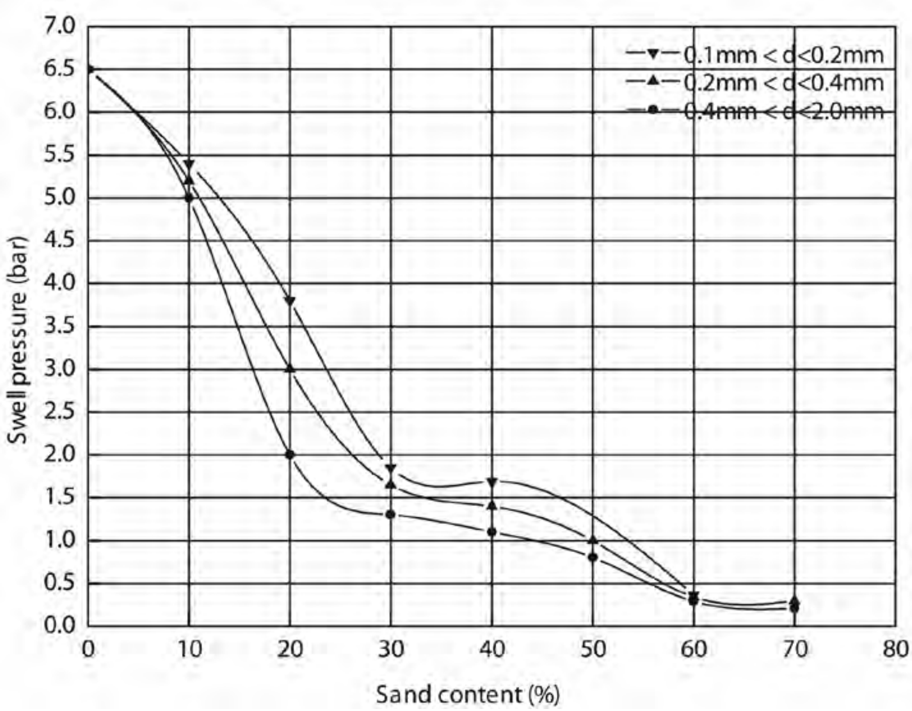


Figure 2.6.4 Variation of swelling pressure with sand content [8]

Another study was carried out in India to improve clay soil property by blending with sand and fly ash. [9]. The materials used had the following properties (Table 2.6.3)

Table 2.6.3 Physical properties of material used[9]

Property	Clay	Sand	Fly ash
Specific gravity	2.627	2.637	1.947
Maximum dry density, MDD (g/cc)	1.910	1.592	1.159
Optimum moisture content, OMC (%)	12.6	7.3	31.8
Liquid limit (%)	43.6	-	41.6
Plastic limit (%)	23.4	-	-
Plasticity index (%)	20.2	-	-
Uniformity coefficient, Cu	-	1.73	-
Coefficient of curvature, Cc	-	1.02	-
Soaked CBR (%)	2.47	9.17	2.04

Both the sand and fly ash were uniformly graded. Moreover, the fly ash had higher range of finer particles and the sand was poorly graded. Standard proctor tests were conducted on sample of clay, clay-sand and clay-sand-fly ash mix. The clay-sand mix were prepared from 10%-40% with an increment of 10%. After selecting suitable proportion of clay-sand, fly ash were mixed from 10%-25% with an increment of 5%. Soaked and un-soaked CBR were conducted on the sample. From the tests conducted, as addition of sand increases, the OMC decreased and the MDD increased. This was explained by the fact that the sand particles were coarser and had got less surface area which required lesser moisture to be compacted. The dry density increased due to the mixture being well-graded and better packing of clay and sand particle. After conducting soaked and un-soaked CBR, the CBR value increased for the stabilized soil. The value of soaked CBR varied from 2.47% for un-stabilized soil to 4.56% for stabilized soil. Un-soaked CBR varied from 5.59% for un-stabilized soil to 7.61% for stabilized soil. The improvement in CBR value is due to the better compaction achieved. The expansion ratio of the stabilized clay decreased from 8% to 3.2% which is 60% decrease in swelling was obtained.

A study at Addis Ababa was done to improve geotechnical properties of expansive soil by blending with red clay soil.[10]. The main purpose of this study was to improve negative properties of expansive soil so that to use it as a construction material in floors, pavement and embankment construction. Ten disturbed expansive soil samples and six undisturbed expansive soil sample were collected. Eight disturbed red clay samples were collected from different parts of Addis Ababa. The laboratory test conducted to attain the objectives of the study were grain size distribution, specific gravity, Atterberg limits, Proctor test, swelling potential and swelling pressure test. Table 2.6.4 & 2.6.5 show the laboratory test result for the blended soil.

Table 2.6.4 Laboratory test result for blended soil [10]

Red clay (%)	Specific gravity	At optimum			LL	PL	PI	SL	Clay fraction	
		Moisture content (%)	Wet density (gm/cc)	Dry Density (gm/cc)					‹2μ	≤1μ
0	2.71	30.02	1.73	1.33	105	27	78	7	67	62
10	2.70	34.84	1.79	»	92	»	65	10	56	42
20	»	30.13	1.73	»	80	»	53	16	53	45
30	»	33.50	1.80	1.35	77	25	52	14	53	47
40	»	30.39	1.82	1.39	71	26	45	17	52	42
50	»	29.12	1.84	1.42	66	29	37	18	52	47

Table 2.6.5 Laboratory test result for blended soil (continued) [10]

Red clay (%)	Activity		Free swell (%)	Volume swell (%)	Swelling pressure (kN/m^2)
	A′	A^2			
0	1.16	1.26	105	18.4	592
10	1.16	1.27	100	14.8	451
20	1.00	1.10	80	12.0	398
30	0.98	1.00	65	9.8	282
40	0.86	0.96	50	8.0	220
50	0.71	0.88	40	6.9	168

Note: $A' = I_p/C$, $A^2 = I_p/C\text{-}5$

After blending, the Atterberg limit test result showed that the blended soil is grouped from moderate to inactive. Moreover, the swelling pressure is reduced considerably.

3. LABORATORTY TEST RESULTS AND DISSCUSSION

In this chapter, laboratory test results are discussed. The expansive soil, natural and crushed sand were tested according to standard testing procedure of ASTM and AASHTO.

3.1 Material Used in the Test

3.1.1 Expansive soil

The expansive soil was collected from Nifas Silk- lafto Subcity in a locality known as Jemmo. The site is generally flat. Additionally, this area is one of the places where expansive soils exist.[3] The soil samples are collected from a depth of 3.0m below natural ground. The soil samples have black to dark grey color.(Figure 3.1.1). The amount of disturbed soil sample collected from the field is around 600kg.

Figure 3.1.1 Expansive soils from Jemmo area

3.1.1.1 Descriptive Test Results of the expansive soil

A total of nine various tests were performed on the sample of expansive soil. For classification of the soil, particle size distribution and gradation, Atterberg limit tests were conducted. Moisture- density relation was also determined. To know its strength, UCS and CBR were carried out. Swelling potential and swelling pressure of the soil were also determined. The gradation curve of the expansive soil is shown in Figure 3.1.1.1 The details of the different laboratory tests performed are presented in the Figure A-1,B-1,D-1,E-1,F-1,F-10. In addition, Table 3.1.1 shows the descriptive laboratory test results of the soil.

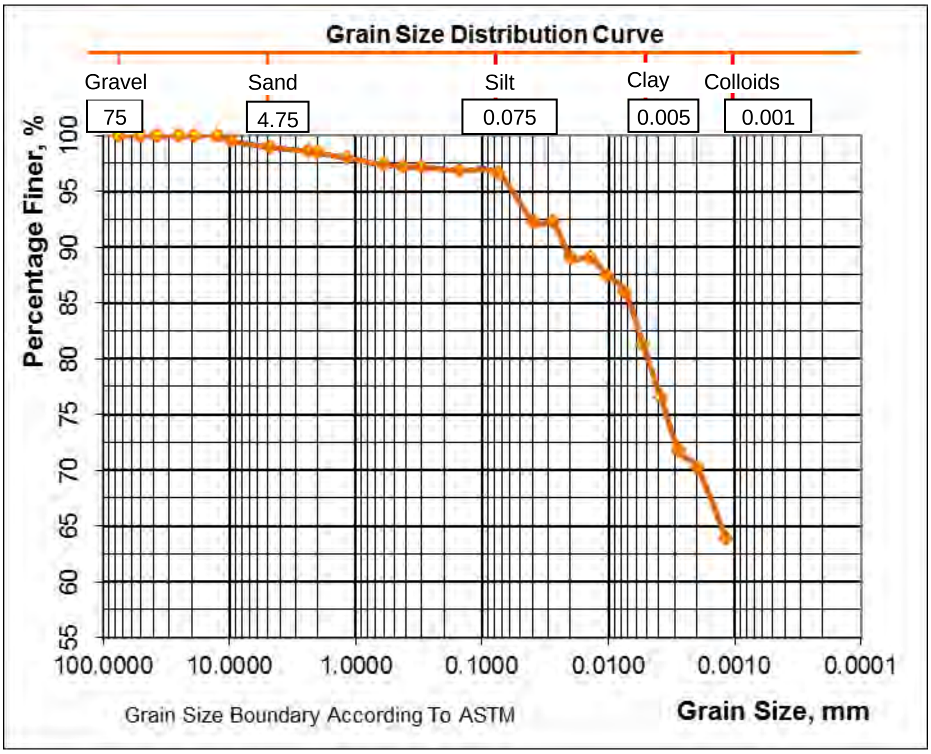


Figure 3.1.1.1 Grain size distribution curve of expansive soil

Table 3.1.1.1 Descriptive Test Results of the expansive soil

<table>
<tr><th colspan="2">Test conducted</th><th>Test result</th></tr>
<tr><td rowspan="5">Gradation, % passing</td><td>9.5mm</td><td>99.5</td></tr>
<tr><td>4.75mm</td><td>99.0</td></tr>
<tr><td>2.0mm</td><td>98.4</td></tr>
<tr><td>0.425mm</td><td>97.2</td></tr>
<tr><td>0.075mm</td><td>96.6</td></tr>
<tr><td rowspan="4">Atterberg limits,%</td><td>LL</td><td>91</td></tr>
<tr><td>PL</td><td>47</td></tr>
<tr><td>PI</td><td>45</td></tr>
<tr><td>SL</td><td>27</td></tr>
<tr><td colspan="2">Activity, A</td><td>0.51</td></tr>
<tr><td colspan="2">Specific gravity</td><td>2.73</td></tr>
<tr><td colspan="2">Free swell</td><td>150</td></tr>
<tr><td colspan="2">Natural moisture,%</td><td>60.6</td></tr>
<tr><td rowspan="2">Moisture-Density Relation</td><td>MDD,g/cc</td><td>1.24</td></tr>
<tr><td>OMC,%</td><td>35.4</td></tr>
<tr><td rowspan="2">USC</td><td>Unconfined compressive strength ,q_u (kPa)</td><td>142</td></tr>
<tr><td>Cohesion ,C (kPa)</td><td>71</td></tr>
<tr><td colspan="2">Swelling potential (%)</td><td>20</td></tr>
<tr><td colspan="2">Swelling pressure, kPa</td><td>367</td></tr>
<tr><td colspan="2">CBR (%)</td><td>1.0</td></tr>
</table>

3.1.1.2 Soil Classification

Using different classification systems mentioned in Table 2.1.1, the expansive soil is characterized. These classifications are presented in Table 3.1.1.2 and Figure 3.1.1.2

Table 3.1.1.2 classification of expansive soil

Type of classification	Expansive soil
USCS	CH
AASHTO	A-7-5
M.I.T	Gravel -1.4%
	Sand-2.0%
	Silt-22.6%
	Clay-72.65%

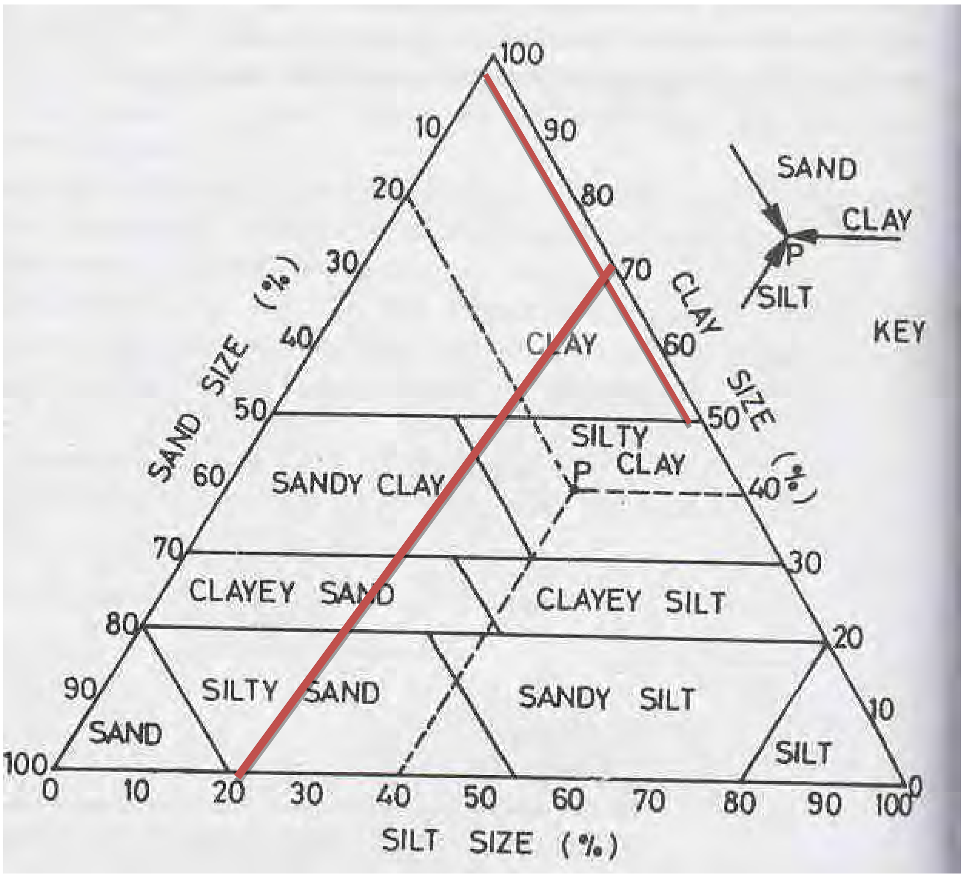
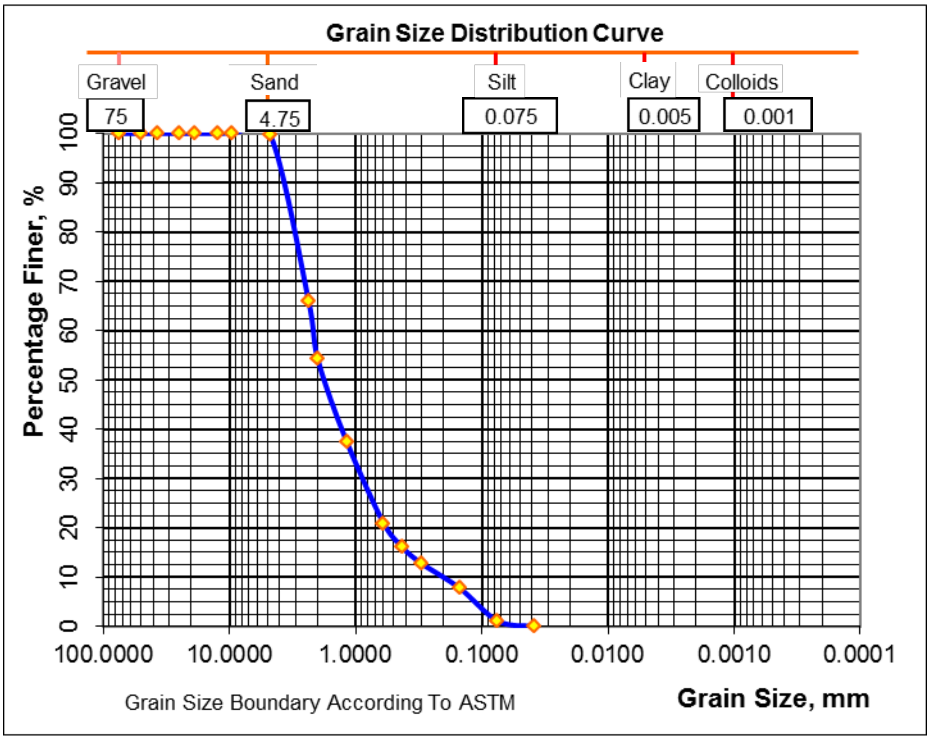


Figure 3.1.1.2 Classification of the expansive Soil sample based on Modified Triangular Diagram

Therefore, from Figure3.1.1.2 the expansive soil falls under the category of Clay soil. Furthermore, free swell is an indicator of the expansiveness of a soil. Since the value of the free swell is above 100% (Table C-1), the soil is under the category of expansive soil. Though, according to Skempton, the activity of the soil which is related through the plasticity of the soil with the quantity of the clay-size particle (<2μm) indicates the soil is inactive clay, the swelling characteristics of the soil from the odometer and the free swell tests has showed high values.

3.1.2 Natural and Crushed Sand

The natural and crushed sands were collected from the available market in Addis Ababa. Basic properties of both sands were investigated by conducting tests at laboratory. Sieve analysis, unit weight and specific gravity of both types of sands were determined according to [12]. Figure 3.1.1a and 3.1.1b show grain size distribution of crushed sand and natural sand respectively. Table 3.1.2 shows the properties of both types of sands.



. Figure 3.1.2a Grain size distribution of crushed sand

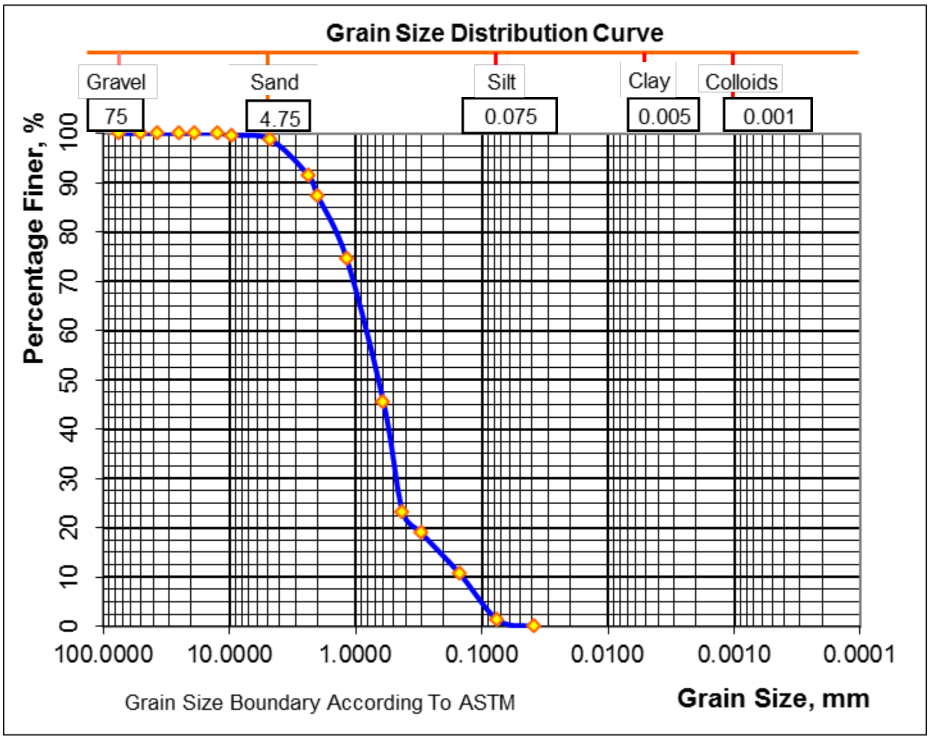


Figure 3.1.2b Grain size distribution of natural sand

Table 3.1.2 Properties of both types of sands

Test conducted		Test result	
		Natural Sand	Crushed Sand
Gradation, % passing	9.5mm	99.7	100
	4.75mm	99.0	99.7
	2.0mm	89.9	54.2
	0.425mm	38.6	16.0
	0.075mm	17.5	1.1
Unit weight (gm/cc)		1.3	1.68
Bulk Specific gravity (SSD)		2.52	2.64

3.2 Laboratory Result of Blended Soil

In this section, laboratory results of blended soil will be discussed for both the natural and crushed sand of various combinations.

3.2.1 Index Tests

Generally, the index tests are used to classify soils. Particle size distribution and gradation curve reveal the different proportion of particle sizes that comprise the soil. Therefore, it is a very important test in order to know and classify the soil type. The particle size distribution curve provides an index to the shear strength of the soil. Generally, well-graded compacted soil has high shear strength [5]. The gradation curves are presented in Figure 3.2.1.1a and 3.2.1.1b and summarized in Table 3.2.1.1a-3.2.1.1b. From the particle size distribution and gradation curves of various combination of both types sands with expansive soil ,it is observed that the curves moves from right to left which imply that the percent of particle size finer than a specific size is reduced. Therefore the particle size distributions of the blended soil are improved.

3.2.1.1 Particle size distribution & gradation

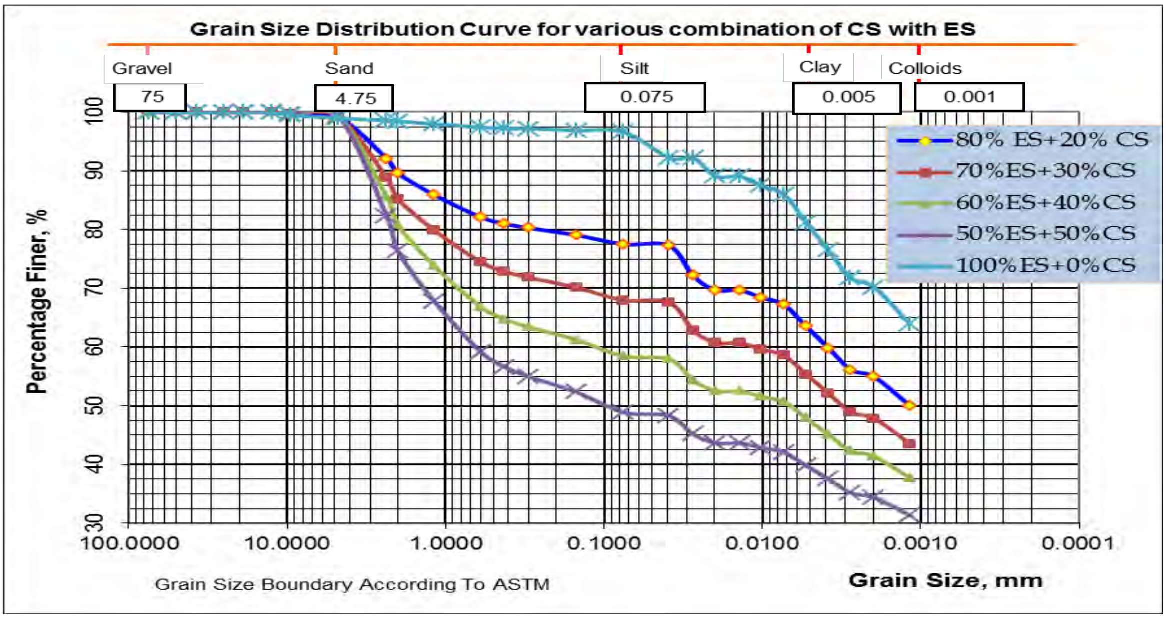


Figure 3.2.1.1a Grain size distribution curve of various proportion of CS with ES

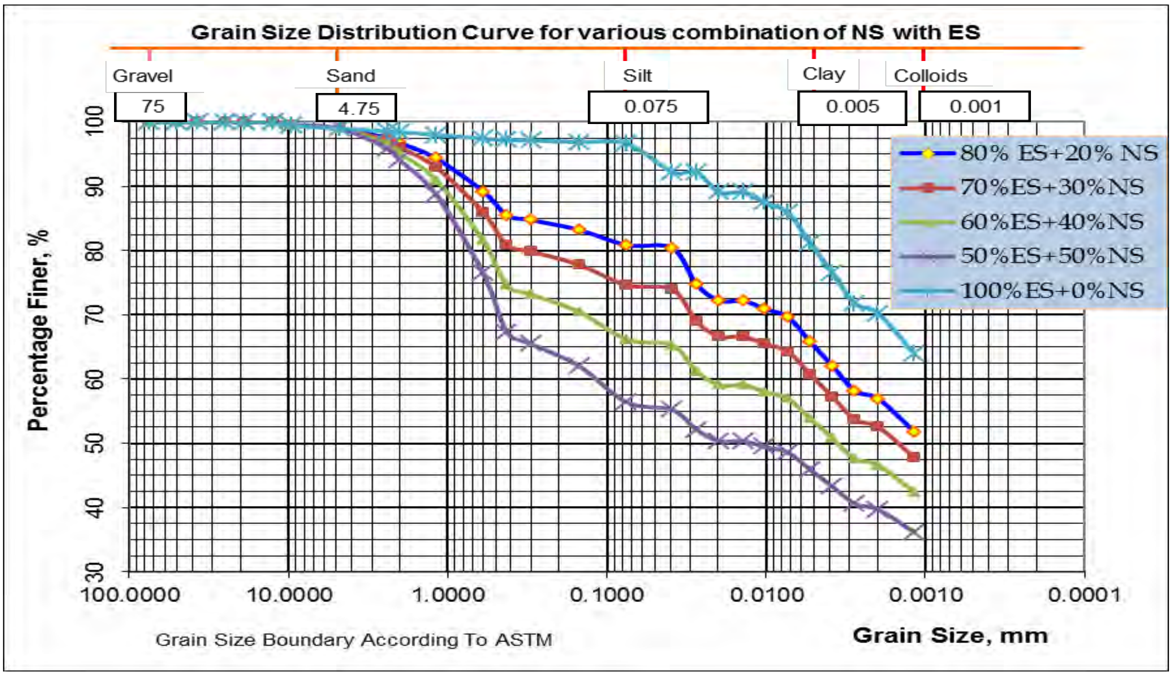


Figure 3.2.1.1b Grain size distribution curve of various proportion of NS with ES

Table 3.2.1.1a Gradation of various combination of CS with ES

Test conducted		Test result			
	Grain size	80%ES+ 20%CS	70%ES+ 30%CS	60%ES+ 40%CS	50%ES+ 50%CS
Gradation, % passing	9.5mm	99.6	99.6	99.7	99.7
	4.75mm	99.1	99.2	99.3	99.3
	2.0mm	89.6	85.2	80.8	76.3
	0.425mm	81.0	72.9	64.7	56.6
	0.075mm	77.5	68.0	58.4	48.9

Table 3.2.1.1b Gradation of various combinations of NS with ES

Test conducted		Test result			
	Grain size	80%ES+ 20%NS	70%ES+ 30%NS	60%ES+ 40%NS	50%ES+ 50%NS
Gradation, % passing	9.5mm	99.5	99.5	99.6	99.6
	4.75mm	99.0	99.0	99.0	99.0
	2.0mm	96.7	96.1	95.1	94.1
	0.425mm	85.5	80.9	74.6	67.3
	0.075mm	80.8	74.6	66.2	56.4

3.2.1.2 Atterberg Limit for various amount of crushed sand

For each percentage of crushed as well as natural sand, Atterberg limit tests have been conducted. As the percentage of crushed and natural sand blended increase, LL and PI are reduced. The SL increases as the percentage of added crushed and natural sand decrease. This is due to the fact that the clay fraction, which is responsible for the plasticity of the soil, is reduced by the addition of both the crushed and natural sand. The summary of the test results are presented in Figure 3.2.1.2a-3.2.1.2f and Table 3.2.1i&3.2j. The details of these test results are shown in Annex B.

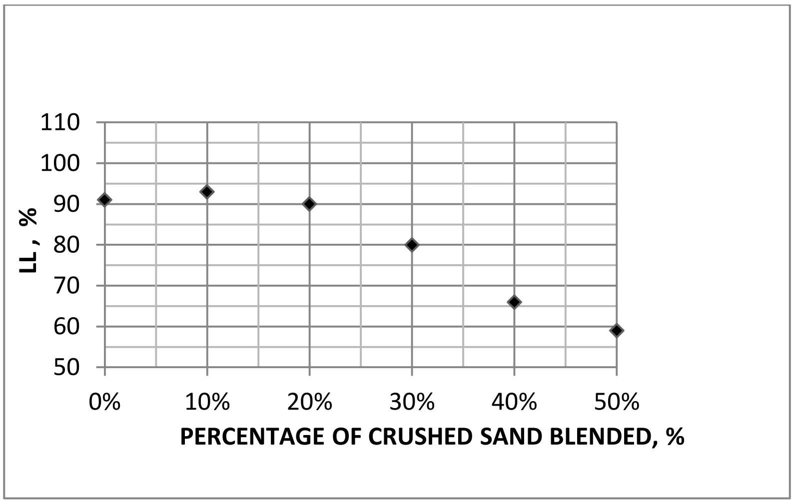


Figure 3.2.1.2a Variation of LL for different proportion of CS blended with ES

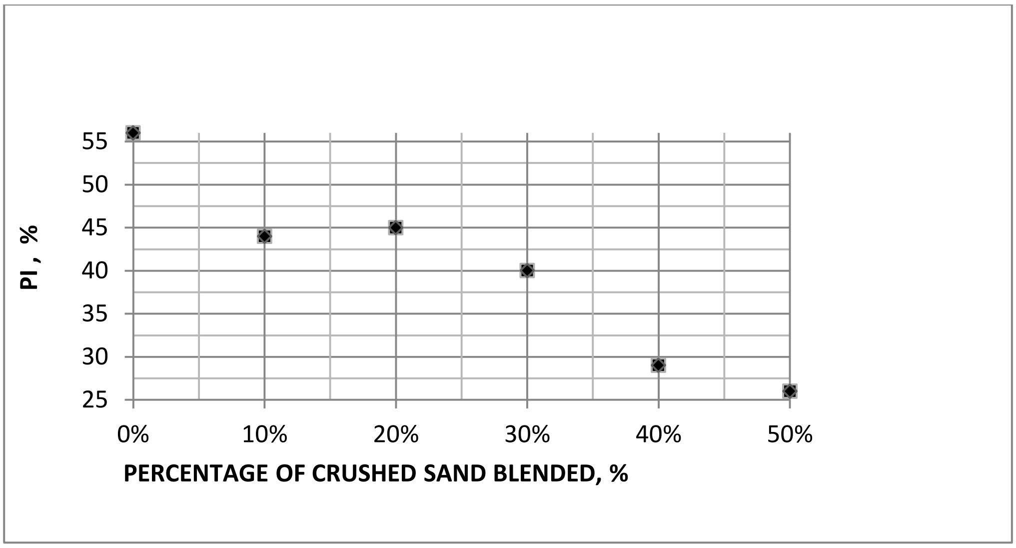


Figure 3.2.1.2b Variation of PI for different proportion of CS blended with ES

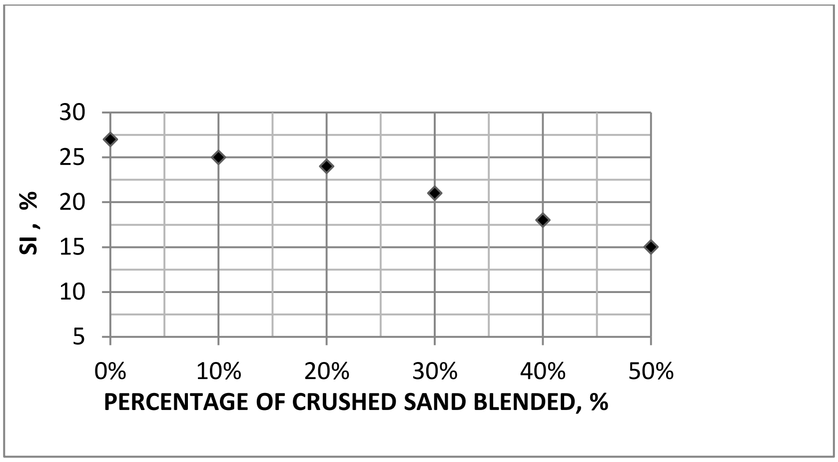


Figure 3.2.1.2c Variation of SL for different proportion of CS blended with ES

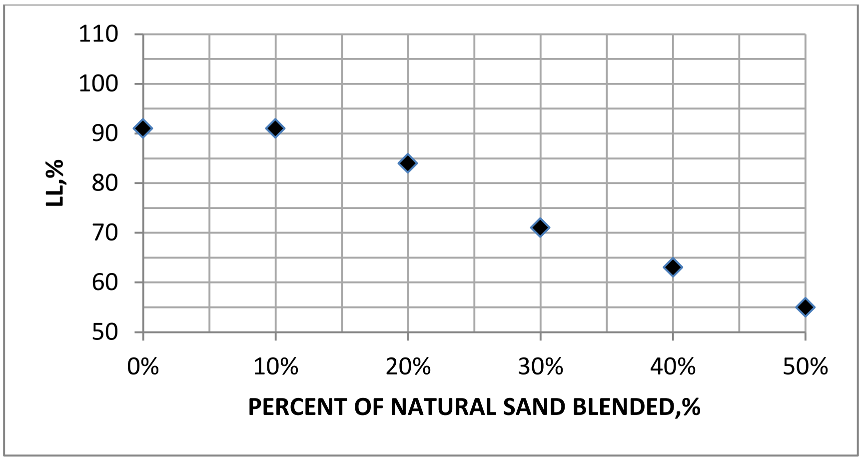


Figure 3.2.1.2d Variation of LL for different proportion of NS blended with ES

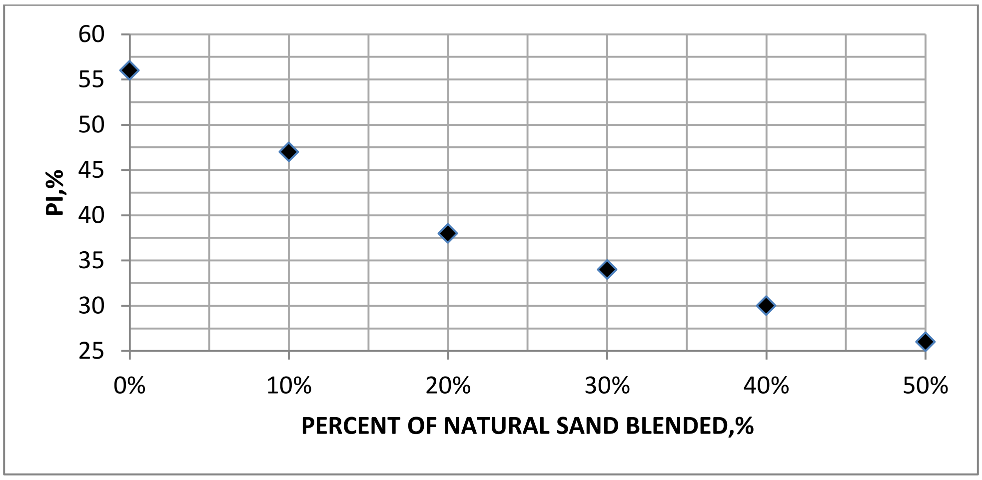


Figure 3.2.1.2e Variation of PI for different proportion of NS blended with ES

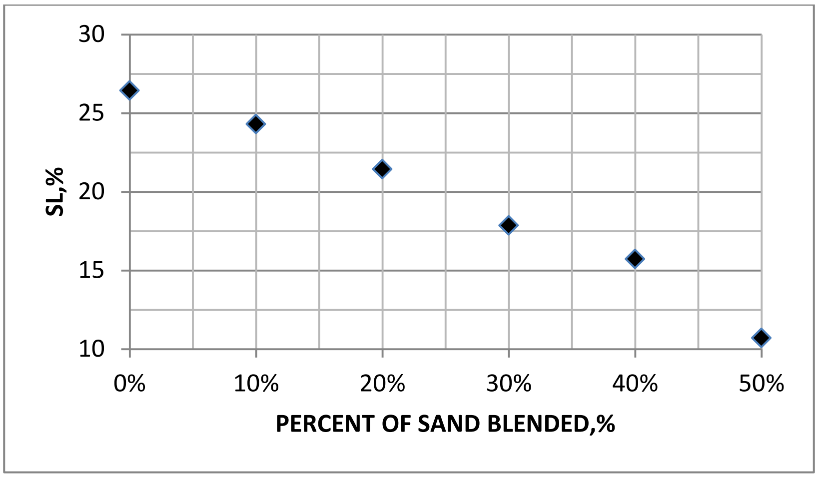


Figure 3.2.1.2f Variation of SL for different proportion of NS blended with ES

Table 3.2.1.1i Summary of Atterberg limits for varies CS Blended

Percent of CS Blended,%	LL	PL	PI
0	91	35	56
10	91	49	44
20	90	45	45
30	80	40	40
40	66	37	29
50	59	33	26

Table 3.2.1.1j Summary of Atterberg limits for varies NS Blended

Percent of NS Blended,%	LL	PL	PI
0	91	35	56
10	91	45	46
20	84	46	38
30	71	37	33
40	63	32	31
50	55	29	26

In conclusion, the amount of fine particles presented in the expansive soil , that are passing sieve No200, are decreased after blending with the crushed and natural sand. Therefore, the plasticity of soil is reduced. This phenomenon is observed in Table 3.2.1.1k and 3.2.1.1m. For instance, from these tables, for 40% crushed sand blended a 39.54% reduction in clay particles has shown. On the other hand, for the same percent of natural sand blended, 31.5% decrease in the amount of clay particles has been achieved. These changes in gradation will explain to most of the improvement achieved in other tests conducted.

Table 3.2.1.1k Summary of particle size passing No 200 for varies CS Blended

Percent of NS Blended,%	Percent Passing No 200,%
0	96.6
20	77.5
30	68.0
40	58.4
50	48.9

Table 3.2.1.1j Summary of particle size passing No 200 for varies NS Blended

Percent of NS Blended,%	Percent Passing No 200,%
0	96.6
20	80.8
30	74.6
40	66.2
50	56.4

Based on USCS classification system, all the combinations of the ES/NS & ES/CS has fallen under the group of MH. On the other hand, using AASHTO classification system all combination except 50%ES+50%NS remain under the category of A-7-5. However, 50%ES+50%NS is grouped A-7-6. This is due to the fact that more 35% of the particles are passing No200 and the materials are classified as silt-clay. The summary of classification systems for various mixture of expansive soil with crushed and natural sand are tabulated
in Table 3.2.1.1n and 3.2.1.1p respectively.

Table 3.2.1.1n Summary of classification for varies CS Blended

Type of classification	Percent of Blended Crushed Sand with Expansive soil				
	100%ES+ 0%CS	80%ES+ 20%CS	70%ES+ 30%CS	60%ES+ 40%CS	50%ES+ 50%CS
USCS	CH	MH	MH	MH	MH
AASHTO	A-7-5	A-7-5	A-7-5	A-7-5	A-7-5

Table 3.2.1.1p Summary of classification for varies NS Blended

Type of classification	Percent of Blended Natural Sand with Expansive soil				
	100%ES +0%NS	80%ES+2 0%NS	70%ES+ 30%NS	60%ES+4 0%NS	50%ES+ 50%NS
USCS	CH	MH	MH	MH	MH
AASHTO	A-7-5	A-7-5	A-7-5	A-7-5	A-7-6

3.2.2 Moisture-Density Relationship

Compaction of a soil is a means of improving the engineering properties of soil by mechanical method. When a soil is compacted, air in the voids will be expelled and the density of the soil mass increases. Compaction increases the shear strength of the soil and hence the stability and bearing capacity. [5]

In this section, the results of the summary of moisture density relationship for different percentage of crushed and natural sand blended with the expansive soil are shown in Figure 3.2.2a,b,c,d,e&d. The details of the Moisture-Density Relation for each percentage are displayed in Annex B.

As the percentage of both types of sand blended increases, the MDD as well increases and on the contrary, the OMC decreases. The more the sand increases, the more the particle size distribution and gradation improved and then a better packing between clay and sand particles will be attained. The clay particles in the soil act as binding material between in the coarser particles of the sand. Hence, the void space in the sand particles will decrease and the density of the soil mass increase. Additionally, coarser particles of the sand need less moisture content to achieve maximum dry density since their surface area are less than that of clay particles. Therefore, the OMC reduced as the percentage of sand blended increases. Tables 3.2.2a and 3.2.2b summarize the variations of MDD and OMC with different proportion of both types+ of sands respectively. Figures 3.2.2a to 3.2.2f also show the variation of MDD and OMC for different proportion of crushed and natural sand respectively.

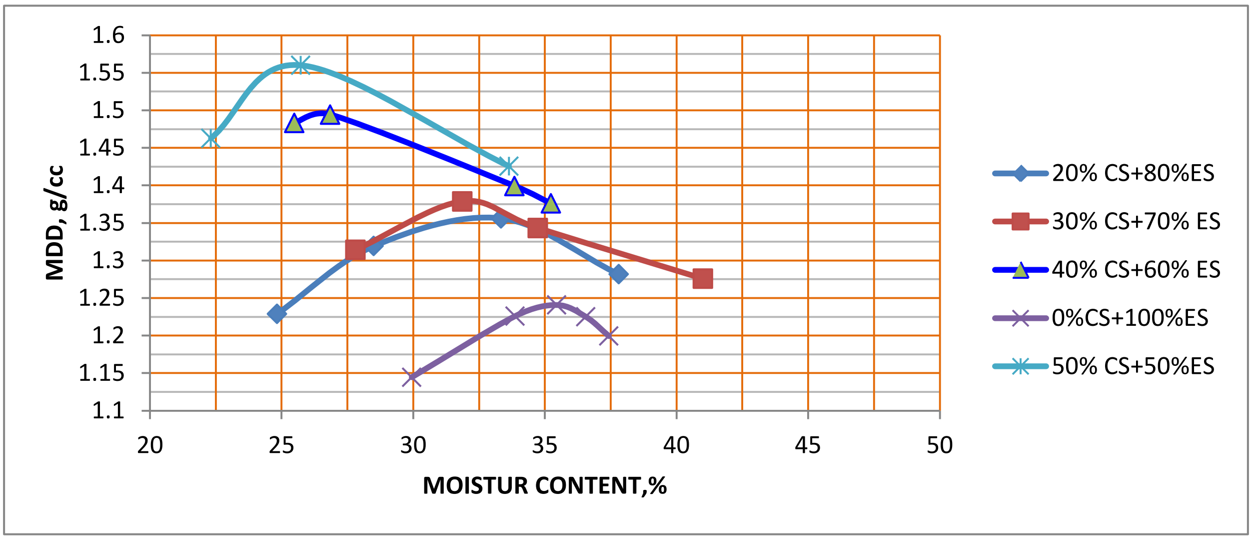


Figure 3.2.2a Moisture-Density relation for different proportion of CS blended with ES

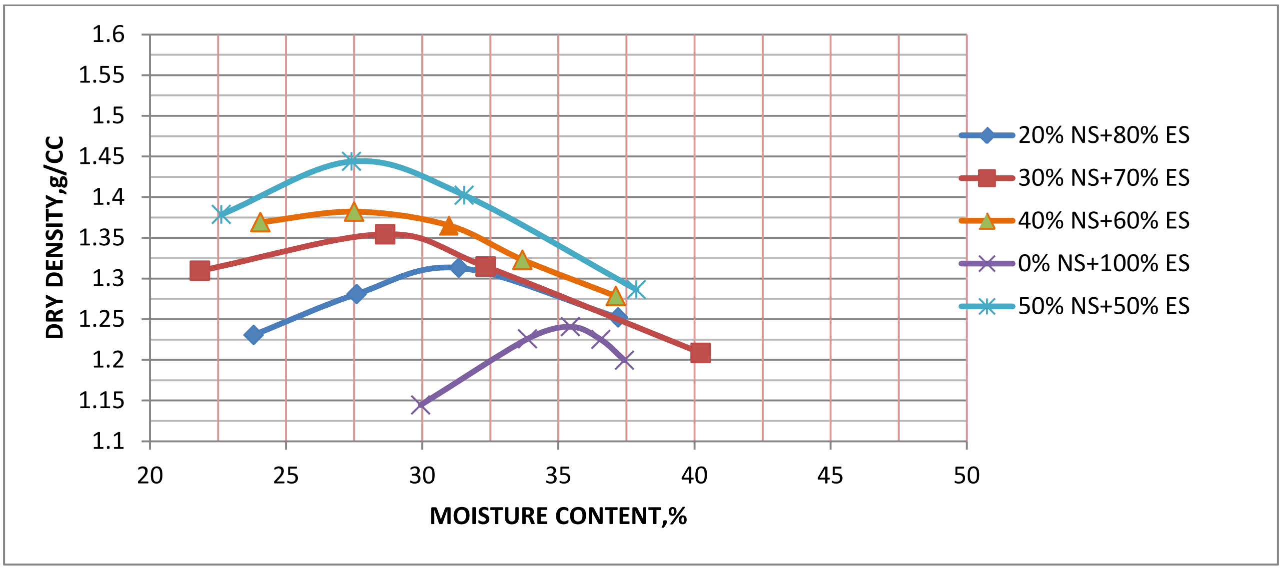


Figure 3.2.2b Moisture-Density relation for different proportion of NS blended with ES

3.2.3 Free swell

This test helps to know the expansiveness of a soil by measuring the volume of soil sample that is soaked & settled down in water for 24 hrs.

In this section, the Free Swell for each percentage of the both type of sands blended are presented in Figure 3.2.3a &b. From the test results, for the lower percentage of both type of sands blended, the value of the free swell do not show significant improvement. But for the higher percentage, it has showed some improvement. Therefore, the blended soil changed from expansive to marginal group. The details of the Free swell test for each percent of sand added is shown in Annex C.

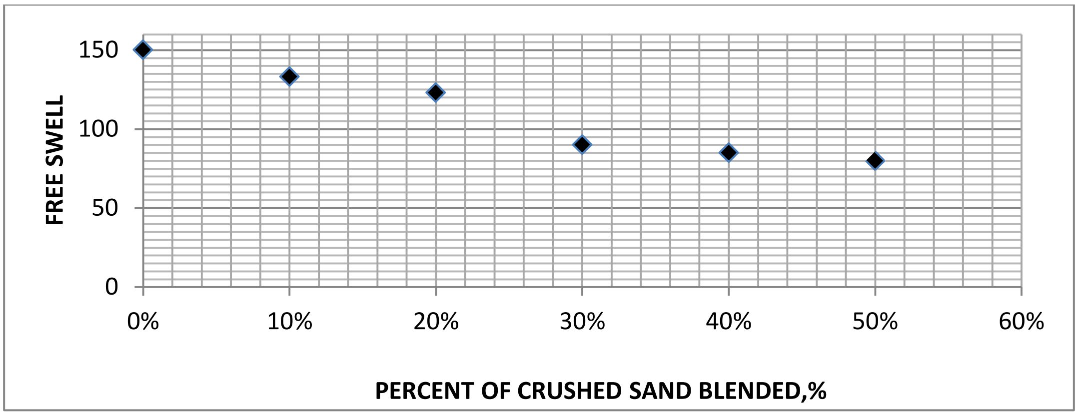


Figure 3.2.3a Variation of Free Swell for different percent of Crushed Sand Blended

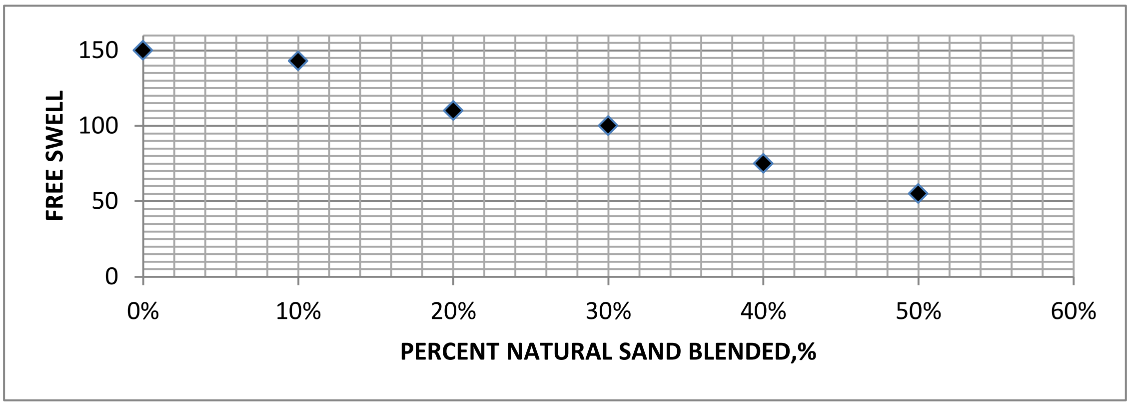


Figure 3.2.3b Variation of Free Swell for different percent of Natural Sand Blended

3.2.4 Swelling Potential and Pressure

Swelling potential is expressed as the percentage of swell of laterally confined sample which has been soaked under a surcharge of 1psi (7kN/m^2) in an odometer. The swelling pressure is defined as the vertical pressure required to prevent volume change of laterally confined sample when it gets water. The magnitude of swelling potential and pressure is governed by the amount and type of clay in the soil, placement condition that involve initial water content, initial density, confining pressure and time allowed to swell. [1] While conducting the swell-consolidation test, 7kN/m^2 was used as a surcharge and a reasonable amount of time (more than 72hr)has been allowed for the sample to swell.

From the results of the swelling test, considerable improvements have been observed by the stabilization of the two types of sands. This phenomenon is explained by the fact that the amount of clay, which is responsible for the swelling nature, is reduced by the addition of sand.

The summary of swelling potential and swelling pressure for the two types of sand are illustrated in Figure 3.2.4a,b,c&d. Details of the swelling potential & pressure are included in Annex D.

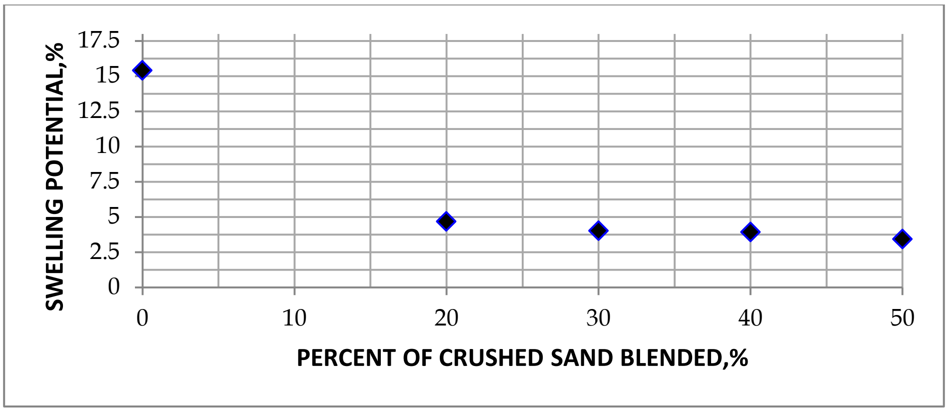


Figure 3.2.4a Variation of Swelling potential for different proportion of Crushed Sand Blended

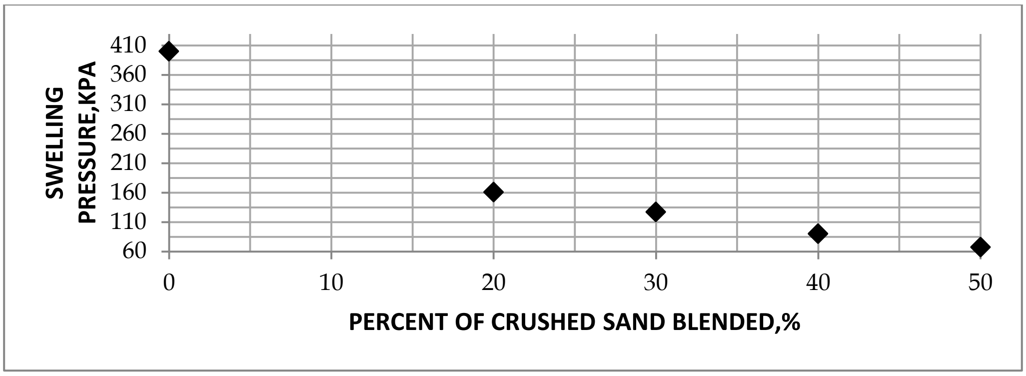


Figure 3.2.4b Variation of Swelling Pressure for different proportion of Crushed Sand Blended

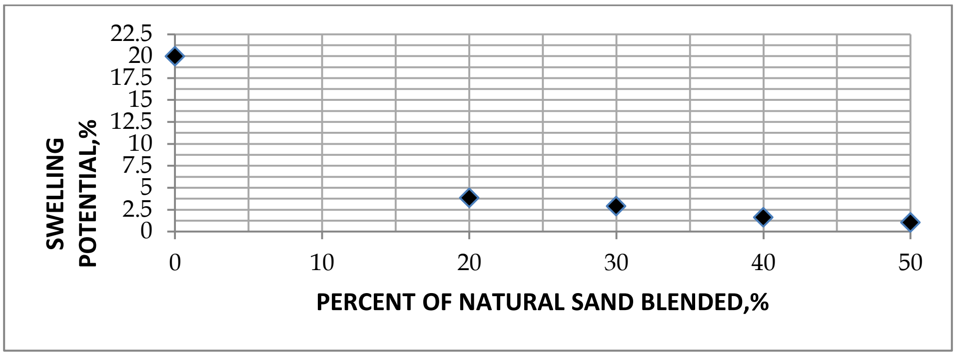


Figure 3.2.4c Variation of Swelling Potential for different proportion of Natural Sand Blended

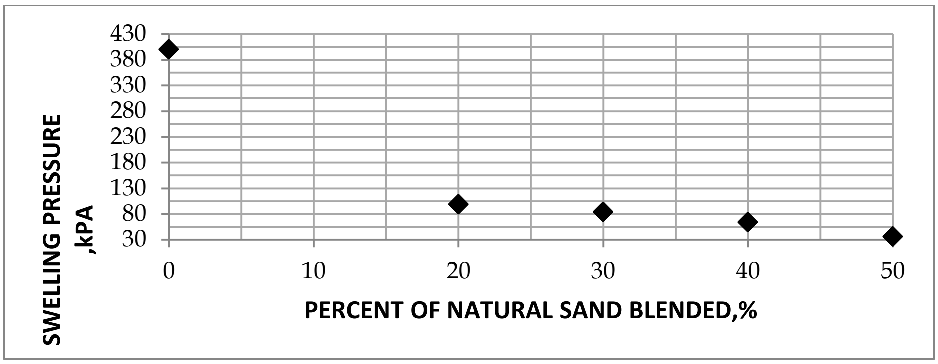


Figure 3.2.4d Variation of Swelling Pressure for different proportion of Natural Sand Blended.

3.2.5 Unconfined Compression Test

In this test a soil sample is subjected to axial load without a confining pressure. It is a special type of triaxial compression test in which σ_3 is zero. This test is simple and quick. As the percentage of both type of sand increases, the unconfined compressive strength also increases. Each sample (NS/ES and CS/ES) are prepared at their respective MDD and OMC for unconfined compression tests. As the percent of sands blended with the expansive soil increased, the OMC decreased. Hence, the force of attraction between the clay particles increases. Therefore, cohesion increases. Ultimately, the unconfined compressive strength that is twice the cohesion is also doubled. The increment in unconfined compressive strength of the new mix is greater for higher percentage of sand blended. Figure 3.2.5a and 3.2.5b show the variation of q_u and c_u for different proportion of both type of sands blended. The detail of the analysis is presented in Annex E.

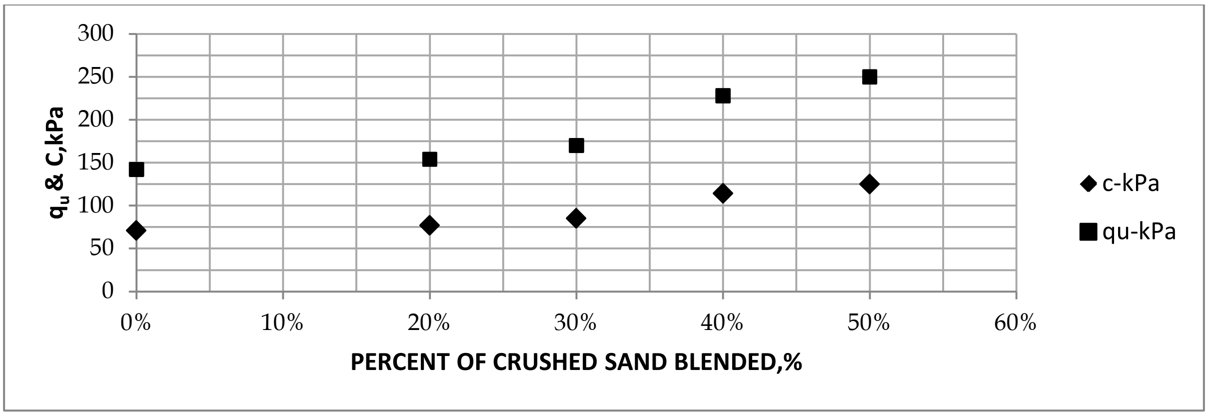


Figure 3.2.5a Variation of q_u and c_u for different proportion of CS Blended

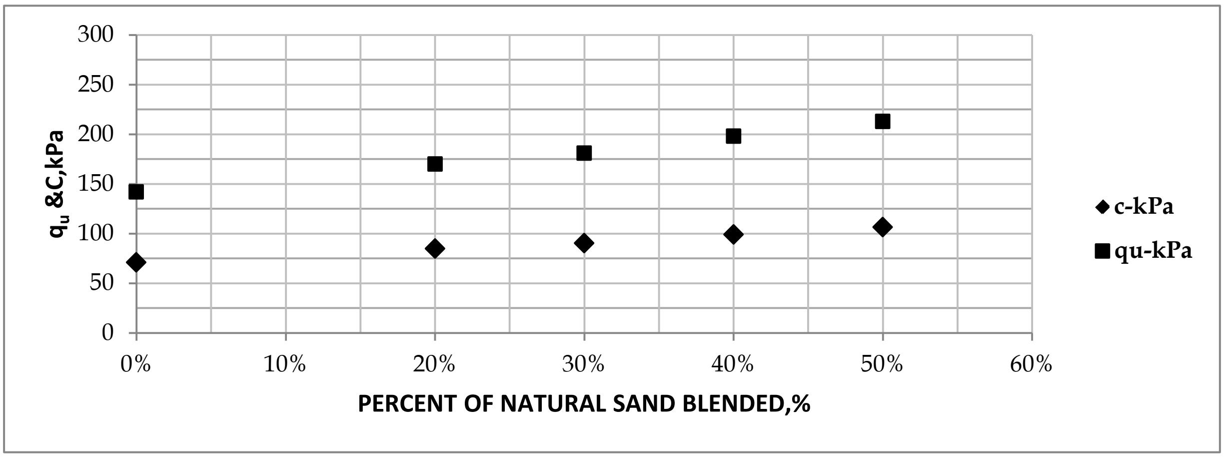


Figure 3.2.5b Variation of q_u and c_u for different proportion of NS Blended

3.2.6 California Bearing Ratio

CBR is used to rate the performance of a soil for use of sub-grade, sub-base and base courses. The CBR is obtained as the ratio of the unit stress required to effect a certain depth of penetration of the piston into the sample at some water content and density to the standard unit stress required to obtain the same depth of penetration on a standard sample of crushed stone. CBR test were carried out for varies combination of the sand and expansive soil for their respective optimum moisture content and maximum dry density after determining by standard compaction. One point CBR test were conduct on all samples and un-soaked CBR test were performed on selected combinations of both type of sands for the reason that those combinations are proposed for actual use in the field considering economy. In Figure 3.2.6a&b, the results of CBR is shown for the two types of sand respectively and the details of the CBR for each percent of both sands blended is included in Annex F.

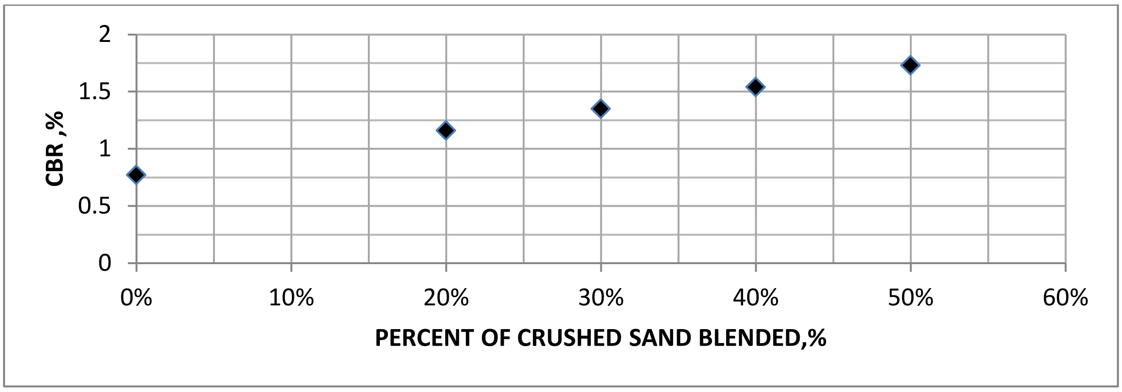


Figure 3.2.6a Variation of Soaked CBR for Different percent of CS Blended

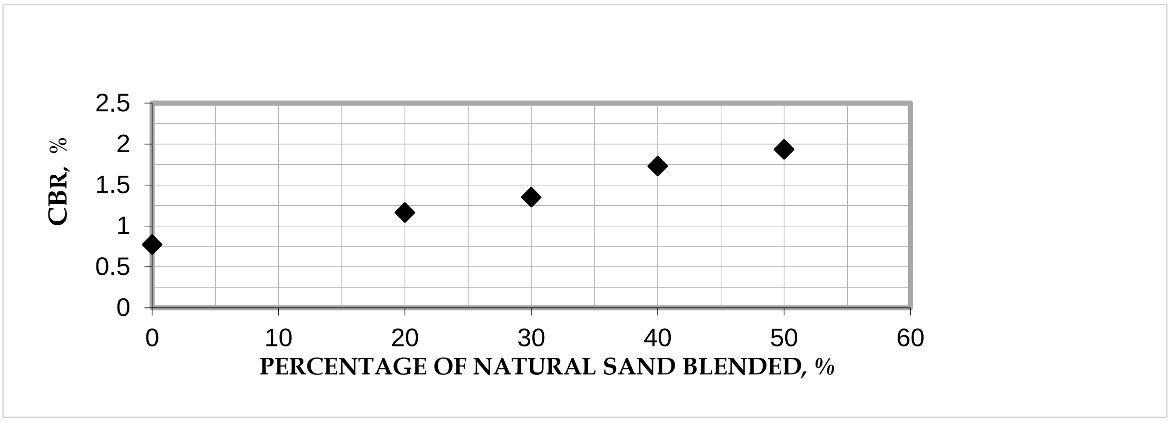


Figure 3.2.6b Variation of Soaked CBR for Different percent of NS Blended

3.2.7 Grand Summary

In this section the summary of all test performed are shown in Table 3.2.7a and 3.2.7be.

Table 3.2.7a Summary of all test performed on varies percent of CS Blended

Type of test		Percent of Blended Crushed Sand with Expansive soil				
		a	b	c	d	e
Sieve Analysis	USCS	CH	MH	MH	MH	MH
	AASHTO	A-7-5	A-7-5	A-7-5	A-7-5	A-7-5
Atterberg Limits	LL,%	91	90	80	66	59
	PL,%	35	45	40	37	33
	PI,%	56	45	40	29	26
	SL,%	27	24	21	18	15
Moisture-Density Relation	MDD,g/cc	1.24	1.36	1.38	1.49	1.56
	OMC,%	35.4	33.33	31.87	26.85	25.73
Free swell,%		150	123	90	85	80
Swell-consolidation	Swelling Potential,%	15	4.66	4.02	3.92	3.42
	Swelling Pressure,kPa	400	161	127	90	67
UCS,q_u,kPa		142	154	170	228	250
CBR,%(Soaked)		1.0	1.16	1.35	1.73	1.93
CBR,%(Unsoaked)		17.52	NP	NP	23.49	NP

Note a- 100%ES+0%CS

b- 80%ES+20%CS

c- 70%ES+30%CS

d- 60%ES+40%CS

e- 50%ES+50%CS

NP – Not performed

Table 3.2.7b Summary of all test performed on varies percent of NS Blended

Type of test		Percent of Blended Natural Sand with Expansive soil				
		f	g	h	i	j
Sieve Analysis	USCS	CH	MH	MH	MH	MH
	AASHTO	A-7-5	A-7-5	A-7-5	A-7-5	A-7-5
Atterberg Limits	LL,%	91	84	71	63	55
	PL,%	35	46	37	33	29
	PI,%	56	38	34	30	26
	SL,%	27	21	18	16	11
Moisture-Density Relation	MDD,g/cc	1.24	1.31	1.35	1.38	1.44
	OMC,%	35.4	31.35	28.65	27.5	27.4
Free swell,%		150	110	100	75	55
Swell-consolidation	Swelling Potential,%	15	3.85	2.9	1.64	1.06
	Swelling Pressure,kPa	400	99	84	64	56
UCS,q_u,kpa		142	170	181	198	213
CBR,%(Soaked)		1.0	1.16	1.35	1.54	1.73
CBR,%(Unsoaked)		17.52	NP	NP	32.35	NP

Note f- 100%ES+0%NS

g- 80%ES+20%NS

h- 70%ES+30%NS

i- 60%ES+40%NS

j- 50%ES+50%NS

NP – Not performed

3.3 Comparisons between the effects of both types of sands on the stabilization of expansive soil

As mentioned earlier, both types of sands have showed a positive effect on the stabilization of expansive soil. However, when their effects are further compared, both types of sands have different effect with regard to swelling and strength characteristics of the blended soil. For instance, the soil samples which are blended with the natural sand had showed a much lower values in consistency limits than crushed sand blended with the soil. Moreover, the swelling characteristics of the blended soils by using the natural sand had reduced significantly than that of crushed sand.

On the other hand, the value of the MDD is higher for the sample prepared by mixing using crushed sand. This is due to the fact that the unit weight of the crushed sand is higher than the natural sand. Additionally, lower values of OMC from the samples of crushed and expansive soil blended had been achieved when compared to the samples prepared using natural sand. This phenomenon revealed that the crushed sand requires lesser moisture to attain maximum dry density.

With regard to unconfined compression strength, the samples which were prepared by mixing different percent of crushed sand with expansive soil attained higher values than those samples prepared from natural sand. Since the OMC is lower for those samples, the cohesion will be higher and in turn resulted in a higher unconfined compressive strength than that of natural sand.

4. Cost comparison

The cost comparison was done for a typical G+1 residential building based on two options. The first option is the conventional construction method, that is replacement of the expansive soil by non-expansive soil and the second option is calculated based on substructure work constructed using stabilized soil of 40% sand by weight of the expansive soil.

During collection of materials, the costs of both types of sands are found to be more or less equivalent in the market of Addis Ababa. Therefore, the cost comparison was not done separately for the two types of sands. Table G-1 shows the cost of excavation and earth work for G+1 residential building by replacing with non – expansive soil and Table G-2 shows cost of excavation and earth work for G+1 residential building by using an expansive soil stabilized by 40% sand. Finally, the summary of the cost comparison among the two options is presented in Table G-4.

Assumptions

The following assumptions are used in the calculation of the costs.

i. The foundation depth is assumed to be 3.00m from the natural ground level.
ii. The foundation types used are assumed to be isolated footings.
iii. The method of blending the sand with the expansive soil on site is assumed to be hand mixing.

Sources used

i. consulting & construction companies
ii. constructionproxy.com

5. CONCLUSION AND RECOMMENDATION

5.1 CONCLUSION

Finally, the following conclusions are drawn.

- From the discussion of the test results, the engineering properties of the expansive soil have been improved. It is found out that the improvements are satisfactory for higher proportion of sand-soil mix. The lower proportions of sand-soil mixture showed a slight improvement in the property of the expansive soil. For instance, for 40% crushed sand blended with expansive soil, PI showed a reduction of 48% similarly, the unconfined compressive strength increased by 60% for 40% crushed sand blended with expansive soil. The same is also true for the natural sand mixed with the expansive soil. A blending of 40% natural sand with the expansive soil has shown a reduction of 46% in PI and the unconfined compressive strength increased by 39%.
- Though the value of the CBR and CBR swell reduced slightly, there is no marked improvement achieved in the class of subgrade strength according to ERA specification.
- After cost comparison, it is recommended that 40% by weight of crushed and natural sands can be used for construction of light weight buildings, for instance, residential building, ware house, etc.

5.2 RECOMMEDATION

- A better result can be attained by introducing third material such as fly ash & lime.
- Another study area could be the use of stone dust in the stabilization of expansive soil.

6. REFERENCES

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7. Annex

Annex A: Grain Size Distributions Curve

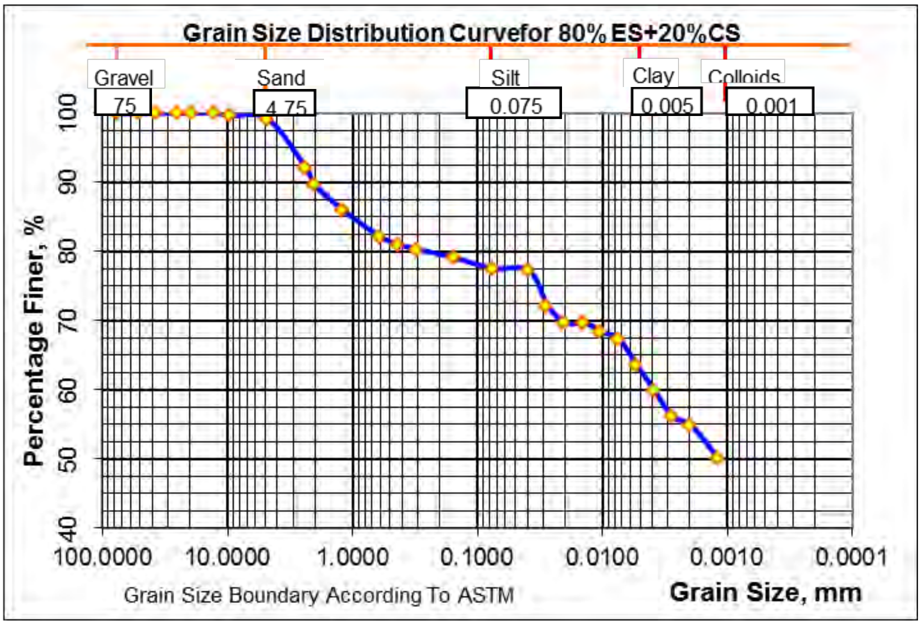

Figure A-1: Grain size distribution curve for 80%ES+20%CS

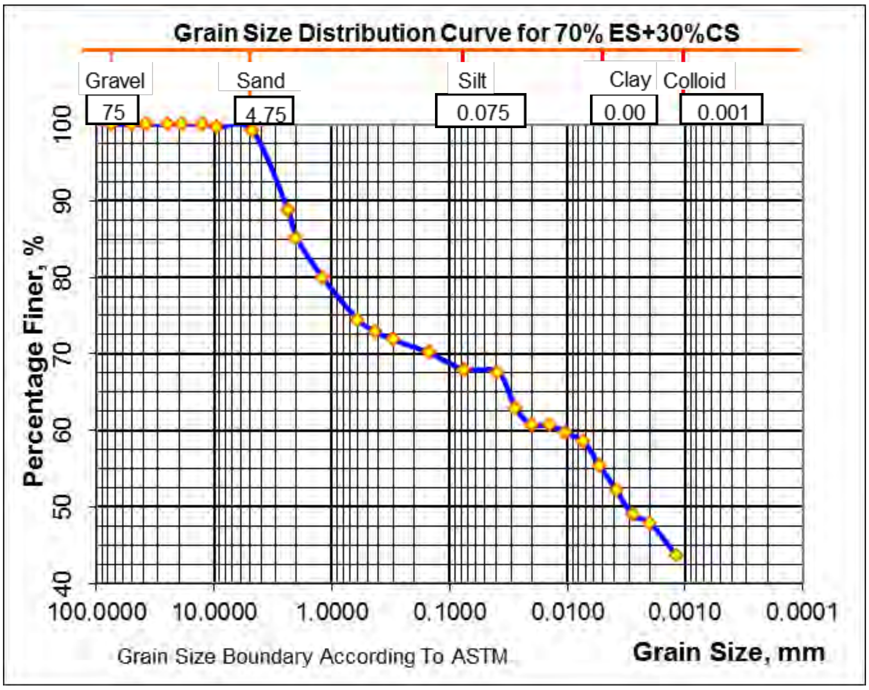

Figure A-2: Grain size distribution curve for 70%ES+30%CS

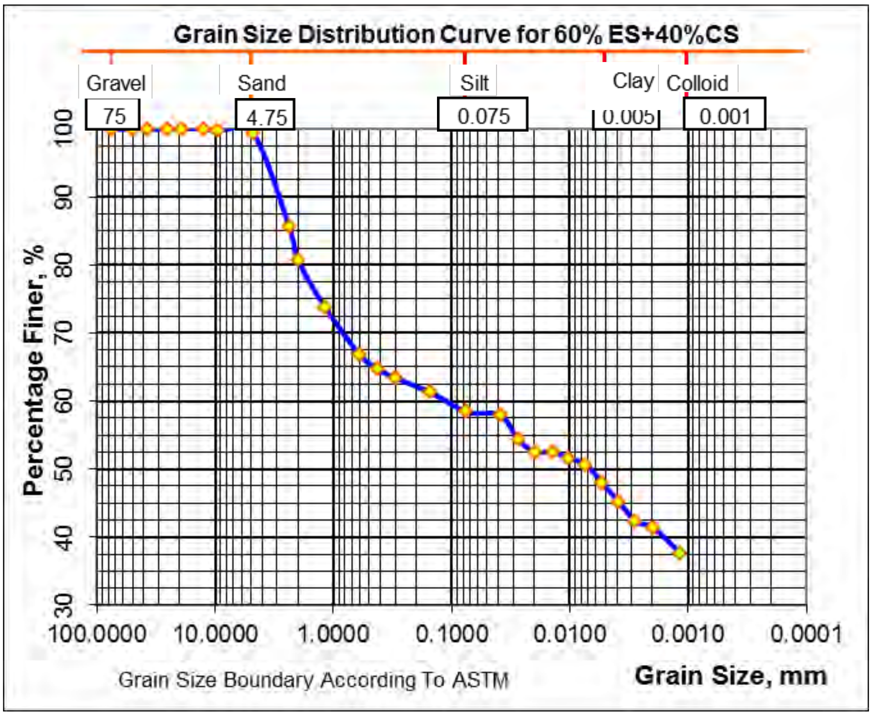

Figure A-3: Grain size distribution curve for 60%ES+40%CS

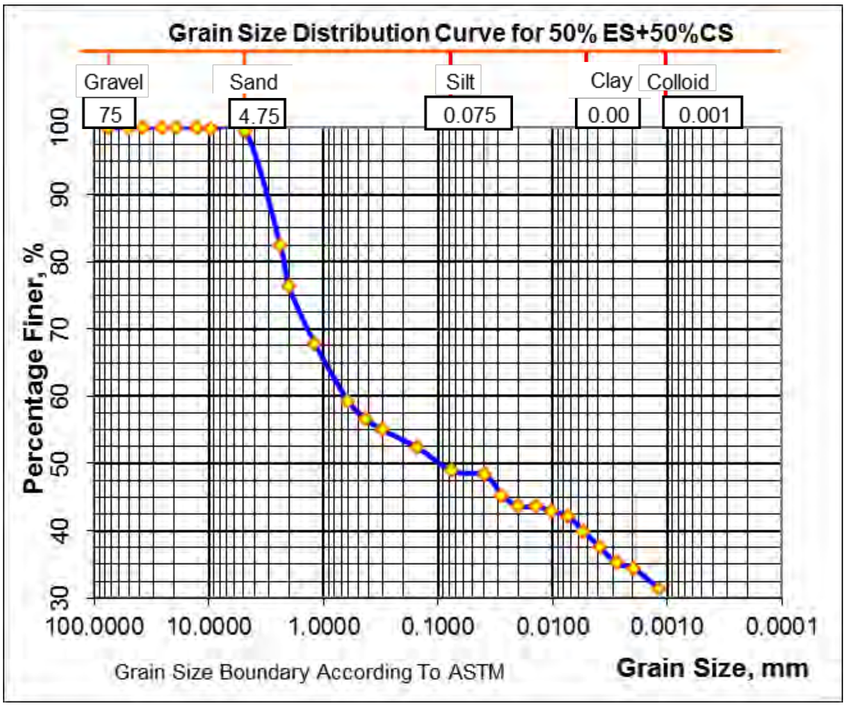

Figure A-4: Grain size distribution curve for 50%ES+50%CS

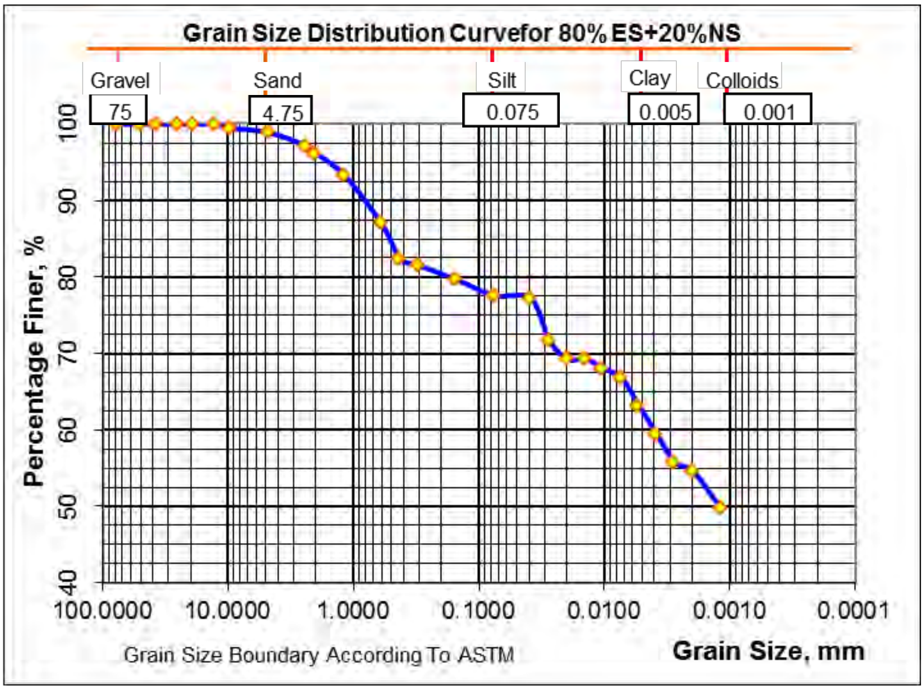

Figure A-5: Grain size distribution curve for 80%ES+20%NS

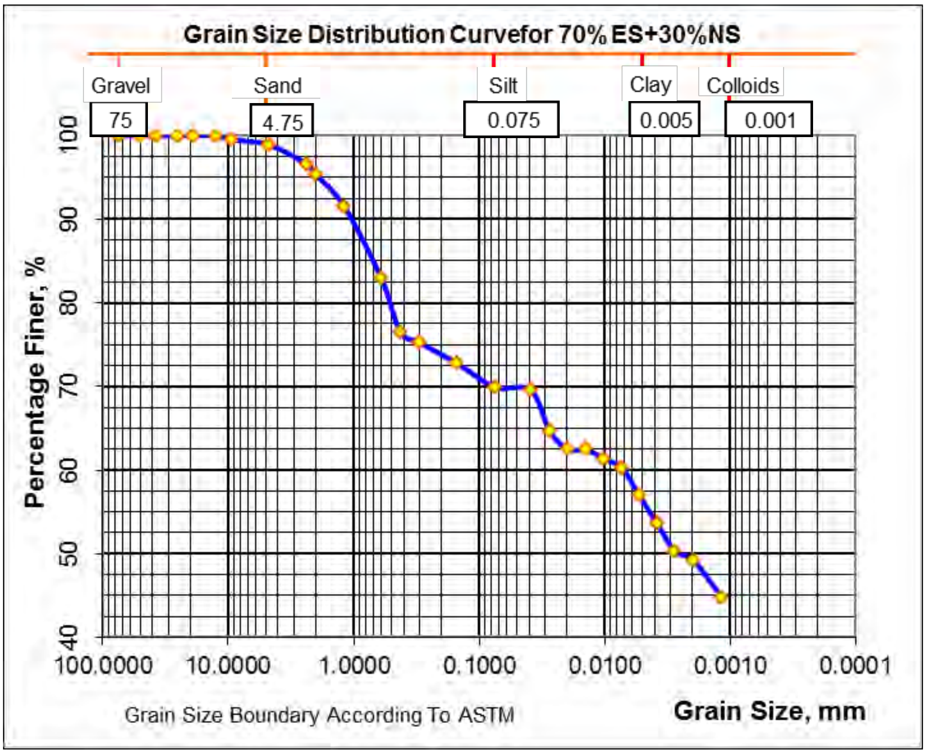

Figure A-6: Grain size distribution curve for 70%ES+30%NS

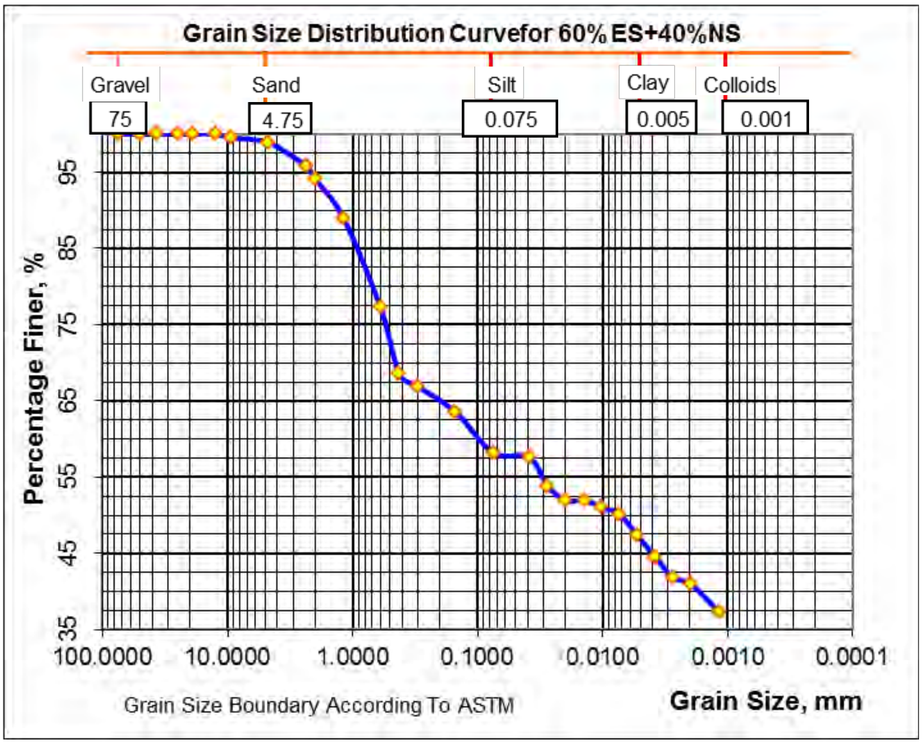

Figure A-7: Grain size distribution curve for 60%ES+40%NS

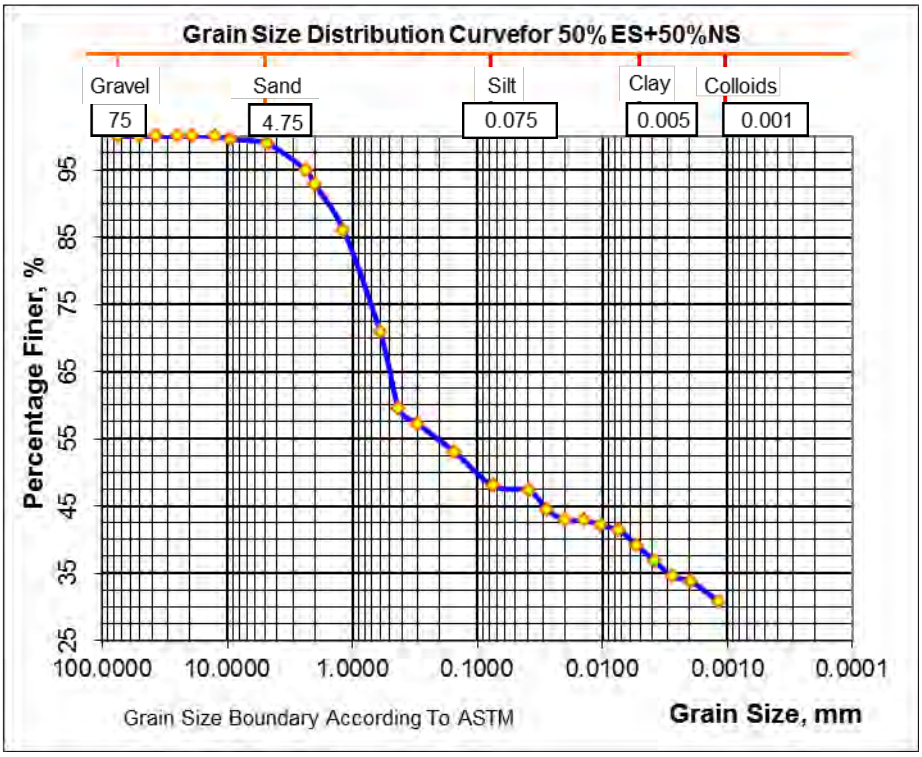

Figure A-8: Grain size distribution curve for 50%ES+50%NS

Annex B: Atterberg Limit Tests Results

Table B-1: Atterberg Limit test for ES

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	MC-C2	p#@1.5	S3,P2,L2	96	Hoo	Goo
Mass of container, g	22.00	20.50	22.10	21.80	22.10	22.20
Mass of container + Wet soil, g	56.70	51.80	50.00	58.10	32.80	30.60
Mass of container + Dry soil, g	40.70	36.90	36.60	40.10	30.10	27.90
Mass of water, g	16.00	14.90	13.40	18.00	2.70	3.10
Mass of dry soil, g	18.70	16.40	14.50	18.30	5.80	6.60
Water content, %	85.56	90.85	92.41	98.36	46.55	46.97
No of blows	36	26	23	16	------	------

Liquid Limit, % = 91 Plastic Limit, % = 47 PI, %= 45

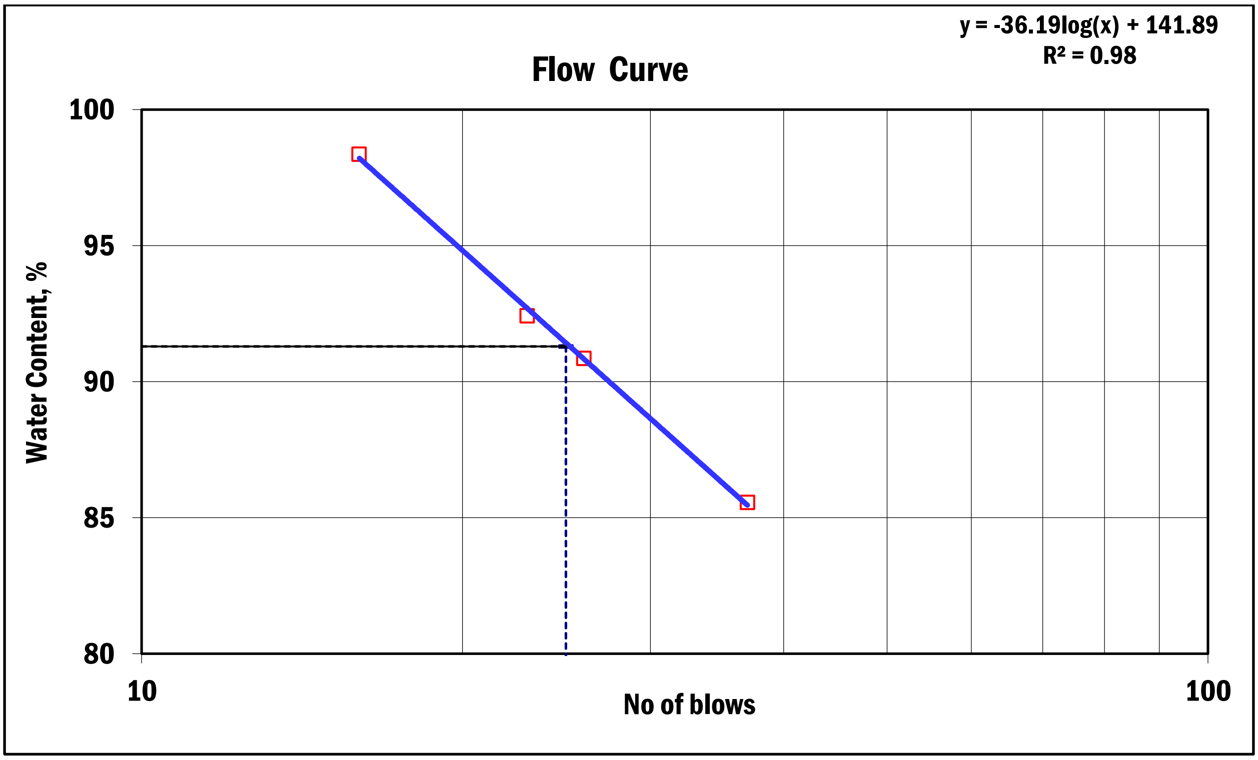

Figure B-1: Flow Curve for ES

Table B-2: Atterberg Limit test for 10%CS+90%ES

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	A1	A35	TP#1	3-1	P280	P280
Mass of container, g	21.80	22.40	20.60	22.40	22.20	22.10
Mass of container + Wet soil, g	35.10	33.00	37.40	38.90	29.60	28.50
Mass of container + Dry soil, g	28.90	27.95	29.25	30.70	27.20	26.35
Mass of water, g	6.20	5.05	8.15	8.20	2.40	2.15
Mass of dry soil, g	7.10	5.55	8.65	8.30	5.00	4.25
Water content, %	87.32	90.99	94.22	98.80	48.00	50.59
No of blows	40	30	25	15	------	------

Liquid Limit, % = 93 Plastic Limit, % = 49 PI, %= 44

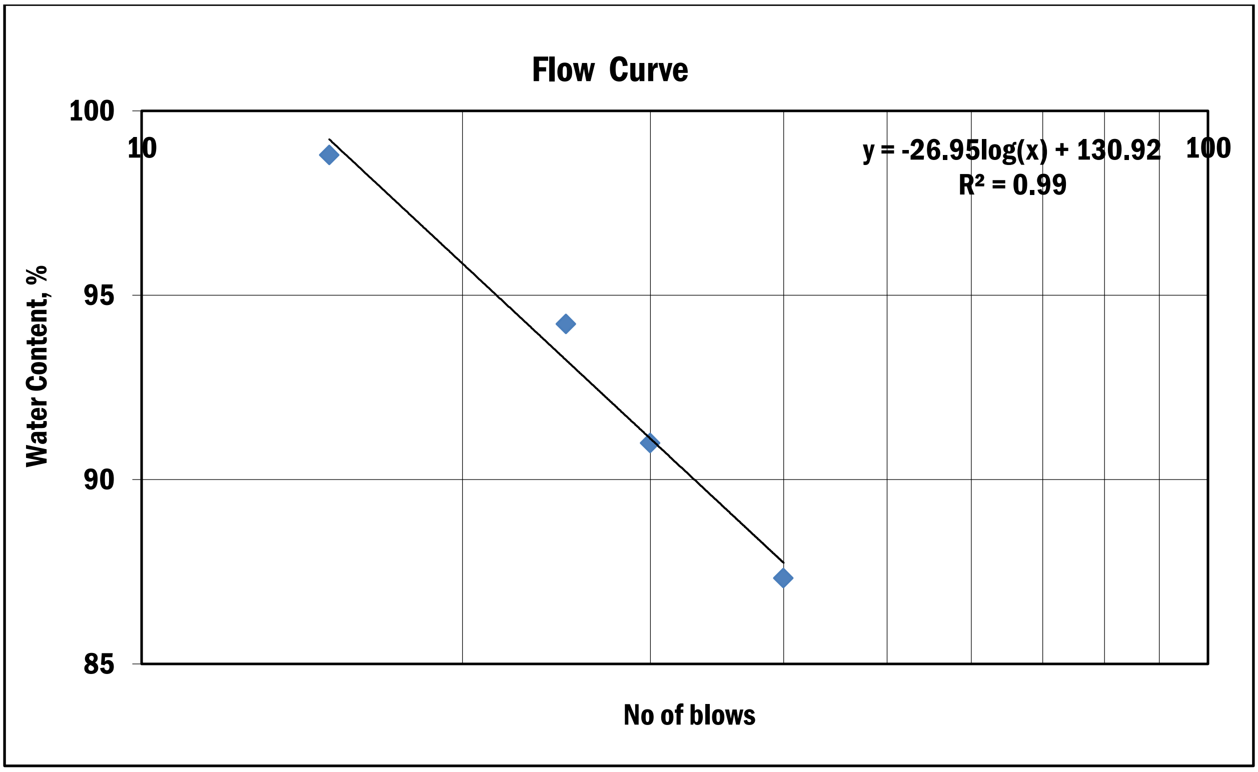

Figure B-2: Flow Curve for 10%CS+90%ES

Table B-3: Atterberg Limit test for 20%CS+80%ES

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	A1	A35	TP#1	3-1	P280	P280
Mass of container, g	22.10	22.10	22.20	22.20	22.20	22.30
Mass of container + Wet soil, g	47.00	47.50	46.90	46.80	28.90	28.50
Mass of container + Dry soil, g	35.40	35.60	35.10	34.90	26.80	26.60
Mass of water, g	11.60	11.90	11.80	11.90	2.10	1.90
Mass of dry soil, g	13.30	13.50	12.90	12.70	4.60	4.30
Water content, %	87.22	88.15	91.47	93.70	45.65	44.19
No of blows	35	26	25	19	------	------

Liquid Limit, % = 90 Plastic Limit, % = 45 PI, %= 45

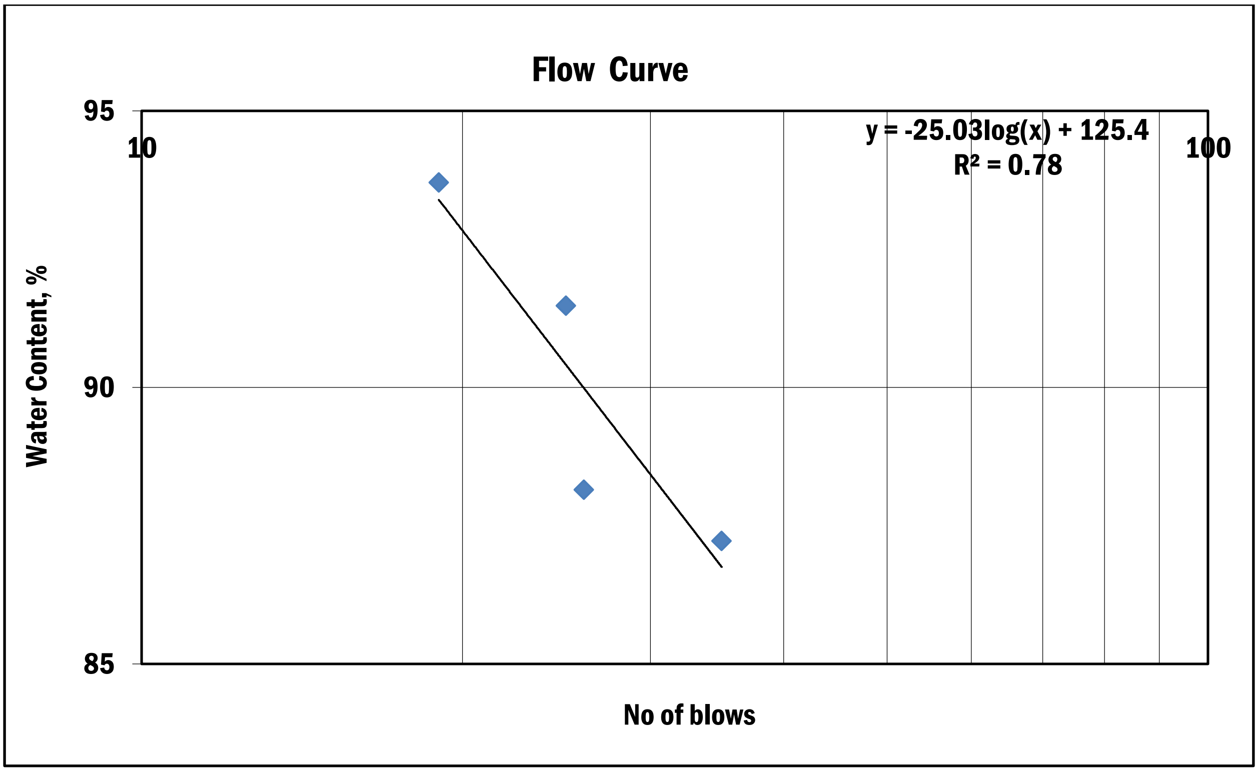

Figure B-3: Flow Curve for 20%CS+80%ES

Table B-4: Atterberg Limit test for 30%CS+70%ES

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	A1	A35	TP#1	3-1	P280	P280
Mass of container, g	22.00	21.80	22.10	21.90	22.40	22.10
Mass of container + Wet soil, g	47.80	47.40	47.40	47.10	29.40	28.80
Mass of container + Dry soil, g	36.80	36.00	36.10	35.60	27.40	26.90
Mass of water, g	11.00	11.40	11.30	11.50	2.00	1.90
Mass of dry soil, g	14.80	14.20	14.00	13.70	5.00	4.80
Water content, %	74.32	80.28	80.71	83.94	40.00	39.58
No of blows	39	27	23	17	------	------

Liquid Limit, % = 80 Plastic Limit, % = 40 PI, %= 40

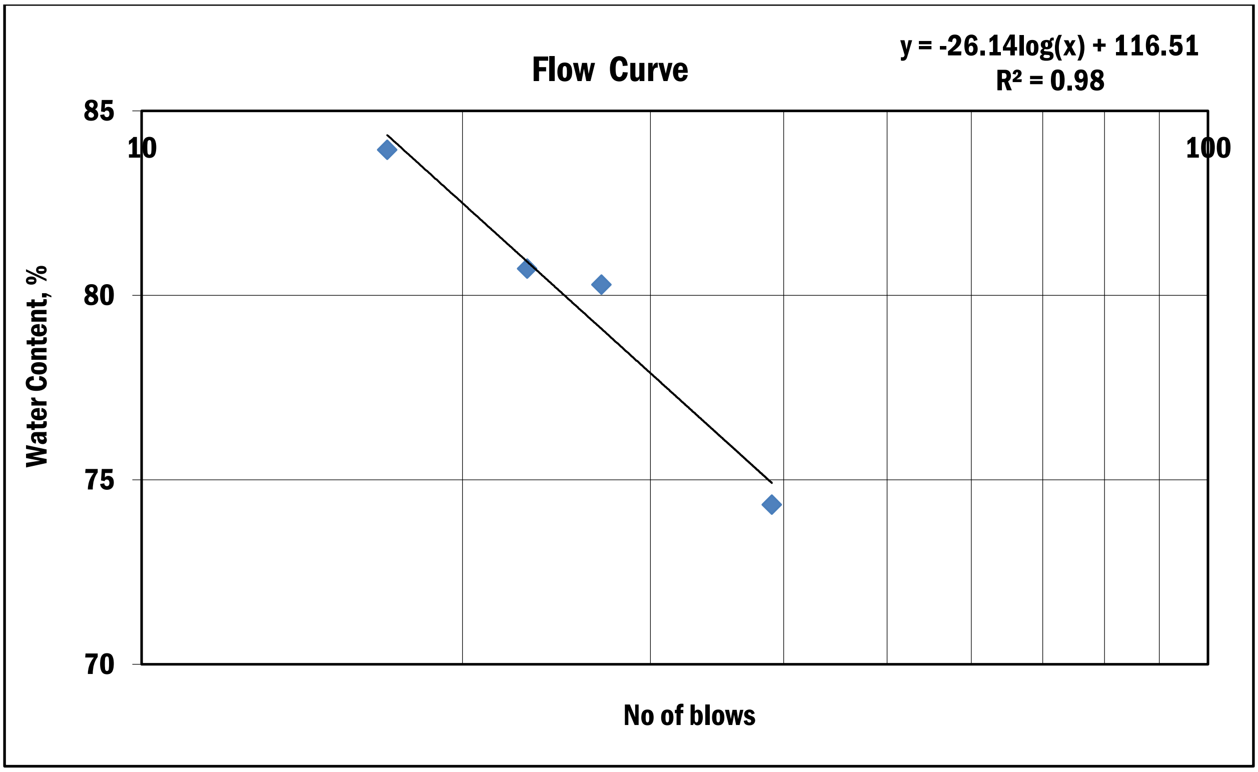

Figure B-4: Flow Curve for 30%CS+70%ES

Table B-5: Atterberg Limit test for 40%CS+60%ES

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	A1	A35	TP#1	3-1	P280	P280
Mass of container, g	22.10	22.10	22.00	22.30	22.20	22.10
Mass of container + Wet soil, g	49.10	47.90	47.00	48.80	30.40	32.70
Mass of container + Dry soil, g	38.60	37.80	37.00	38.00	28.20	29.80
Mass of water, g	10.50	10.10	10.00	10.80	2.20	2.90
Mass of dry soil, g	16.50	15.70	15.00	15.70	6.00	7.70
Water content, %	63.64	64.33	66.67	68.79	36.67	37.66
No of blows	35	30	22	19	------	------

Liquid Limit, % = 66 Plastic Limit, % = 37 PI, %= 29

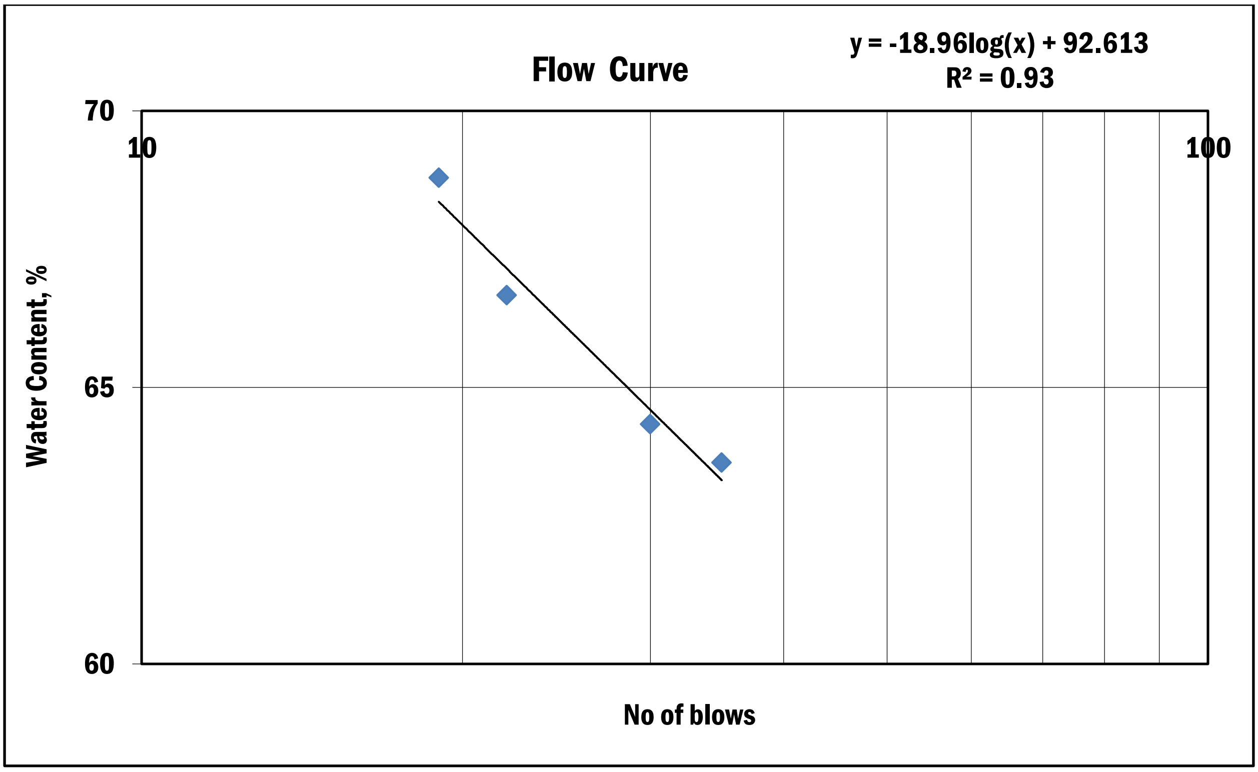

Figure B-5: Flow Curve for 40%CS+60%ES

Table B-6: Atterberg Limit test for 50%CS+50%ES

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	A1	A35	TP#1	3-1	P280	P280
Mass of container, g	22.20	21.70	22.00	22.10	22.20	22.00
Mass of container + Wet soil, g	44.10	38.60	39.40	39.00	33.30	30.60
Mass of container + Dry soil, g	36.00	32.40	32.90	32.50	30.50	28.50
Mass of water, g	8.10	6.20	6.50	6.50	2.80	2.10
Mass of dry soil, g	13.80	10.70	10.90	10.40	8.30	6.50
Water content, %	58.70	57.94	59.63	62.50	33.73	32.31
No of blows	33	29	21	17	------	------

Liquid Limit, % = 59 Plastic Limit, % = 33 PI, %= 26

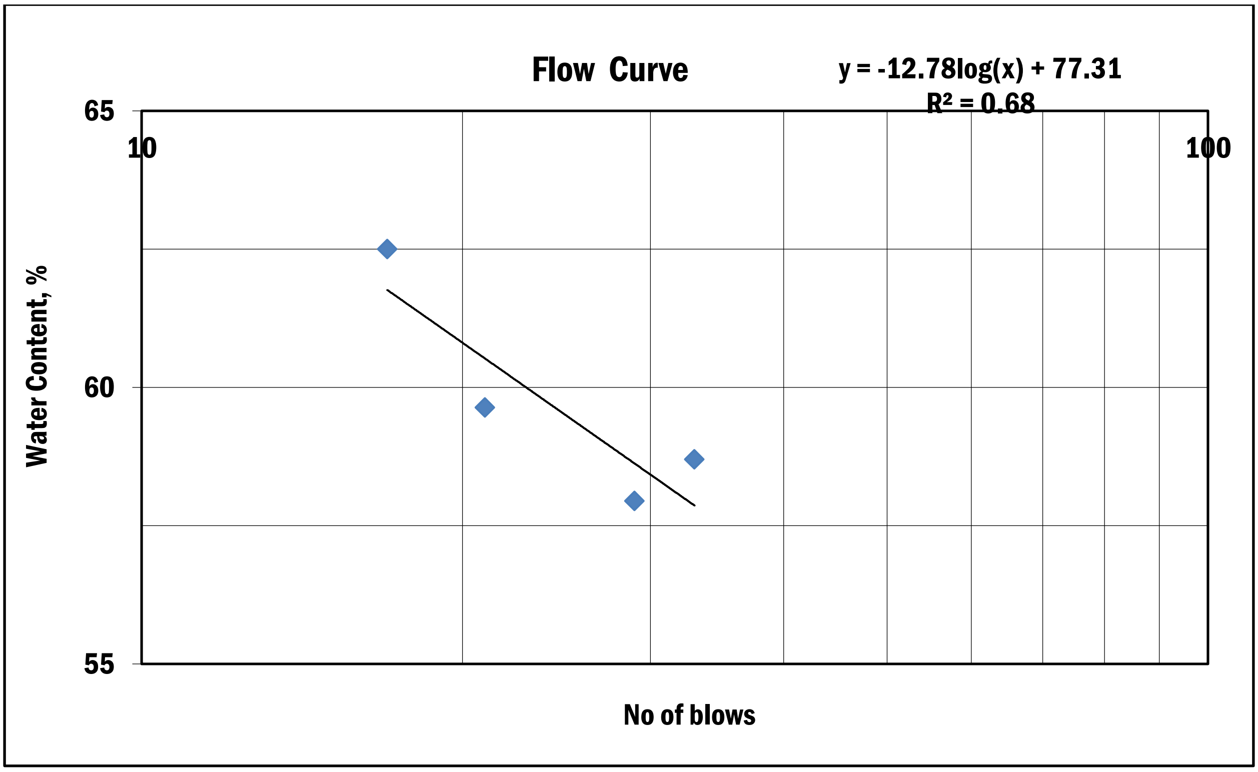

Figure B-6: Flow Curve for 50%CS+50%ES

Table B-7: Atterberg Limit test for 10%NS+90%ES

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	C3	S-2	A1	21		
Mass of container, g	23.20	20.50	22.20	20.00	22.20	22.10
Mass of container + Wet soil, g	36.50	36.60	38.30	39.00	28.40	28.50
Mass of container + Dry soil, g	30.30	29.10	30.60	29.80	26.50	26.50
Mass of water, g	6.20	7.50	7.70	9.20	1.90	2.00
Mass of dry soil, g	7.10	8.60	8.40	9.80	4.30	4.40
Water content, %	87.32	87.21	91.67	93.88	44.19	45.45
No of blows	37	33	26	18	------	------

Liquid Limit, % = 91 Plastic Limit, % = 45 PI, %= 46

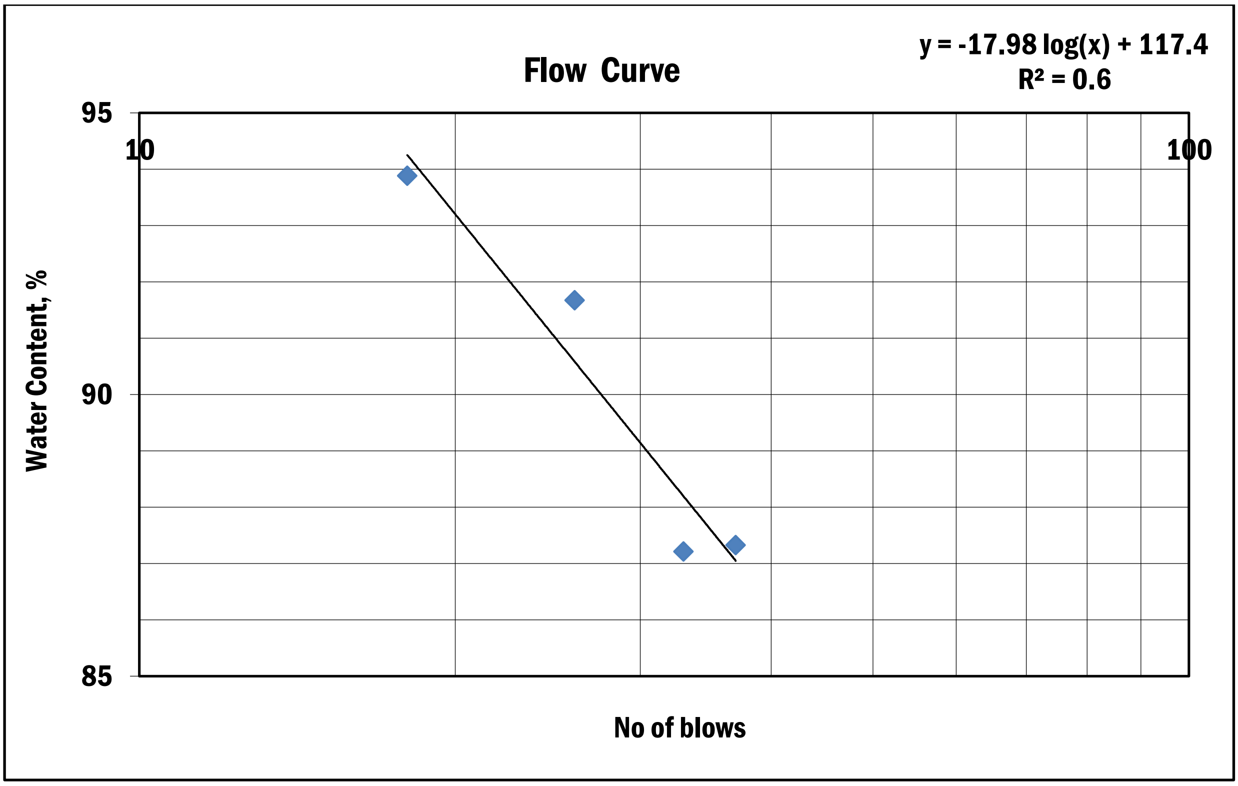

Figure B-7: Flow Curve for 10%NS+90%ES

Table B-8: Atterberg Limit test for 20%NS+80%ES

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	C3	S-2	A1	21		
Mass of container, g	21.80	22.20	22.40	22.30	22.10	22.10
Mass of container + Wet soil, g	39.10	36.10	36.60	39.20	28.70	29.40
Mass of container + Dry soil, g	31.50	29.80	30.10	31.30	26.60	27.10
Mass of water, g	7.60	6.30	6.50	7.90	2.10	2.30
Mass of dry soil, g	9.70	7.60	7.70	9.00	4.50	5.00
Water content, %	78.35	82.89	84.42	87.78	46.67	46.00
No of blows	38	30	24	18	------	------

Liquid Limit, % = 84 Plastic Limit, % = 46 PI, %= 38

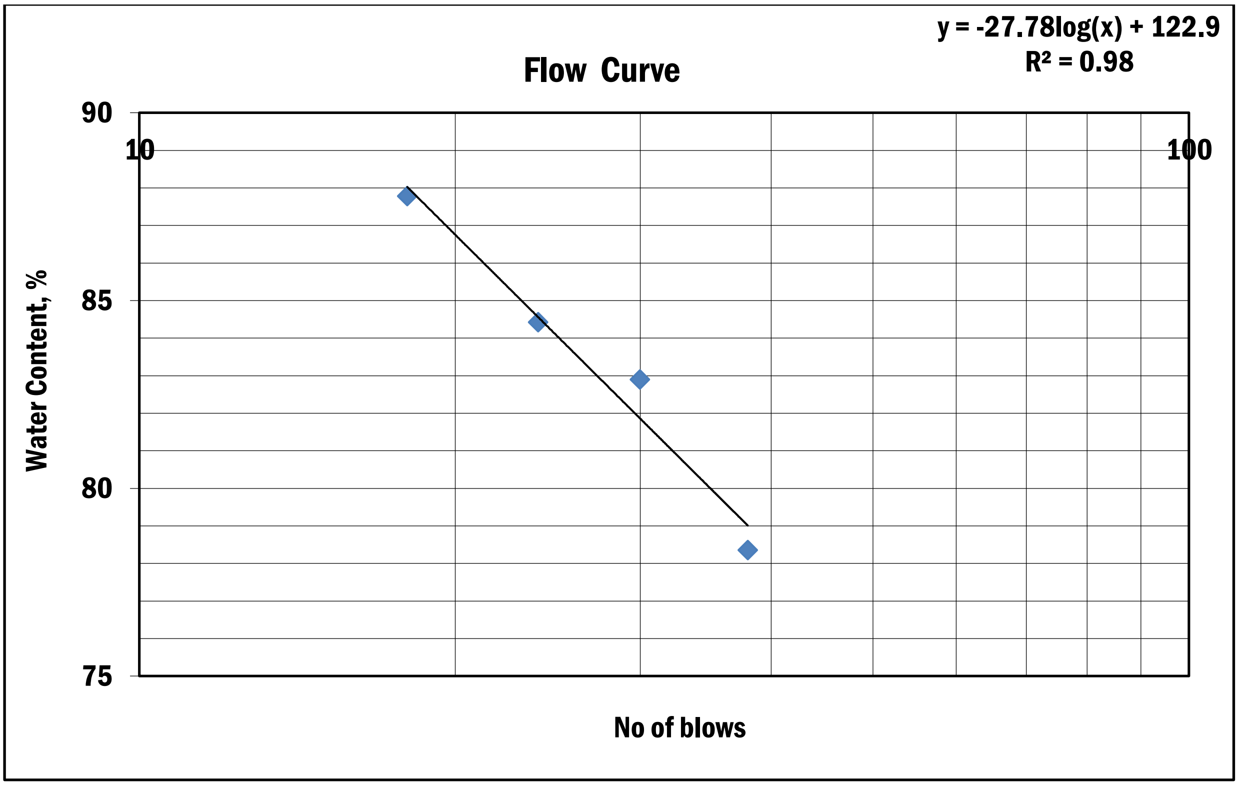

Figure B-8: Flow Curve for 20%NS+80%ES

Table B-9: Atterberg Limit test for 30%NS+70%ES

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	C3	S-2	A1	21		
Mass of container, g	22.40	21.80	22.20	22.40	22.00	22.00
Mass of container + Wet soil, g	37.20	38.00	39.40	39.40	31.10	29.50
Mass of container + Dry soil, g	31.20	31.30	32.20	32.20	28.80	27.30
Mass of water, g	6.00	6.70	7.20	7.20	2.30	2.20
Mass of dry soil, g	8.80	9.50	10.00	9.80	6.80	5.30
Water content, %	68.18	70.53	72.00	73.47	33.82	41.51
No of blows	40	30	23	18	------	------

Liquid Limit, % = 71 Plastic Limit, % = 38 PI, %= 34

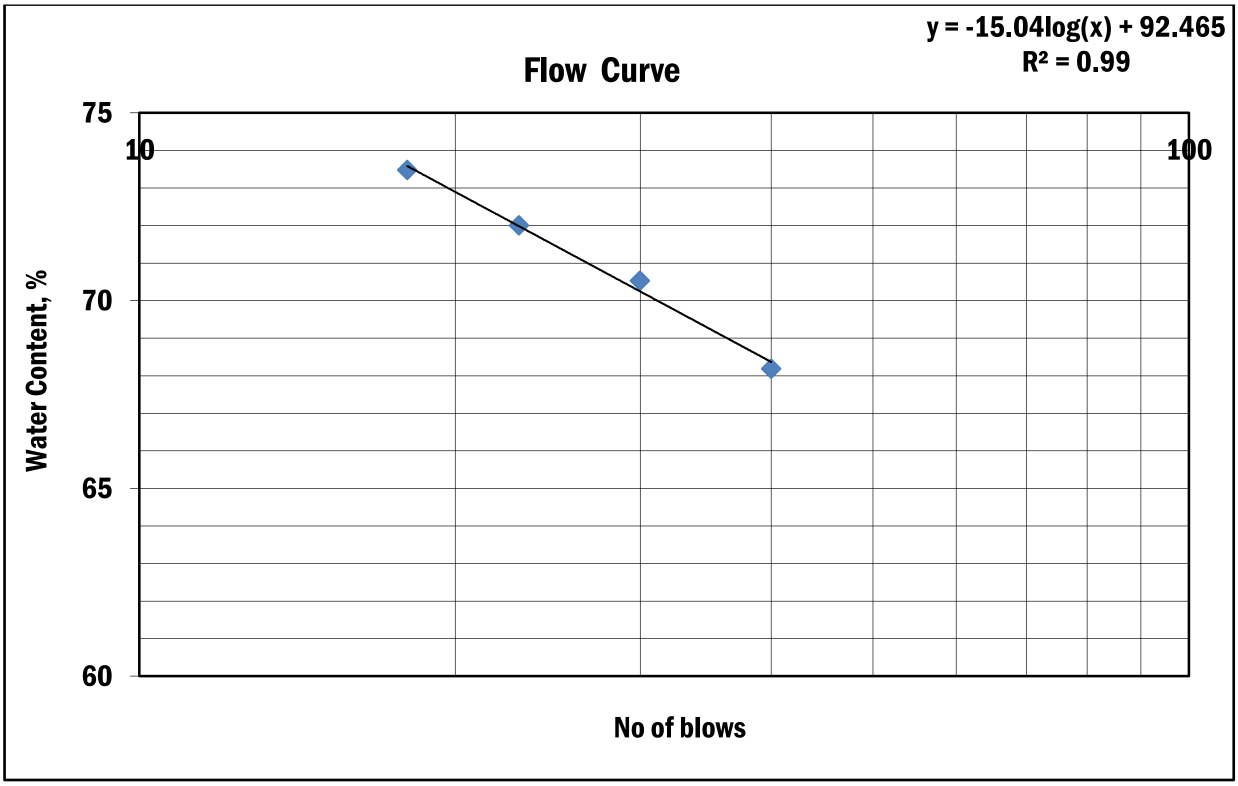

Figure B-9: Flow Curve for 30%NS+70%ES

Table B-10: Atterberg Limit test for 40%NS+60%ES

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	C3	S-2	A1	21		
Mass of container, g	22.00	22.30	22.10	22.00	22.00	22.00
Mass of container + Wet soil, g	42.40	39.10	38.00	38.10	32.00	32.90
Mass of container + Dry soil, g	34.80	32.70	31.80	31.80	29.50	30.30
Mass of water, g	7.60	6.40	6.20	6.30	2.50	2.60
Mass of dry soil, g	12.80	10.40	9.70	9.80	7.50	8.30
Water content, %	59.38	61.54	63.92	64.29	33.33	31.33
No of blows	37	27	24	19	------	------

Liquid Limit, % = 63 Plastic Limit, % = 32 PI, %= 30

Figure B-10: Flow Curve for 40%NS+60%ES

Table B-11: Atterberg Limit test for 50%NS+50%ES

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	C3	S-2	A1	21		
Mass of container, g	22.30	22.20	22.10	22.40	22.30	22.30
Mass of container + Wet soil, g	39.10	40.40	40.50	39.90	29.70	29.40
Mass of container + Dry soil, g	33.20	34.00	34.00	33.50	28.00	27.80
Mass of water, g	5.90	6.40	6.50	6.40	1.70	1.60
Mass of dry soil, g	10.90	11.80	11.90	11.10	5.70	5.50
Water content, %	54.13	54.24	54.62	57.66	29.82	29.09
No of blows	34	28	23	16	------	------

Liquid Limit, % = 55 Plastic Limit, % = 29 PI, %= 26

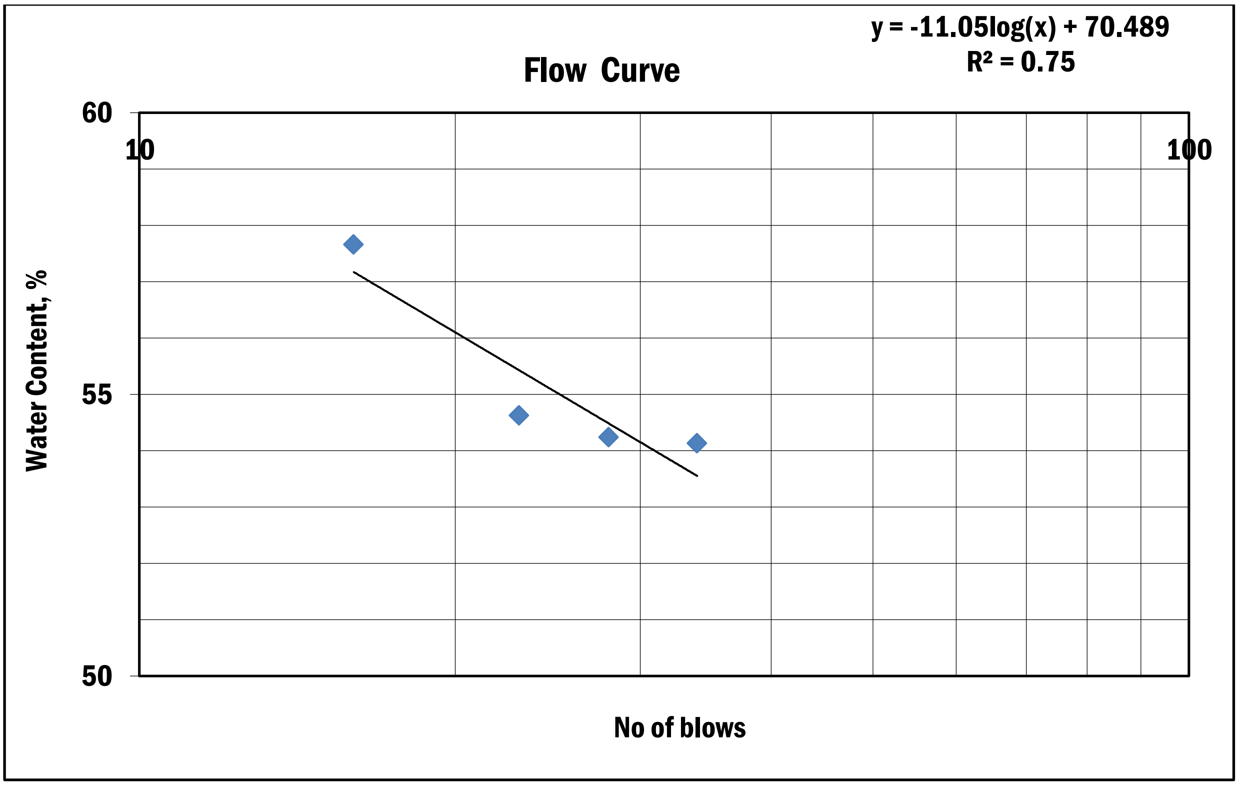

Figure B-11: Flow Curve for 50%NS+50%ES

Annex C: Moisture-Density Relationship

Table C-1: Moisture-Density Relationship for ES

A	Mold	No.	1	2	3	4	5
B	Wt. of Mold + Wet Soil	grams	4548.6	4693.9	4731.1	4723.8	4700.6
C	Wt. of Mold	grams	3144.7	3144.7	3144.7	3144.7	3144.7
D	Wt. Wet Soil	grams	1403.9	1549.2	1586.4	1579.1	1555.9
E	Volume of Mold	cu.cm.	944.0	944.0	944.0	944.0	944.0
F	Wet Density	gr/cu.cm.	1.487	1.641	1.681	1.673	1.648
G	Container	No.	Hoo	96	C6	H1	21
H	Wt. Cont + Wet soil	grams	45.1	47.0	47.5	47.1	47.6
I	Wt. Cont + Dry soil	grams	39.8	40.7	40.8	40.3	40.6
J	Weight of Water	grams	5.3	6.3	6.7	6.8	7.0
K	Weight of Container	grams	22.1	22.1	21.9	21.7	21.9
L	Weight of Dry Soil	grams	17.7	18.6	18.9	18.6	18.7
M	Moisture Content	%	29.9	33.9	35.4	36.6	37.4
N	Dry Density	gr/cu.cm.	1.144	1.226	1.241	1.225	1.199

Maximum Dry Density (MDD):

MDD = 1.241

Optimum Moisture Content (OMC) :

OMC = 35.4 %

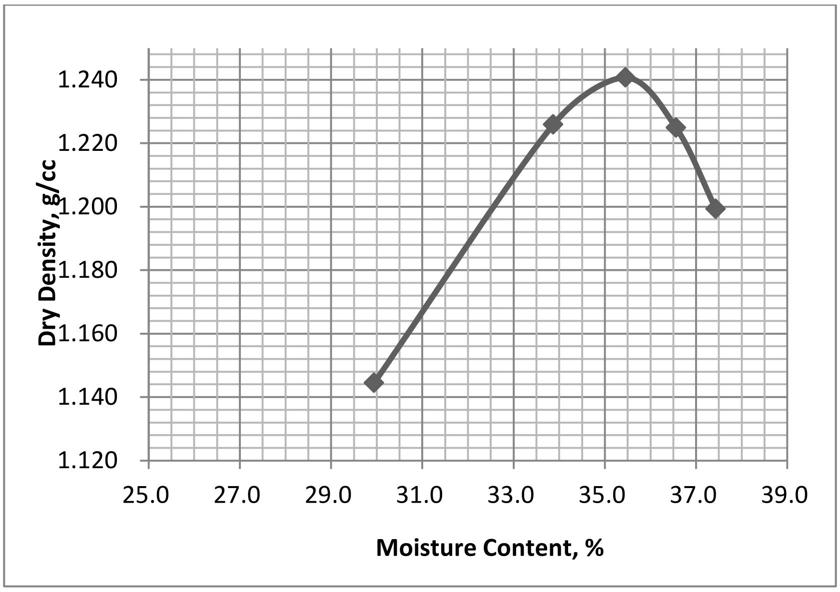

Figure C-1: Moisture-Density Relationship for ES

Table C-2: Moisture-Density Relationship for 20%CS+80%ES

A	Mold	No.	1	2	3	4	5
B	Wt. of Mold + Wet Soil	grams	5841.2	4743.1	4849.9	4810.2	6033.1
C	Wt. of Mold	grams	4392.9	3143.0	3143.0	3143.0	4392.9
D	Wt. Wet Soil	grams	1448.3	1600.1	1706.9	1667.2	1640.2
E	Volume of Mold	cu.cm.	944.0	944.0	944.0	944.0	944.0
F	Wet Density	gr/cu.cm.	1.534	1.695	1.808	1.766	1.738

G	Container	No.	A1	C3	A35	3-1	3-1
H	Wt. Cont + Wet soil	grams	41.2	47.6	45.3	44.6	47.3
I	Wt. Cont + Dry soil	grams	37.4	41.9	39.5	38.4	39.6
J	Weight of Water	grams	3.8	5.7	5.8	6.2	7.7
K	Weight of Container	grams	22.1	21.9	22.1	22.0	22.4
L	Weight of Dry Soil	grams	15.3	20.0	17.4	16.4	17.2

M	Moisture Content	%	24.8	28.5	33.3	37.8	44.8
N	Dry Density	gr/cu.cm.	1.229	1.319	1.356	1.282	1.200

Maximum Dry Density (MDD):

Optimum Moisture Content (OMC) :

MDD = 1.356 gm/cc

OMC = 33.3 %

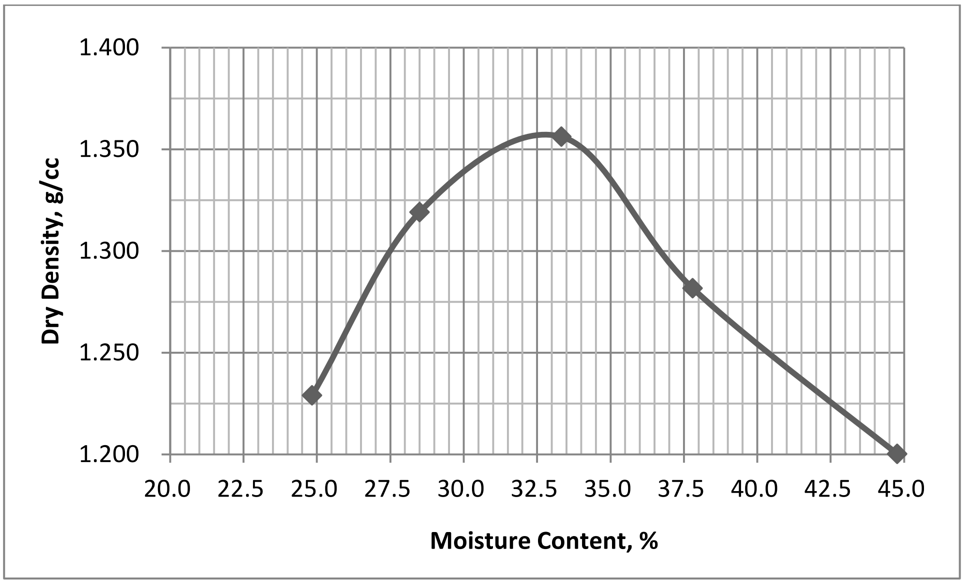

Figure C-2: Moisture-Density Relationship for 20%CS+80%ES

Table C-3: Moisture-Density Relationship for 30%CS+70%ES

A	Mold	No.	1	2	3	4
B	Wt. of Mold + Wet Soil	grams	4728.5	4859.3	4851.5	4841.1
C	Wt. of Mold	grams	3143.0	3143.0	3143.0	3143.0
D	Wt. Wet Soil	grams	1585.5	1716.3	1708.5	1698.1
E	Volume of Mold	cu.cm.	944.0	944.0	944.0	944.0
F	Wet Density	gr/cu.cm.	1.680	1.818	1.810	1.799

G	Container	No.	HOO	B4	21	S-2,P-2,L-2
H	Wt. Cont + Wet soil	grams	46.1	46.1	47.6	47.3
I	Wt. Cont + Dry soil	grams	40.9	40.3	41.0	40.0
J	Weight of Water	grams	5.2	5.8	6.6	7.3
K	Weight of Container	grams	22.2	22.1	22.0	22.2
L	Weight of Dry Soil	grams	18.7	18.2	19.0	17.8

M	Moisture Content	%	27.8	31.9	34.7	41.0
N	Dry Density	gr/cu.cm.	1.314	1.379	1.343	1.276

Maximum Dry Density (MDD): **Optimum Moisture Content (OMC) :**

MDD = 1.379 OMC = 31.9 %

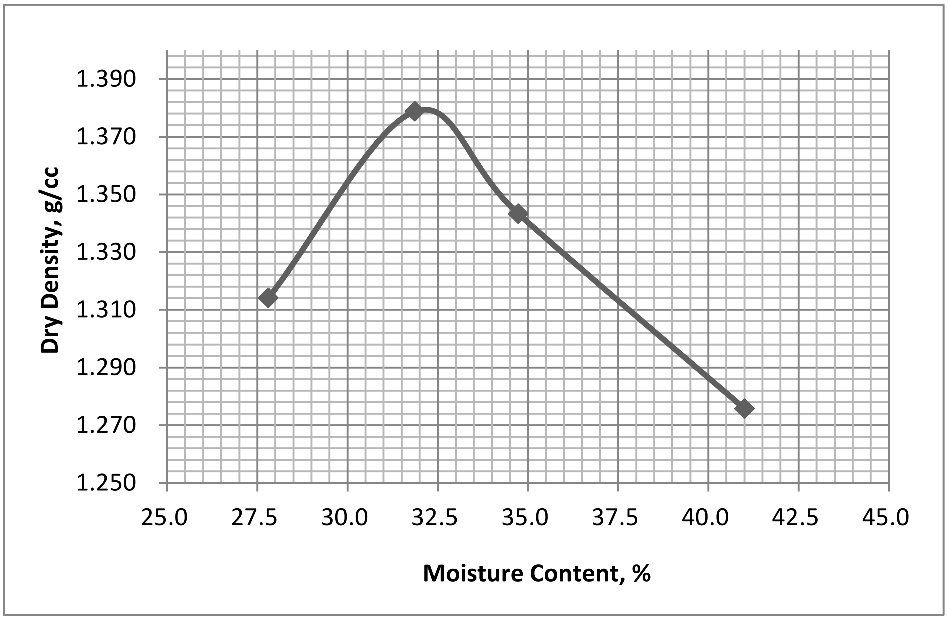

Figure C-3: Moisture-Density Relationship for 30%CS+70%ES

Table C-4: Moisture-Density Relationship for 40%CS+60%ES

A	Mold	No.	1	2	3	4	5
B	Wt. of Mold + Wet Soil	grams	4810.0	4900.0	4932.5	4911.0	4899.4
C	Wt. of Mold	grams	3143.0	3143.0	3143.0	3143.0	3143.0
D	Wt. Wet Soil	grams	1667.0	1757.0	1789.5	1768.0	1756.4
E	Volume of Mold	cu.cm.	944.0	944.0	944.0	944.0	944.0
F	Wet Density	gr/cu.cm.	1.766	1.861	1.896	1.873	1.861

G	Container	No.	PO	L1	C3	TP#1	C33
H	Wt. Cont + Wet soil	grams	47.3	47.9	49.4	48.2	48.4
I	Wt. Cont + Dry soil	grams	42.7	42.7	43.6	41.6	41.6
J	Weight of Water	grams	4.6	5.2	5.8	6.6	6.8
K	Weight of Container	grams	22.3	22.3	22.0	22.1	22.3
L	Weight of Dry Soil	grams	20.4	20.4	21.6	19.5	19.3

M	Moisture Content	%	22.5	25.5	26.9	33.8	35.2
N	Dry Density	gr/cu.cm.	1.441	1.483	1.494	1.399	1.376

Maximum Dry Density (MDD): **Optimum Moisture Content (OMC) :** **Optimum Moisture Content (OMC) :**

MDD = 1.494 OMC = OMC = 26.9 %

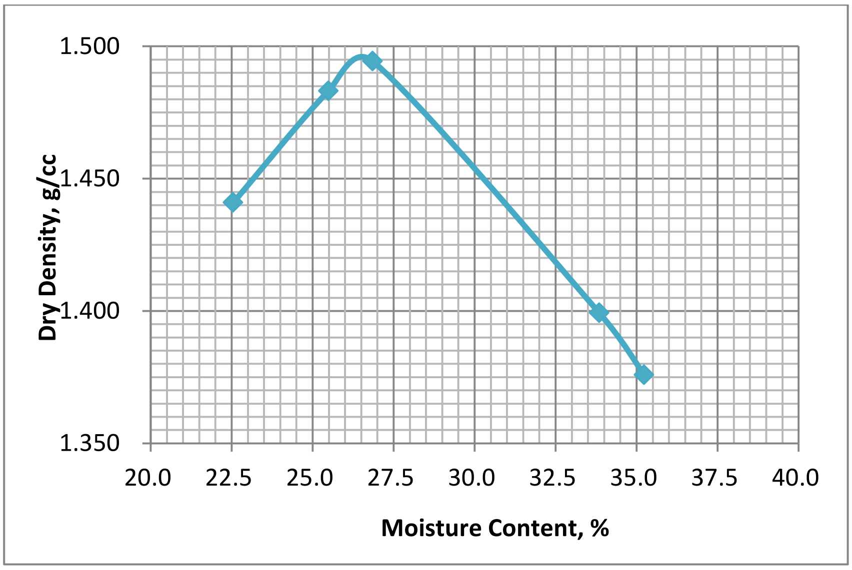

Figure C-4: Moisture-Density Relationship for 40%CS+60%ES

Table C-5: Moisture-Density Relationship for 50%CS+50%ES

A	Mold	No.	1	2	3	4	5
B	Wt. of Mold + Wet Soil	grams	4785.0	4832.1	4994.5	4989.0	4923.0
C	Wt. of Mold	grams	3143.0	3143.0	3143.0	3143.0	3143.0
D	Wt. Wet Soil	grams	1642.0	1689.1	1851.5	1846.0	1780.0
E	Volume of Mold	cu.cm.	944.0	944.0	944.0	944.0	944.0
F	Wet Density	gr/cu.cm.	1.739	1.789	1.961	1.956	1.886
G	Container	No.	R3	L1	C3	TP#1	#1
H	Wt. Cont + Wet soil	grams	49.3	51.1	48.7	51.8	51.7
I	Wt. Cont + Dry soil	grams	44.6	45.9	43.4	44.4	43.7
J	Weight of Water	grams	4.7	5.2	5.3	7.4	8.0
K	Weight of Container	grams	22.6	22.6	22.8	22.4	22.4
L	Weight of Dry Soil	grams	22.0	23.3	20.6	22.0	21.3
M	Moisture Content	%	21.4	22.3	25.7	33.6	37.6
N	Dry Density	gr/cu.cm.	1.433	1.463	1.560	1.463	1.371

Maximum Dry Density (MDD): **Optimum Moisture Content (OMC) :**

MDD = 1.560 OMC = 25.7 %

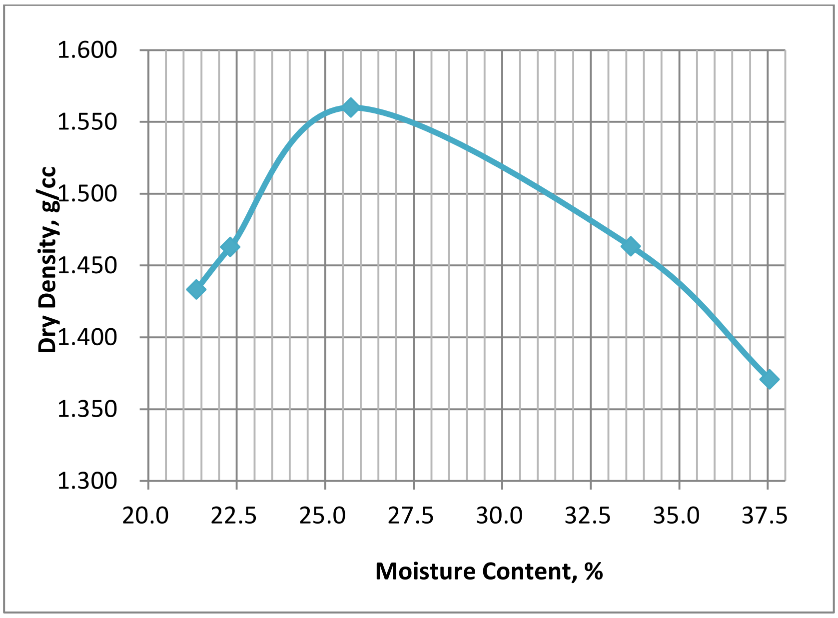

Figure C-5: Moisture-Density Relationship for 50%CS+50%ES

Table C-6: Moisture-Density Relationship for 20%NS+80%ES

A	Mold	No.	1	2	3	4	
B	Wt. of Mold + Wet Soil	grams	**4579.2**	**4684.6**	**4768.9**	**4762.4**	**4697.4**
C	Wt. of Mold	grams	**3140.9**	**3140.9**	**3140.9**	**3140.9**	**3140.9**
D	Wt. Wet Soil	grams	1438.3	1543.7	1628.0	1621.5	1556.5
E	Volume of Mold	cu.cm.	**944.0**	**944.0**	**944.0**	**944.0**	**944.0**
F	Wet Density	gr/cu.cm.	1.524	1.635	1.725	1.718	1.649

G	Container	No.	A1	A35	3-1	3-1	3-1
H	Wt. Cont + Wet soil	grams	**48.3**	**47.7**	**46.4**	**44.8**	**45.8**
I	Wt. Cont + Dry soil	grams	**43.3**	**42.2**	**40.6**	**38.7**	**38.5**
J	Weight of Water	grams	5.0	5.5	5.8	6.1	7.3
K	Weight of Container	grams	**22.3**	**22.3**	**22.1**	**22.3**	**21.8**
L	Weight of Dry Soil	grams	21.0	19.9	18.5	16.4	16.7

M	Moisture Content	%	23.8	27.6	31.4	37.2	43.7
N	Dry Density	gm/cu.cm.	1.231	1.281	1.313	1.252	1.147

Maximum Dry Density (MDD): **Optimum Moisture Content (OMC) :**

MDD = 1.313 gm/cc OMC = 31.4 %

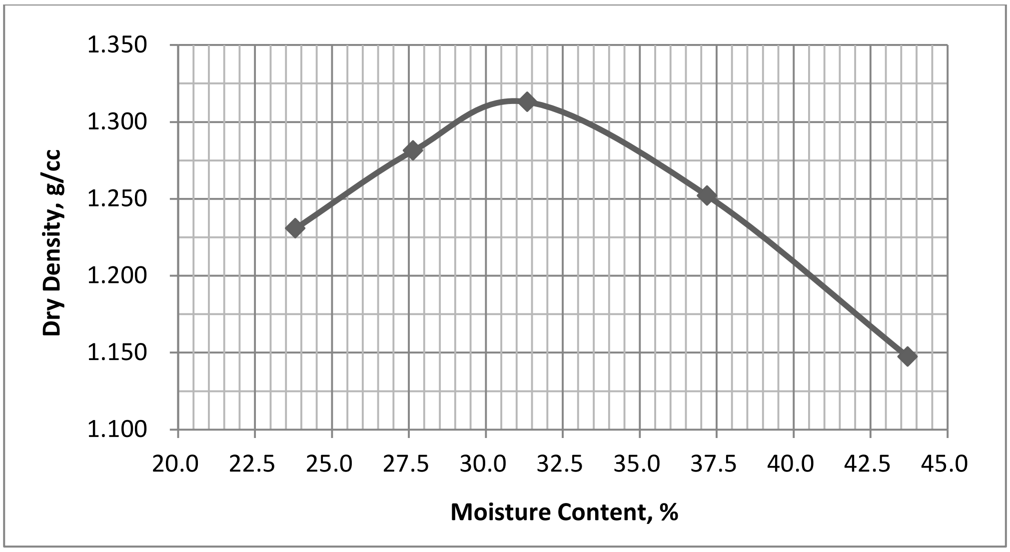

Figure C-6: Moisture-Density Relationship for 20%NS+80%ES

Table C-7: Moisture-Density Relationship for 30%NS+70%ES

A	Mold	No.	1	2	3	4
B	Wt. of Mold + Wet Soil	grams	4647.2	4785.7	4783.0	4740.6
C	Wt. of Mold	grams	3140.8	3140.8	3140.8	3140.8
D	Wt. Wet Soil	grams	1506.4	1644.9	1642.2	1599.8
E	Volume of Mold	cu.cm.	944.0	944.0	944.0	944.0
F	Wet Density	gr/cu.cm.	1.596	1.742	1.740	1.695

G	Container	No.	A1	A35	3-1	3-1
H	Wt. Cont + Wet soil	grams	47.4	46.5	44.4	46.3
I	Wt. Cont + Dry soil	grams	42.9	41.0	39.0	39.3
J	Weight of Water	grams	4.5	5.5	5.4	7.0
K	Weight of Container	grams	22.3	21.8	22.3	21.9
L	Weight of Dry Soil	grams	20.6	19.2	16.7	17.4

M	Moisture Content	%	21.8	28.6	32.3	40.2
N	Dry Density	gr/cu.cm.	1.310	1.354	1.315	1.209

Maximum Dry Density (MDD): **Optimum Moisture Content (OMC) :**

MDD = 1.354 OMC = 28.6 %

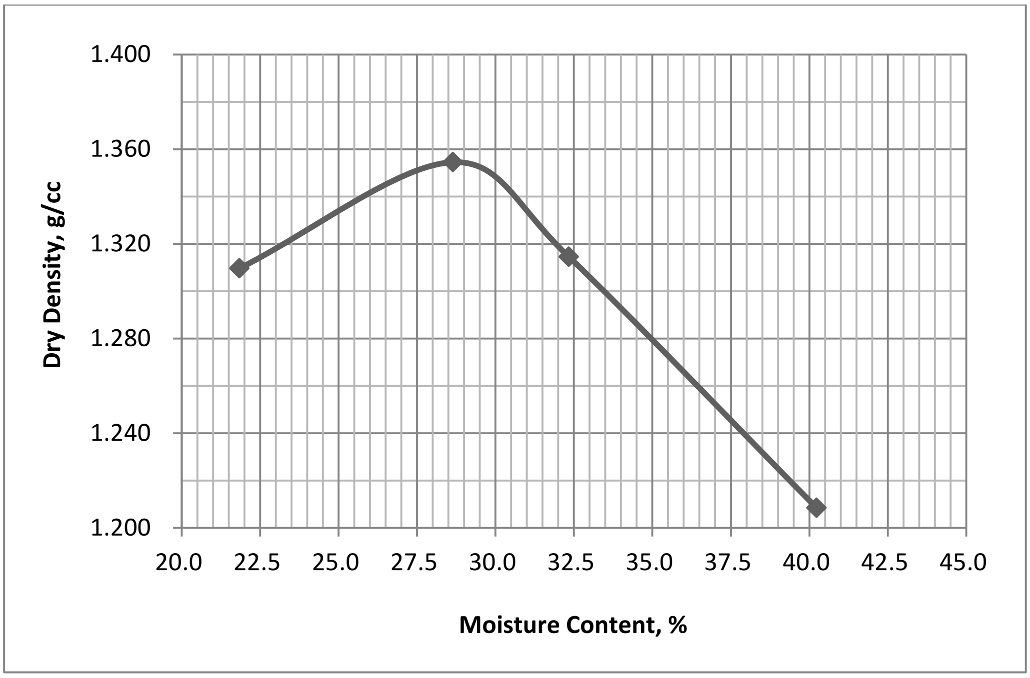

Figure C-7: Moisture-Density Relationship for 30%NS+70%ES

Table C-8: Moisture-Density Relationship for 40%NS+60%ES

A	Mold	No.	1	2	3	4	5
B	Wt. of Mold + Wet Soil	grams	4642.0	4746.9	4807.0	4700.0	4682.0
C	Wt. of Mold	grams	3143.4	3143.4	3143.4	3143.4	3143.4
D	Wt. Wet Soil	grams	1498.6	1603.5	1663.6	1556.6	1538.6
E	Volume of Mold	cu.cm.	944.0	944.0	944.0	944.0	944.0
F	Wet Density	gr/cu.cm.	1.588	1.699	1.762	1.649	1.630

G	Container	No.	A1	A1	A1	A35	3-1
H	Wt. Cont + Wet soil	grams	45.6	45.6	47.6	46.3	48.5
I	Wt. Cont + Dry soil	grams	41.5	41.1	42.1	40.0	41.3
J	Weight of Water	grams	4.1	4.5	5.5	6.3	7.2
K	Weight of Container	grams	22.4	22.4	22.0	20.5	21.9
L	Weight of Dry Soil	grams	19.1	18.7	20.1	19.5	19.4

M	Moisture Content	%	21.5	24.1	27.4	32.3	37.1
N	Dry Density	gr/cu.cm.	1.307	1.369	1.384	1.246	1.189

Maximum Dry Density (MDD): **Optimum Moisture Content (OMC) :**

MDD = 1.384 OMC = 27.4

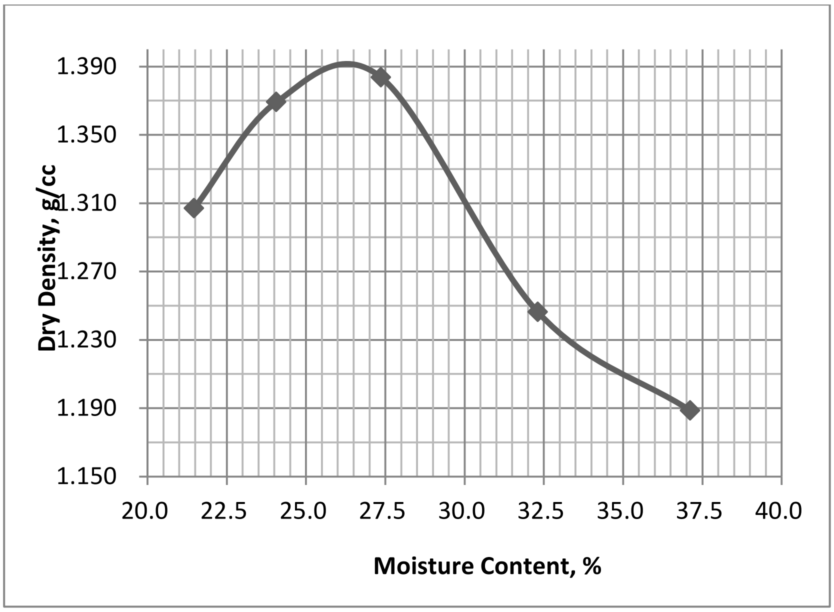

Figure C-8: Moisture-Density Relationship for 40%NS+60%ES

Table C-9: Moisture-Density Relationship for 50%NS+50%ES

A	Mold	No.	1	2	3	4
B	Wt. of Mold + Wet Soil	grams	4736.8	4877.2	4882.0	4814.7
C	Wt. of Mold	grams	3143.4	3143.4	3143.4	3143.4
D	Wt. Wet Soil	grams	1593.4	1733.8	1738.6	1671.3
E	Volume of Mold	cu.cm.	944.0	944.0	944.0	944.0
F	Wet Density	gr/cu.cm.	1.688	1.837	1.842	1.770

G	Container	No.	A1	A35	3-1	3-1
H	Wt. Cont + Wet soil	grams	45.8	45.1	45.2	45.6
I	Wt. Cont + Dry soil	grams	41.5	39.7	39.9	39.2
J	Weight of Water	grams	4.3	5.4	5.3	6.4
K	Weight of Container	grams	22.5	20.0	23.1	22.3
L	Weight of Dry Soil	grams	19.0	19.7	16.8	16.9

M	Moisture Content	%	22.6	27.4	31.5	37.9
N	Dry Density	gr/cu.cm.	1.376	1.442	1.400	1.284

Maximum Dry Density (MDD): **Optimum Moisture Content (OMC) :**

MDD = 1.442 OMC = 27.4

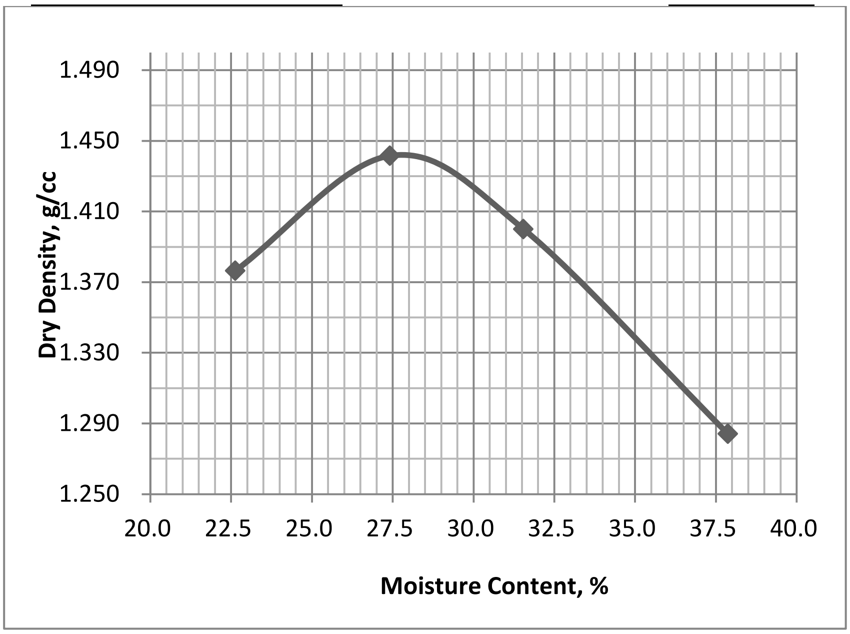

Figure C-9: Moisture-Density Relationship for 50%NS+50%ES

Annex D: Free Swell Test

Table D-1: Free Swell for ES

	SPECIMEN NUMBER	1	2
1	GRADUATED CYLINDER NUMBER	1	2
2	INITIAL VOLUME OF DRY SOIL SAMPLE PASSING SIEVE NO.40 (V_i) IN - CC	10	10
3	FINAL VOLUME OF THE SOIL SAMPLE PASSING SIEVE NO.40 AFTER 24HR SOAKING IN 100CC OF DEMINERALIZED/DISTLLED WATER (V_F) IN CC	25	25
4	**FREE SWELL VOLUME = (V_F - V_I)/V_I**	**150**	**150**
		150	

Table D-2: Free Swell for 10%CS+90% ES

	SPECIMEN NUMBER	1	2
1	GRADUATED CYLINDER NUMBER	1	2
2	INITIAL VOLUME OF DRY SOIL SAMPLE PASSING SIEVE NO.40 (V_i) IN - CC	10	10
3	FINAL VOLUME OF THE SOIL SAMPLE PASSING SIEVE NO.40 AFTER 24HR SOAKING IN 100CC OF DEMINERALIZED/DISTLLED WATER (V_F) IN CC	23	23.5
4	**FREE SWELL VOLUME = (V_F - V_I)/V_I**	**130**	**135**
		132.5	

Table D-3: Free Swell for 20%CS+80% ES

	SPECIMEN NUMBER	1	2
1	GRADUATED CYLINDER NUMBER	1	2
2	INITIAL VOLUME OF DRY SOIL SAMPLE PASSING SIEVE NO.40 (V_i) IN - CC	10	10
3	FINAL VOLUME OF THE SOIL SAMPLE PASSING SIEVE NO.40 AFTER 24HR SOAKING IN 100CC OF DEMINERALIZED/DISTLLED WATER (V_F) IN CC	22.5	22
4	**FREE SWELL VOLUME = (V_F - V_I)/V_I**	**125**	**120**
		122.5	

Table D-4: Free Swell for 30%CS+70% ES

	SPECIMEN NUMBER	1	2
1	GRADUATED CYLINDER NUMBER	1	2
2	INITIAL VOLUME OF DRY SOIL SAMPLE PASSING SIEVE NO.40 (V_i) IN - CC	10	10
3	FINAL VOLUME OF THE SOIL SAMPLE PASSING SIEVE NO.40 AFTER 24HR SOAKING IN 100CC OF DEMINERALIZED/DISTLLED WATER (V_F) IN CC	18.5	19.5
4	**FREE SWELL VOLUME = (V_F - V_I)/V_I**	**85**	**95**
		90	

Table D-5: Free Swell for 40%CS+60% ES

	SPECIMEN NUMBER	1	2
1	GRADUATED CYLINDER NUMBER	1	2
2	INITIAL VOLUME OF DRY SOIL SAMPLE PASSING SIEVE NO.40 (V_i) IN - CC	10	10
3	FINAL VOLUME OF THE SOIL SAMPLE PASSING SIEVE NO.40 AFTER 24HR SOAKING IN 100CC OF DEMINERALIZED/DISTLLED WATER (V_F) IN CC	18	19
4	**FREE SWELL VOLUME = $(V_F - V_I)/V_I$**	**80**	**90**
		85	

Table D-6: Free Swell for 50%CS+50% ES

	SPECIMEN NUMBER	1	2
1	GRADUATED CYLINDER NUMBER	1	2
2	INITIAL VOLUME OF DRY SOIL SAMPLE PASSING SIEVE NO.40 (V_i) IN - CC	10	10
3	FINAL VOLUME OF THE SOIL SAMPLE PASSING SIEVE NO.40 AFTER 24HR SOAKING IN 100CC OF DEMINERALIZED/DISTLLED WATER (V_F) IN CC	18	18
4	**FREE SWELL VOLUME = $(V_F - V_I)/V_I$**	**80**	**80**
		80	

Table D-7: Free Swell for 10%NS+90% ES

SPECIMEN NUMBER	1	2
GRADUATED CYLINDER NUMBER	1	2
INITIAL VOLUME OF DRY SOIL SAMPLE PASSING SIEVE NO.40 (V_i) IN - CC	10	10
FINAL VOLUME OF THE SOIL SAMPLE PASSING SIEVE NO.40 AFTER 24HR SOAKING IN 100CC OF DEMINERALIZED/DISTLLED WATER (V_F) IN CC	24	24.5
	140	**145**
FREE SWELL VOLUME = (V_F - V_I)/V_I	**142.5**	

Table D-8: Free Swell for 20%NS+80% ES

SPECIMEN NUMBER	1	2
GRADUATED CYLINDER NUMBER	1	2
INITIAL VOLUME OF DRY SOIL SAMPLE PASSING SIEVE NO.40 (V_i) IN - CC	10	10
FINAL VOLUME OF THE SOIL SAMPLE PASSING SIEVE NO.40 AFTER 24HR SOAKING IN 100CC OF DEMINERALIZED/DISTLLED WATER (V_F) IN CC	21	21
	110	**110**
FREE SWELL VOLUME = (V_F - V_I)/V_I	**110**	

Table D-9: Free Swell for 30%NS+70% ES

	SPECIMEN NUMBER	1	2
1	GRADUATED CYLINDER NUMBER	1	2
2	INITIAL VOLUME OF DRY SOIL SAMPLE PASSING SIEVE NO.40 (V_i) IN - CC	10	10
3	FINAL VOLUME OF THE SOIL SAMPLE PASSING SIEVE NO.40 AFTER 24HR SOAKING IN 100CC OF DEMINERALIZED/DISTLLED WATER (V_F) IN CC	20	20
		100	**100**
4	**FREE SWELL VOLUME = (V_F - V_I)/V_I**	**100**	

Table D-10: Free Swell for 40%NS+60% ES

	SPECIMEN NUMBER	1	2
1	GRADUATED CYLINDER NUMBER	1	2
2	INITIAL VOLUME OF DRY SOIL SAMPLE PASSING SIEVE NO.40 (V_i) IN - CC	10	10
3	FINAL VOLUME OF THE SOIL SAMPLE PASSING SIEVE NO.40 AFTER 24HR SOAKING IN 100CC OF DEMINERALIZED/DISTLLED WATER (V_F) IN CC	18	17
4	**FREE SWELL VOLUME = (V_F - V_I)/V_I**	**80**	**70**
		75	

Table D-11: Free Swell for 50%NS+50% ES

	SPECIMEN NUMBER	1	2
1	GRADUATED CYLINDER NUMBER	1	2
2	INITIAL VOLUME OF DRY SOIL SAMPLE PASSING SIEVE NO.40 (V_i) IN - CC	10	10
3	FINAL VOLUME OF THE SOIL SAMPLE PASSING SIEVE NO.40 AFTER 24HR SOAKING IN 100CC OF DEMINERALIZED/DISTLLED WATER (V_F) IN CC	15	16
4	**FREE SWELL VOLUME = (V_F - V_I)/V_I**	**50**	**60**
		55	

Annex E: Swell-Consolidation Test Results

Table E-1: Swell-Consolidation Test for ES

[A] In the beginning of the test

Sample type :	Remolded
Ring Area,cm^2:	19.64
Height of sample, mm:	20
Seating Load,Kpa	7
Initial Void Ratio, e_o:	1.14
Initial moisture content,%	38.64
Specific Gravity:	2.73
Wet density/cm3	1.85

[B] In the end of the test

Final Moisture Content,%	38.57
Dry specimen wt (m_s), gm:	57.82
Dry density/cm3	1.48
Height of Solids(H_s), mm	10.78
Final Void Ratio, e_f:	0.91

[C]Calculation table:

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, E
Loading					
7	9.200	0.00	20.00	9.22	0.85
7	12.280	3.08	23.08	12.30	1.14
50	10.458	1.26	21.26	10.47	0.97
100	10.260	1.06	21.06	10.28	0.95
200	9.808	0.61	20.61	9.82	0.91
400	9.100	-0.10	19.90	9.12	0.85
800	6.794	-2.41	17.59	6.81	0.63

Swelling potential & Pressure

Initial Dial Reading		**9.2**
Final Dial Reading		**12.28**
Swell , [%]		**15.40**
Swelling pressure,kpa		**400.00**

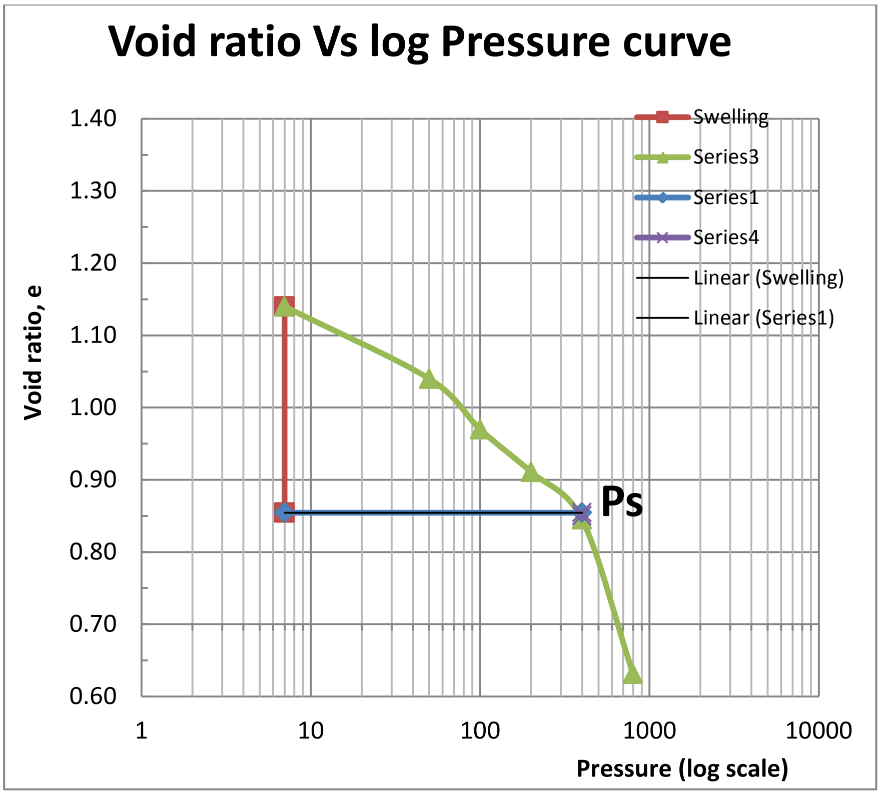

Figure E-1: Void Ratio Vs log Pressure curve for ES

Table E-2: Swell-Consolidation Test for 20%CS+80%ES

[A] In the beginning of the test

Sample type :	
Ring Area,cm^2:	19.6375
Height of sample,mm:	20
Seating Load,Kpa	7
Initial Void Ratio, e_o:	0.92
Initial moisture content,%	37.10
Specific Gravity:	2.60
Wet density,g/cm3	1.96

[B] In the end of the test

Final Moisture Content,%	40.71
Dry specimen wt (m_s), gm:	55.65
Dry density,g/cm3	1.37
Height of Solids(H_s), mm	10.90
Final Void Ratio, e_f:	0.814

[C]Calculation table:

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, E
Loading					
7	5.188	0.00	20.00	9.10	0.835
7	6.120	0.93	20.93	10.03	0.921
50	5.876	0.69	20.69	9.79	0.898
100	5.534	0.35	20.35	9.45	0.867
200	4.956	-0.23	19.77	8.87	0.814

Swelling potential & Pressure

Initial Dial Reading		**5.188**
Final Dial Reading		**6.12**
Swell , [%]		**4.66**
Swelling pressure,kpa		**161.00**

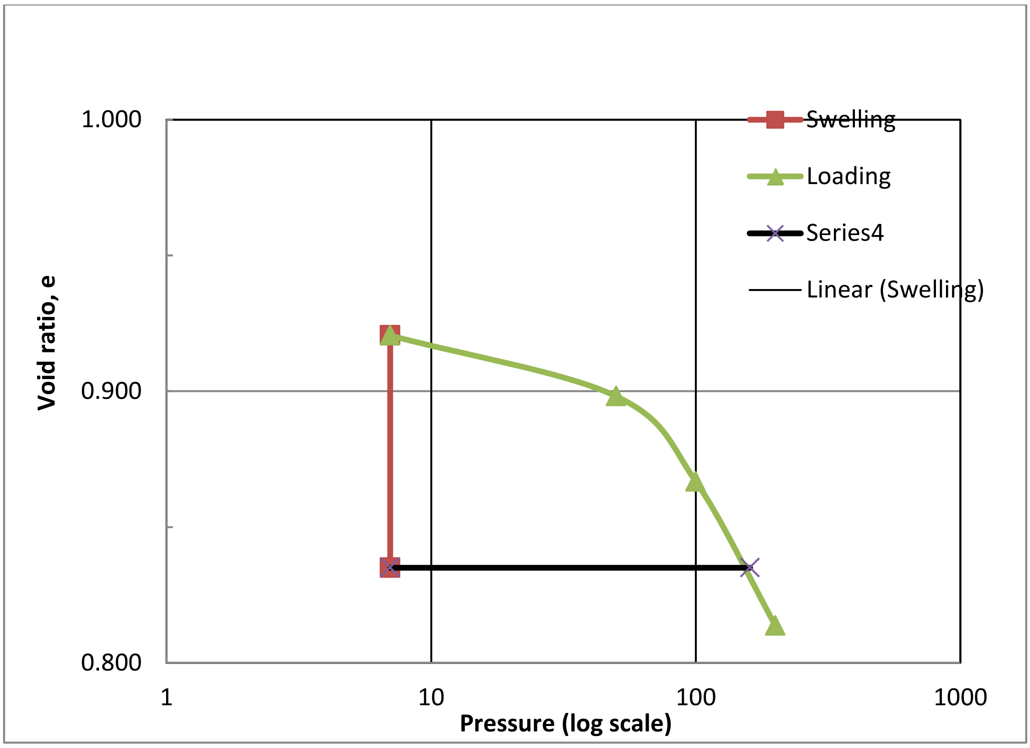

Figure E-2: Void Ratio Vs log Pressure curve for 20%CS+80%ES

Table E-3: Swell-Consolidation Test for 30%CS+70%ES

[A] In the beginning of the test

Sample type :	
Ring Area,cm^2:	19.6375
Height of sample,mm:	20
Seating Load,Kpa	7
Initial Void Ratio, e_o:	0.80
Initial moisture content,%	34.58
Specific Gravity:	2.61
Wet density,g/cm3	1.99

[B] In the end of the test

Final Moisture Content,%	37.10
Dry specimen wt (m_s), gm:	59.01
Dry density,g/cm3	1.46
Height of Solids(H_s), mm	11.54
Final Void Ratio, e_f:	0.669

[C]Calculation table:

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, E
Loading					
7	5.320	0.00	20.00	8.46	0.734
7	6.130	0.81	20.81	9.27	0.804
50	5.890	0.57	20.57	9.03	0.783
100	5.590	0.27	20.27	8.73	0.757
200	4.570	-0.75	19.25	7.71	0.669

Swelling potential & Pressure

Initial Dial Reading		**5.32**
Final Dial Reading		**6.13**
Swell , [%]		**4.05**
Swelling pressure,kpa		**126.47**

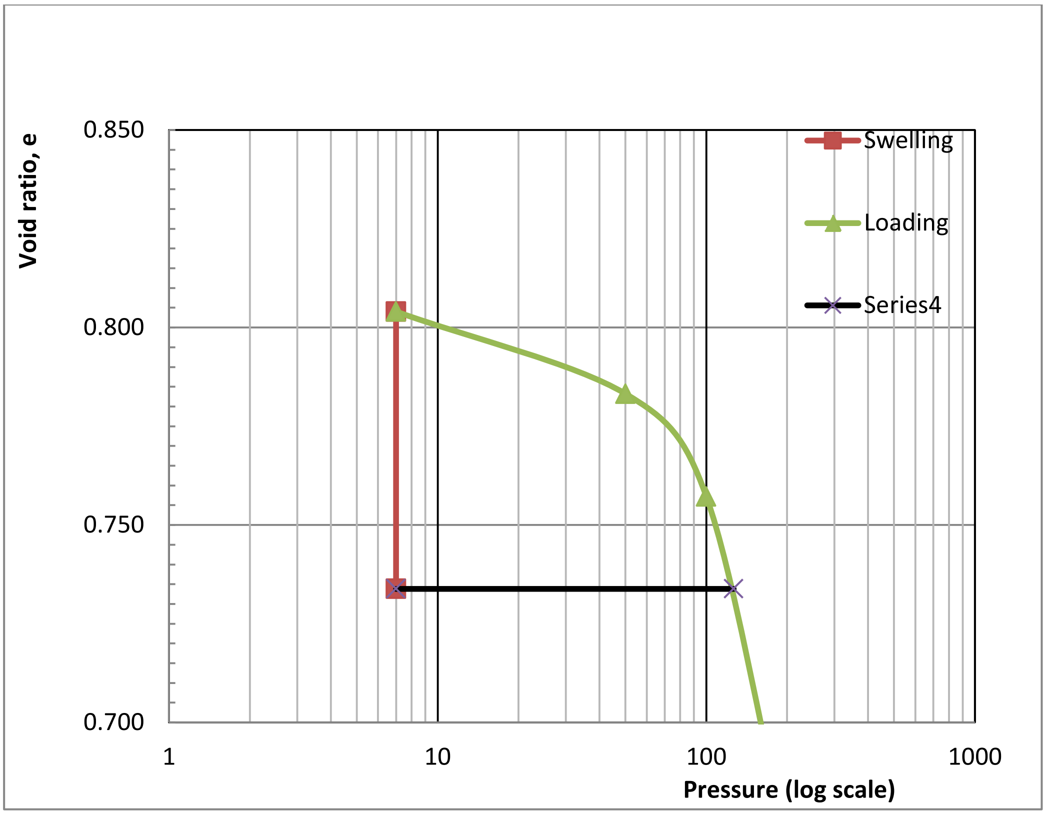

Figure E-3: Void Ratio Vs log Pressure curve for 30%CS+70%ES

Table E-4: Swell-Consolidation Test for 40%CS+60%ES

[A] In the beginning of the test

Sample type :	
Ring Area,cm^2:	19.6375
Height of sample,mm:	20
Seating Load,Kpa	7
Initial Void Ratio, e_o:	0.89
Initial moisture content,%	25.03
Specific Gravity:	2.61
Wet density,g/cm3	1.97

[B] In the end of the test

Final Moisture Content,%	37.42
Dry specimen wt (m_s), gm:	56.32
Dry density,g/cm3	1.40
Height of Solids(H_s), mm	10.99
Final Void Ratio, e_f:	0.81

[C]Calculation table:

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, E
Loading					
7	2.660	0.00	20.00	9.01	0.820
7	3.452	0.79	20.79	9.80	0.892
50	3.148	0.49	20.49	9.50	0.864
100	2.530	-0.13	19.87	8.88	0.808
200	0.000				
400					

Swelling potential & Pressure

Initial Dial Reading		**2.66**
Final Dial Reading		**3.45**
Swell , [%]		**3.96**
Swelling pressure,kpa		**89.48**

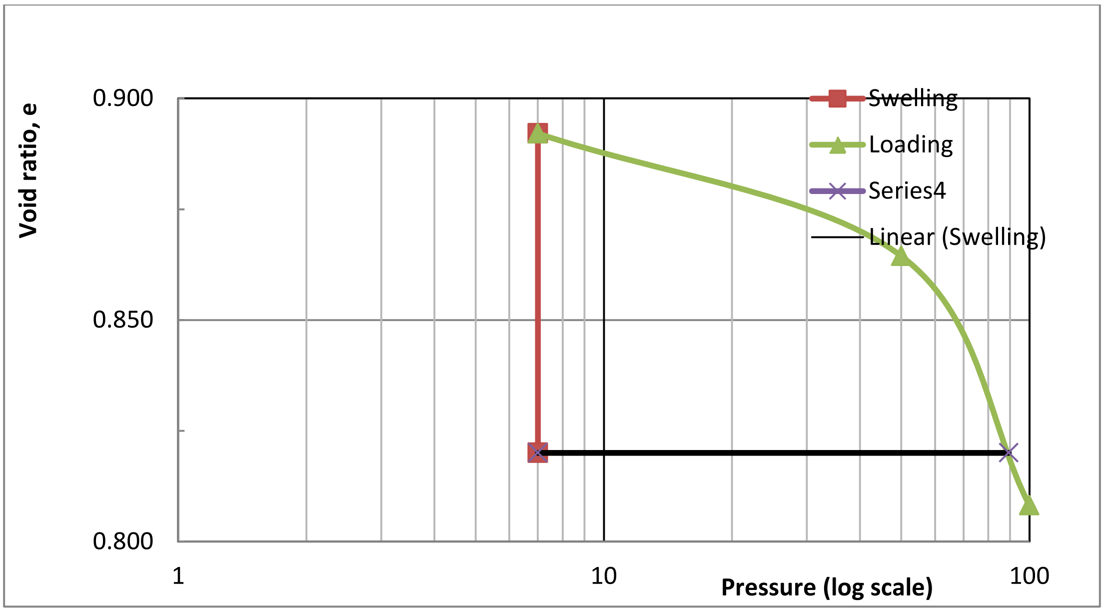

Figure E-4: Void Ratio Vs log Pressure curve for 40%CS+60%ES

Table E-5: Swell-Consolidation Test for 50%CS+50%ES

[A] In the beginning of the test

Sample type :	
Ring Area,cm^2:	19.6375
Height of sample,mm:	20
Seating Load,Kpa	7
Initial Void Ratio, e_o:	0.71
Initial moisture content,%	21.78
Specific Gravity:	2.62
Wet density,g/cm3	2.04

[B] In the end of the test

Final Moisture Content,%	30.71
Dry specimen wt (m_s), gm:	62.35
Dry density,g/cm3	1.58
Height of Solids(H_s), mm	12.14
Final Void Ratio, e_f:	0.597

[C]Calculation table:

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, E
Loading					
7	1.130	0.00	20.00	7.86	0.647
7	1.914	0.78	20.78	8.64	0.712
50	1.268	0.14	20.14	8.00	0.658
100	0.516	-0.61	19.39	7.24	0.597

Swelling potential & Pressure

Initial Dial Reading		**1.13**
Final Dial Reading		**1.91**
Swell , [%]		**3.92**
Swelling pressure,kpa		**66.67**

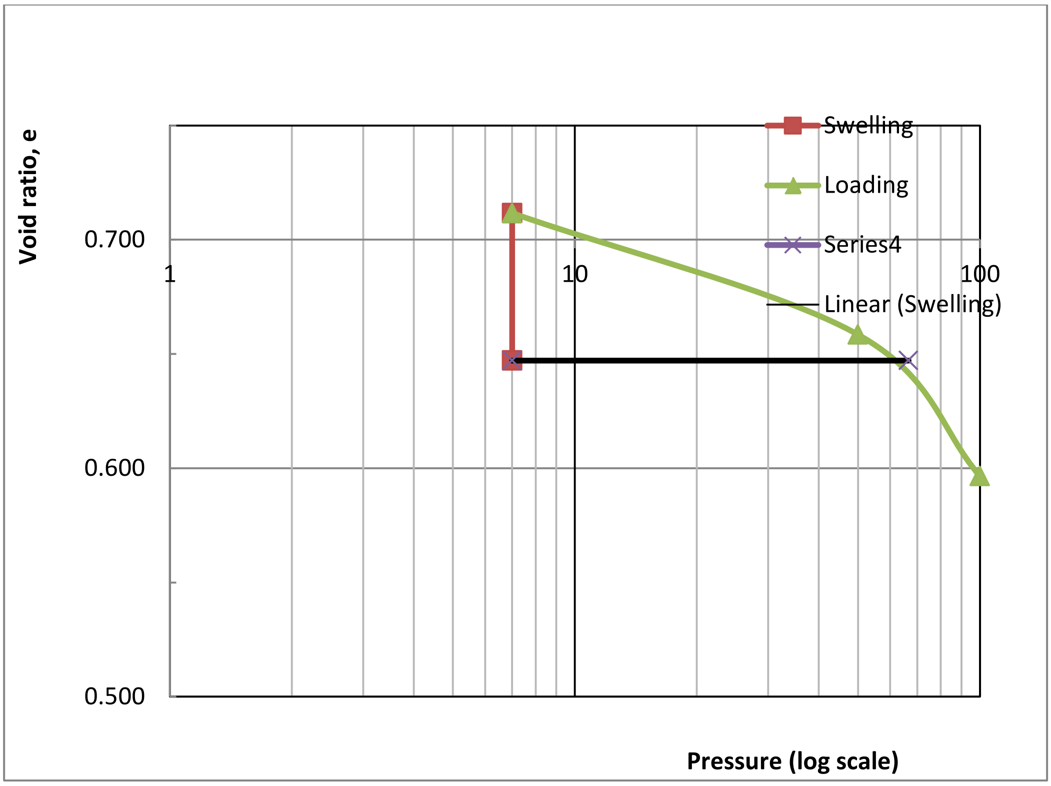

Figure E-5: Void Ratio Vs log Pressure curve for 50%CS+50%ES

Table E-6: Swell-Consolidation Test for 20%NS+80%ES

[A] In the beginning of the test

Sample type :	
Ring Area,cm^2:	19.6375
Height of sample,mm:	20
Seating Load,Kpa	7
Initial Void Ratio, e_o:	0.89
Initial moisture content,%	25.03
Specific Gravity:	2.58
Wet density,g/cm3	2.03

[B] In the end of the test

Final Moisture Content,%	43.64
Dry specimen wt (m_s), gm:	55.49
Dry density,g/cm3	1.39
Height of Solids(H_s), mm	10.97
Final Void Ratio, e_f:	0.822

[C]Calculation table:

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, E
Loading					
7	4.920	0.00	20.00	9.03	0.823
7	5.690	0.77	20.77	9.80	0.894
50	5.312	0.39	20.39	9.42	0.859
100	4.900	-0.02	19.98	9.01	0.822

Swelling potential & Pressure

Initial Dial Reading		**4.92**
Final Dial Reading		**5.69**
Swell , [%]		**3.85**
Swelling pressure,kpa		**98.65**

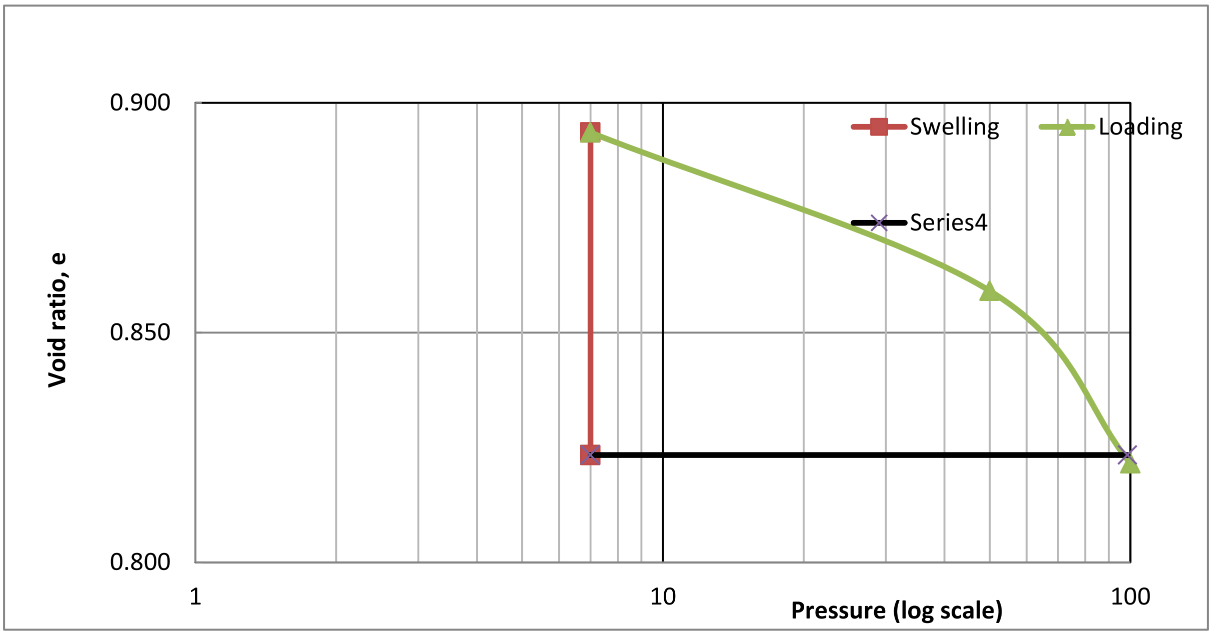

Figure E-6: Void Ratio Vs log Pressure curve for 20%NS+80%ES

Table E-7: Swell-Consolidation Test for 30%NS+70%ES

[A] In the beginning of the test

Sample type :	
Ring Area,cm^2:	19.6375
Height of sample,mm:	20
Seating Load,Kpa	7
Initial Void Ratio, e_o:	0.78
Initial moisture content,%	26.44
Specific Gravity:	2.57
Wet density,g/cm3	2.06

[B] In the end of the test

Final Moisture Content,%	38.74
Dry specimen wt (m_s), gm:	58.24
Dry density,g/cm3	1.46
Height of Solids(H_s), mm	11.54
Final Void Ratio, e_f:	0.728

[C]Calculation table:

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, E
Loading					
7	4.950	0.00	20.00	8.46	0.733
7	5.530	0.58	20.58	9.04	0.783
50	5.210	0.26	20.26	8.72	0.755
100	4.900	-0.05	19.95	8.41	0.728

Swelling potential & Pressure

Initial Dial Reading		**4.95**
Final Dial Reading		**5.53**
Swell , [%]		**2.90**
Swelling pressure,kpa		**83.87**

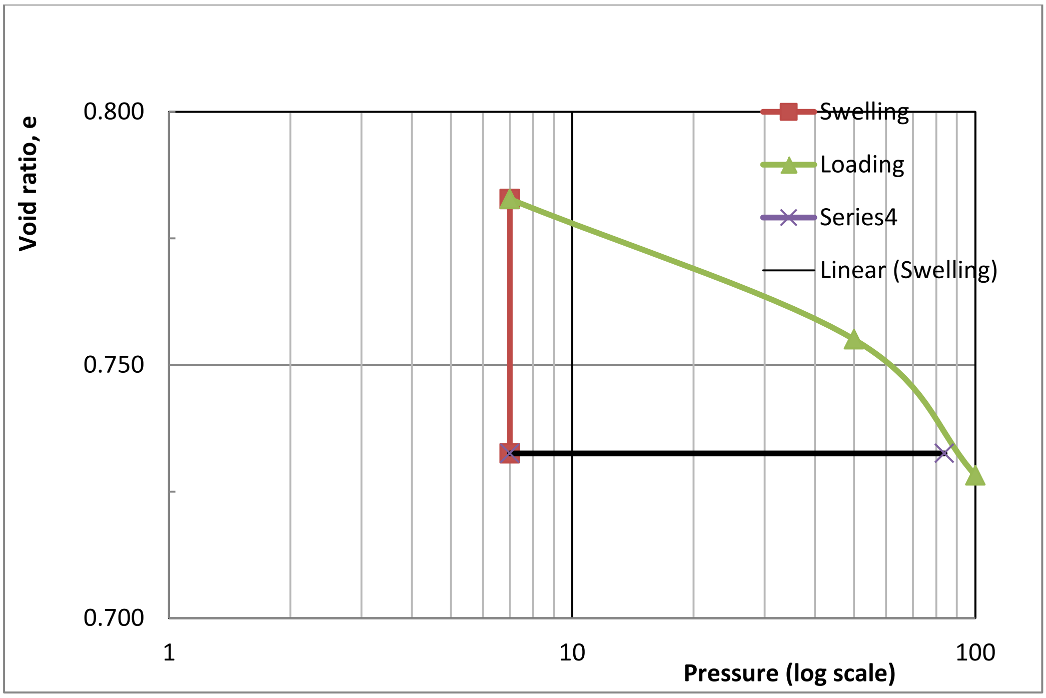

Figure E-7: Void Ratio Vs log Pressure curve for 30%NS+70%ES

Table E-8: Swell-Consolidation Test for 40%NS+60%ES

[A] In the beginning of the test

Sample type :	
Ring Area,cm^2:	19.6375
Height of sample,mm:	20
Seating Load,Kpa	7
Initial Void Ratio, e_o:	0.84
Initial moisture content,%	30.22
Specific Gravity:	**2.56**
Wet density,g/cm3	1.91

[B] In the end of the test

Final Moisture Content,%	35.11
Dry specimen wt (m_s), gm:	55.58
Dry density,g/cm3	1.41
Height of Solids(H_s), mm	11.05
Final Void Ratio, e_f:	0.792

[C]Calculation table:

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, E
Loading					
7	3.552	0.00	20.00	8.95	0.810
7	3.880	0.33	20.33	9.28	0.840
50	3.630	0.08	20.08	9.03	0.817
100	3.350	-0.20	19.80	8.75	0.792

Swelling potential & Pressure

Initial Dial Reading		**3.552**
Final Dial Reading		**3.88**
Swell , [%]		**1.64**
Swelling pressure,kpa		**63.93**

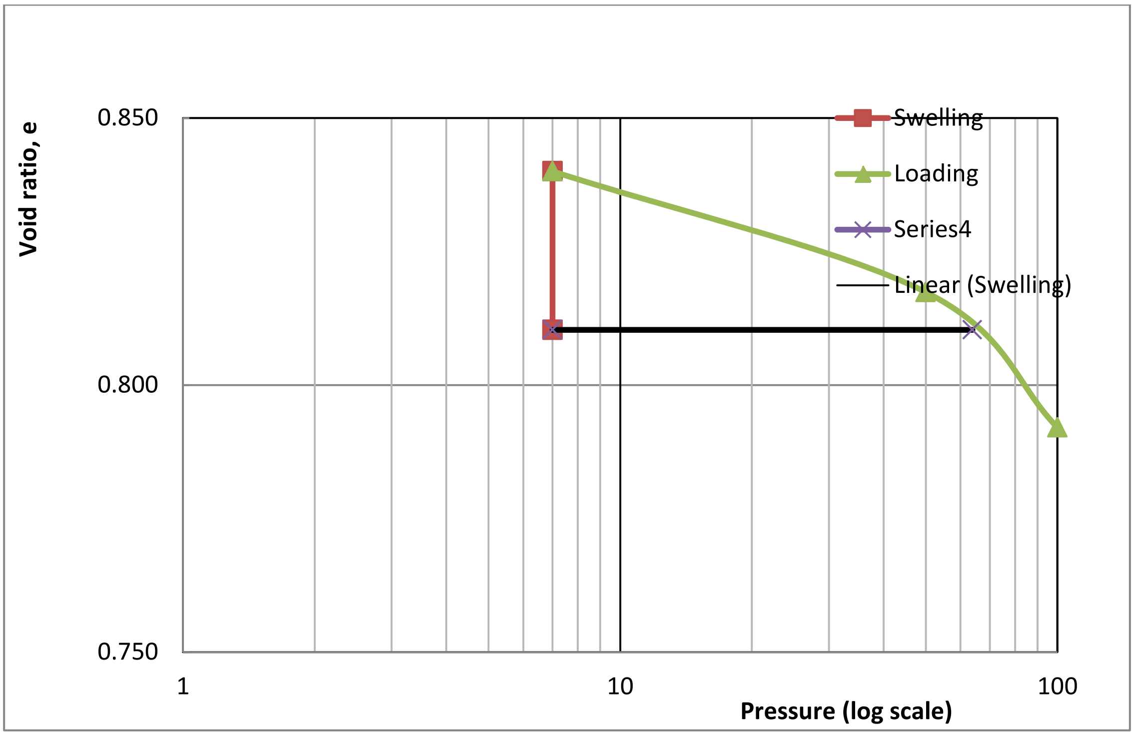

Figure E-8: Void Ratio Vs log Pressure curve for 40%NS+60%ES

Table E-9: Swell-Consolidation Test for 50%NS+50%ES

[A] In the beginning of the test

Sample type :	
Ring Area,cm^2:	19.6375
Height of sample,mm:	20
Seating Load,Kpa	7
Initial Void Ratio, e_o:	0.63
Initial moisture content,%	26.32
Specific Gravity:	2.56
Wet density,g/cm3	2.03

[B] In the end of the test

Final Moisture Content,%	31.43
Dry specimen wt (m_s), gm:	62.16
Dry density,g/cm3	1.59
Height of Solids(H_s), mm	12.39
Final Void Ratio, e_f:	0.606

[C]Calculation table:

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, E
Loading					
7	3.800	0.00	20.00	7.61	0.614
7	4.012	0.21	20.21	7.82	0.631
50	3.700	-0.10	19.90	7.51	0.606
100	3.500	-0.30	19.70	7.31	0.590

Swelling potential & Pressure

Initial Dial Reading		**3.8**
Final Dial Reading		**4.01**
Swell , [%]		**1.06**
Swelling pressure,kpa		**36.24**

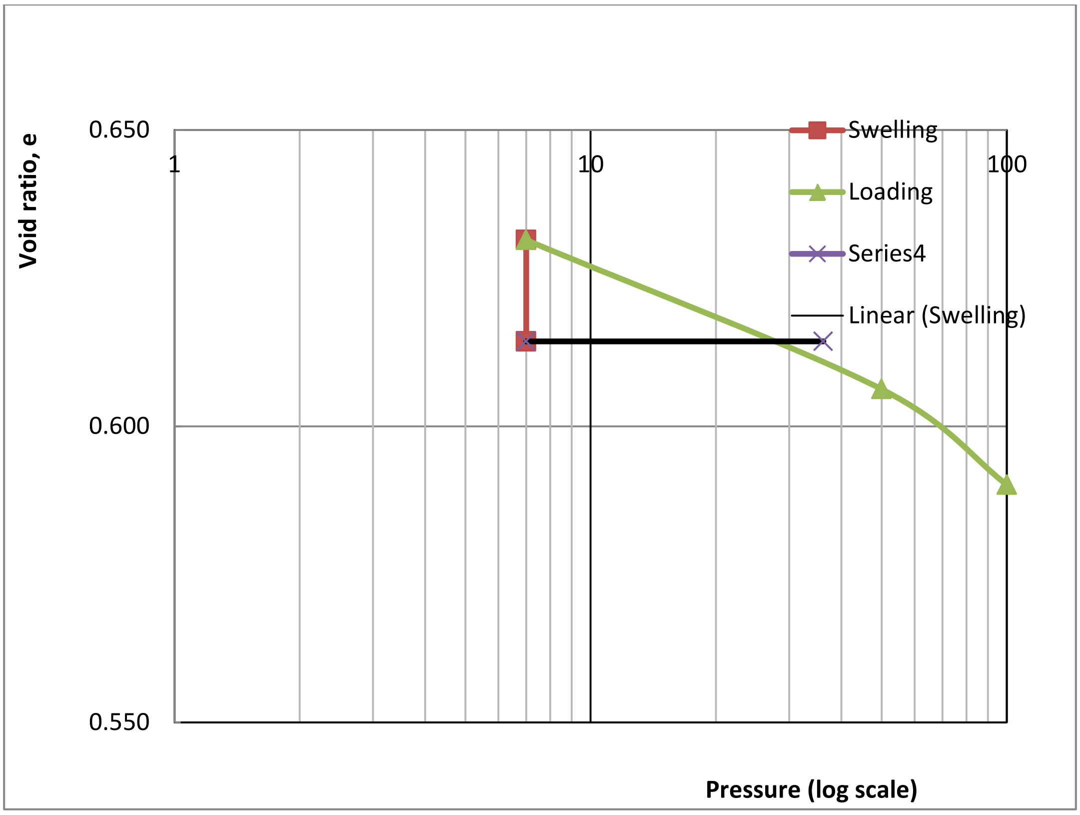

Figure E-9: Void Ratio Vs log Pressure curve for 50%NS+50%ES

Annex F: Unconfined Compression Strength Test Results

Table F-1: Unconfined Compression Strength Test for ES

Diameter of the test specimen (d) - cm :		
Initial length of the test specimen (L_o) - cm:	at 15% strain , ΔL	11.4
Weight of Soil Sample (gm) :		

A) MOISTURE CONTENT DETERMINATION

		SPECIMEN NUMBER	1
	1	MOISTURE CAN AND LID NUMBER(TOP/SIDE)	p-1,l-1,s2
	2	M_C=MASS OF EMPTY,CLEAN CAN + LID (M_C) (gram)	22.5
	3	M_{CMS}=MASS OF CAN,LID, AND MOIST SOIL (M_{CMS})(gram)	46.6
	4	M_{CDS}=MASS OF CAN,LID, AND DRY SOIL (M_{CDS})(gram)	45.9
	5	M_S=MASS OF SOIL SOLIDS (M_S)(gram)=M_{CDS}-M_C	23.4
	6	M_W=MASS OF PORE WATER(M_W) (gram) = M_{CDS}-M_{Cms}	0.70
	7	WATER CONTENT,W% = M_W/M_S	2.99

Initial Area of the test specimen (A_o) - cm^2:	11.341
Initial Volume of the test specimen (V_o) - cm^3:	86.192
Wet/Bulk density (g/cm^3) :	1.744
Water Content (W %):	2.99
Dry density(g/cm3):	1.693

B) UNCONFINED COMPRESSION TEST DATA (Deformation Dial: 1 unit = 0.010mm; Load Dial: .00142kN/Div)

DEFORMATION DIAL READING	LOAD DIAL READING	SAMPLE DEFORMATION,ΔL (mm)	STRAIN ($\varepsilon=\Delta L/L_O$)	% STRAIN(ε*100)	CORRECTED AREA,($A'=A_0/1-\varepsilon$)	LOAD (P) (KN)	STRESS ($\sigma_c=P/A'$)(kpa)
0	0.0	0.000	0.000	0.000	11.341	0.000	0.000
20	5.0	0.200	0.003	0.263	11.371	0.007	6.244
40	17.0	0.400	0.005	0.526	11.401	0.024	21.174
60	30.0	0.600	0.008	0.789	11.431	0.043	37.266
80	43.0	0.800	0.011	1.053	11.462	0.061	53.273
100	55.0	1.000	0.013	1.316	11.492	0.078	67.959
120	68.0	1.200	0.016	1.579	11.523	0.097	83.798
140	78.0	1.400	0.018	1.842	11.554	0.111	95.864
160	88.0	1.600	0.021	2.105	11.585	0.125	107.865
180	97.0	1.800	0.024	2.368	11.616	0.138	118.577
200	105.0	2.000	0.026	2.632	11.648	0.149	128.010
220	108.0	2.200	0.029	2.895	11.679	0.153	131.312
240	114.0	2.400	0.032	3.158	11.711	0.162	138.231
260	116.0	2.600	0.034	3.421	11.743	0.165	140.274
280	117.0	2.800	0.037	3.684	11.775	0.166	141.098
300	116.0	3.000	0.039	3.947	11.807	0.165	139.510
320	115.0	3.200	0.042	4.211	11.840	0.163	137.928
340	112.0	3.400	0.045	4.474	11.872	0.159	133.961
360	108.0	3.600	0.047	4.737	11.905	0.153	128.821
380	98.0	3.800	0.050	5.000	11.938	0.139	116.570
400	93.0	4.000	0.053	5.263	11.971	0.132	110.316

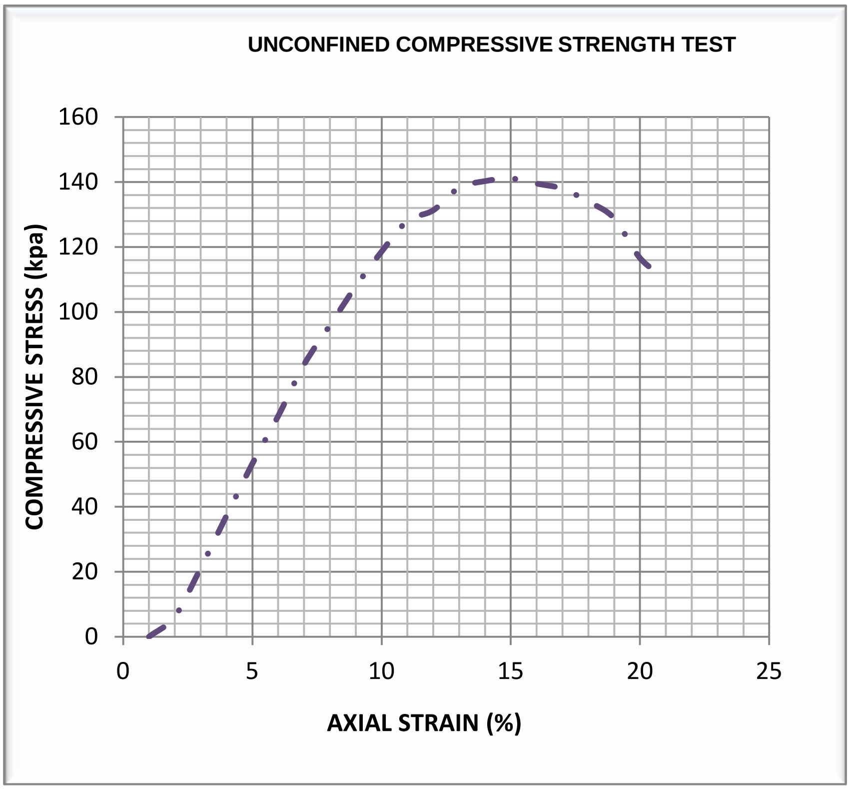

Figure F-1: Unconfined Compression Strength Curve for ES

Unconfined Compressive Strength (q_u) :	**142kpa**
Cohesion (C=qu/2) :	**71kpa**

Table F-2: Unconfined Compression Strength Test for 20%CS+80%ES

Diameter of the test specimen (d) - cm :

Initial length of the test specimen (L_o) - cm: at 15% strain , ΔL 11.1

Weight of Soil Sample (gm) :

A) MOISTURE CONTENT DETERMINATION

	SPECIMEN NUMBER	1
1	MOISTURE CAN AND LID NUMBER(TOP/SIDE)	1'
2	M_C=MASS OF EMPTY,CLEAN CAN + LID (M_C) (gram)	22.5
3	M_{CMS}=MASS OF CAN,LID, AND MOIST SOIL (M_{CMS})(gram)	37.8
4	M_{CDS}=MASS OF CAN,LID, AND DRY SOIL (M_{CDS})(gram)	34
5	M_S=MASS OF SOIL SOLIDS (M_S)(gram)=M_{CDS}-M_C	11.5
6	M_W=MASS OF PORE WATER(M_W) (gram) = M_{CDS}-M_{Cms}	3.80
7	WATER CONTENT,W% = M_W/M_S	33.04

Initial Area of the test specimen (A_o) - cm^2: 11.341

Initial Volume of the test specimen (V_o) - cm^3: 83.923

Wet/Bulk density (g/cm^3) : 1.681

Water Content (W %): 33.04

Dry density(g/cm3): 1.263

B) UNCONFINED COMPRESSION TEST DATA (Deformation Dial: 1 unit = 0.010mm; Load Dial: .00142kN/Div)

DEFORMATION DIAL READING	LOAD DIAL READING	SAMPLE DEFORMATION, ΔL(mm)	STRAIN ($\varepsilon=\Delta L/L_O$)	% STRAIN(ε*100)	CORRECTED AREA,($A'=A_0/1-\varepsilon$)	LOAD (P) (KN)	STRESS ($\sigma_c=P/A'$)(kpa)
0	0.0	0.000	0.000	0.000	11.341	0.000	0.000
20	9.0	0.200	0.003	0.270	11.372	0.013	11.238
40	15.0	0.400	0.005	0.541	11.403	0.021	18.680
60	23.0	0.600	0.008	0.811	11.434	0.033	28.565
80	31.0	0.800	0.011	1.081	11.465	0.044	38.395
100	40.0	1.000	0.014	1.351	11.496	0.057	49.407
120	50.0	1.200	0.016	1.622	11.528	0.071	61.589
140	61.0	1.400	0.019	1.892	11.560	0.087	74.933
160	70.0	1.600	0.022	2.162	11.592	0.099	85.752
180	77.0	1.800	0.024	2.432	11.624	0.109	94.066
200	84.0	2.000	0.027	2.703	11.656	0.119	102.333
220	92.0	2.200	0.030	2.973	11.688	0.131	111.768
240	99.0	2.400	0.032	3.243	11.721	0.141	119.937
260	105.0	2.600	0.035	3.514	11.754	0.149	126.851
280	110.0	2.800	0.038	3.784	11.787	0.156	132.519
300	118.0	3.000	0.041	4.054	11.820	0.168	141.757
320	123.0	3.200	0.043	4.324	11.854	0.175	147.348
340	129.0	3.400	0.046	4.595	11.887	0.183	154.099
360	127.0	3.600	0.049	4.865	11.921	0.180	151.280
380	124.0	3.800	0.051	5.135	11.955	0.176	147.287
400	120.0	4.000	0.054	5.405	11.989	0.170	142.130
420	116.0	4.200	0.057	5.676	12.023	0.165	136.999
440	112.0	4.400	0.059	5.946	12.058	0.159	131.896

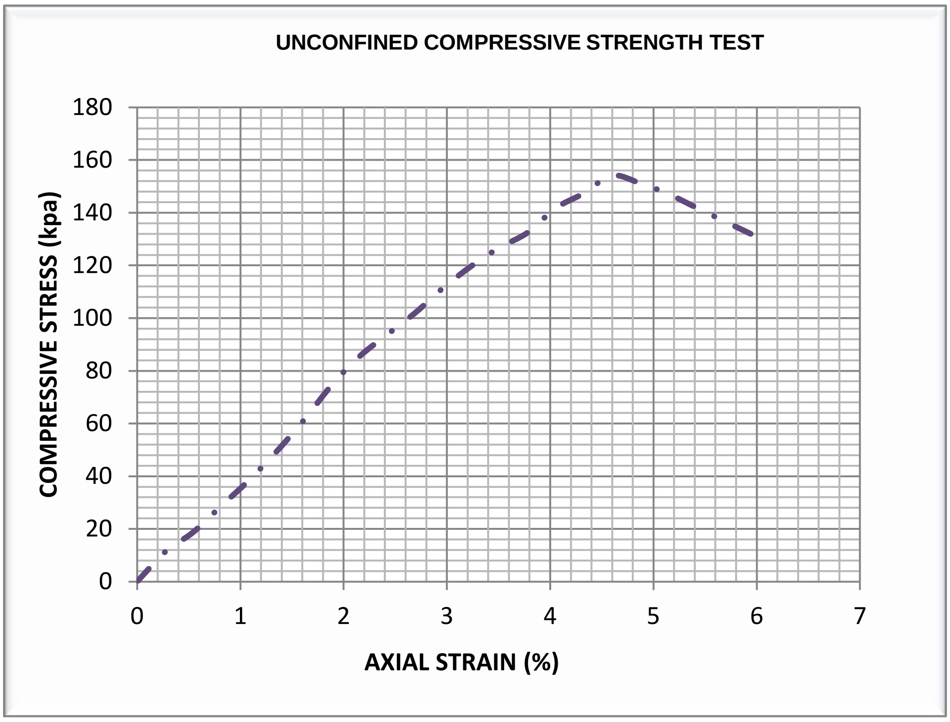

Figure F-2: Unconfined Compression Strength curve for 20%CS+80%ES

Unconfined Compressive Strength (q_u) :	**154kpa**
Cohesion (C=qu/2) :	**77kpa**

Table F-3: Unconfined Compression Strength Test for 30%CS+70%ES

Diameter of the test specimen (d) - cm : 3.80

Initial length of the test specimen (L_o) - cm: 7.60 at 15% strain , ΔL 11

Weight of Soil Sample (gm) : 159.3

A) MOISTURE CONTENT DETERMINATION

	SPECIMEN NUMBER	1
1	MOISTURE CAN AND LID NUMBER(TOP/SIDE)	85
2	M_C=MASS OF EMPTY,CLEAN CAN + LID (M_C) (gram)	22
3	M_{CMS}=MASS OF CAN,LID, AND MOIST SOIL (M_{CMS})(gram)	58.1
4	M_{CDS}=MASS OF CAN,LID, AND DRY SOIL (M_{CDS})(gram)	50.1
5	M_S=MASS OF SOIL SOLIDS (M_S)(gram)=M_{CDS}-M_C	28.1
6	M_W=MASS OF PORE WATER(M_W) (gram) = M_{CDS}-M_{Cms}	8.00
7	WATER CONTENT,W% = M_W/M_S	28.47

Initial Area of the test specimen (A_o) - cm^2: 11.341

Initial Volume of the test specimen (V_o) - cm^3: 86.192

Wet/Bulk density (g/cm^3) : 1.848

Water Content (W %): 28.47

Dry density(g/cm3): 1.438

B) UNCONFINED COMPRESSION TEST DATA (Deformation Dial: 1 unit = 0.010mm; Load Dial: .00142kN/Div)

DEFORMATION DIAL READING	LOAD DIAL READING	SAMPLE DEFORMATION,ΔL(mm)	STRAIN (ε=ΔL/L_O)	% STRAIN(ε*100)	CORRECTED AREA,(A'=A_0/1-ε)	LOAD (P) (KN)	STRESS (σ_c=P/A')(kpa)
0	0.0	0.000	0.000	0.000	11.341	0.000	0.000
20	33.0	0.200	0.003	0.263	11.371	0.047	41.210
40	55.0	0.400	0.005	0.526	11.401	0.078	68.503
60	71.0	0.600	0.008	0.789	11.431	0.101	88.197
80	84.0	0.800	0.011	1.053	11.462	0.119	104.069
100	98.0	1.000	0.013	1.316	11.492	0.139	121.091
120	105.0	1.200	0.016	1.579	11.523	0.149	129.394
140	114.0	1.400	0.018	1.842	11.554	0.162	140.109
160	120.0	1.600	0.021	2.105	11.585	0.170	147.088
180	125.0	1.800	0.024	2.368	11.616	0.178	152.805
200	129.0	2.000	0.026	2.632	11.648	0.183	157.270
220	134.0	2.200	0.029	2.895	11.679	0.190	162.924
240	138.0	2.400	0.032	3.158	11.711	0.196	167.333
260	140.0	2.600	0.034	3.421	11.743	0.199	169.296
280	141.0	2.800	0.037	3.684	11.775	0.200	170.041
300	141.0	3.000	0.039	3.947	11.807	0.200	169.576
320	141.0	3.200	0.042	4.211	11.840	0.200	169.112
340	141.0	3.400	0.045	4.474	11.872	0.200	168.647
360	140.0	3.600	0.047	4.737	11.905	0.199	166.990
380	135.0	3.800	0.050	5.000	11.938	0.192	160.581
400	130.0	4.000	0.053	5.263	11.971	0.185	154.205

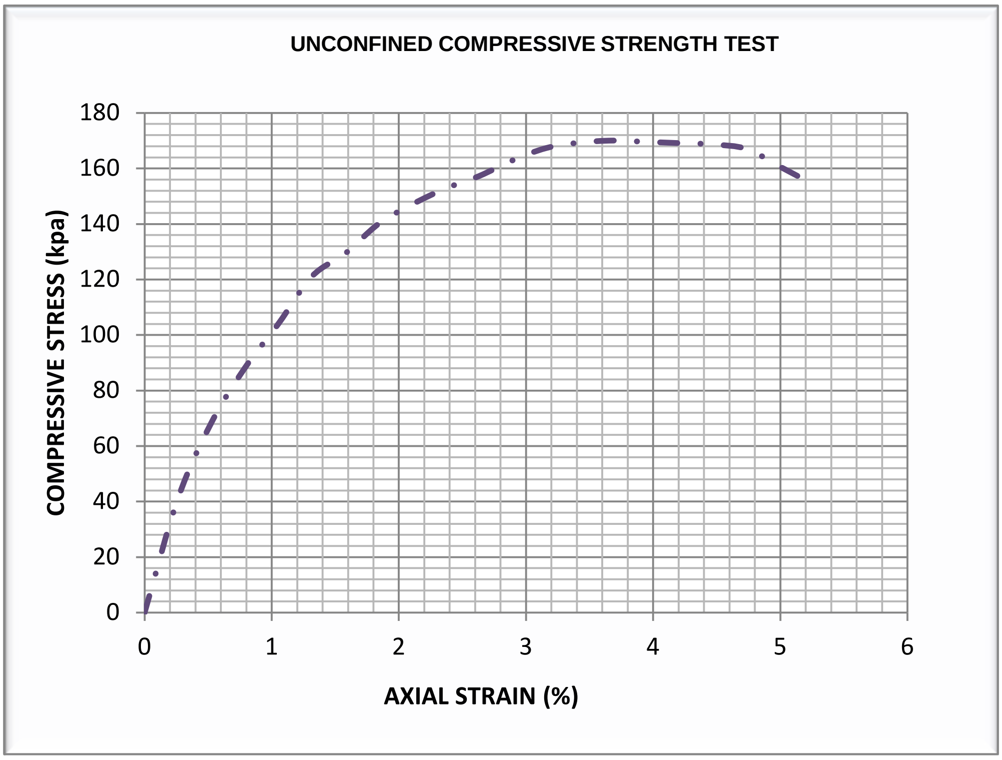

Figure F-3: Unconfined Compression Strength curve for 30%CS+70%ES

Unconfined Compressive Strength (q_u) :	**170kpa**
Cohesion (C=qu/2) :	**85kpa**

Table F-4: Unconfined Compression Strength Test for 40%CS+60%ES

Diameter of the test specimen (d) - cm : 3.80

Initial length of the test specimen (L_o) - cm: 7.60 at 15% strain , ΔL

Weight of Soil Sample (gm) : 166.2

A) MOISTURE CONTENT DETERMINATION

	SPECIMEN NUMBER	1
1	MOISTURE CAN AND LID NUMBER(TOP/SIDE)	22A
2	M_C=MASS OF EMPTY,CLEAN CAN + LID (M_C) (gram)	22.2
3	M_{CMS}=MASS OF CAN,LID, AND MOIST SOIL (M_{CMS})(gram)	69.4
4	M_{CDS}=MASS OF CAN,LID, AND DRY SOIL (M_{CDS})(gram)	61.1
5	M_S=MASS OF SOIL SOLIDS (M_S)(gram)=M_{CDS}-M_C	38.9
6	M_W=MASS OF PORE WATER(M_W) (gram) = M_{CDS}-M_{Cms}	8.30
7	WATER CONTENT,W% = M_W/M_S	21.34

Initial Area of the test specimen (A_o) - cm^2: 11.341

Initial Volume of the test specimen (V_o) - cm^3: 86.192

Wet/Bulk density (g/cm^3) : 1.928

Water Content (W %): 21.34

Dry density(g/cm3): 1.589

B) UNCONFINED COMPRESSION TEST DATA (Deformation Dial: 1 unit = 0.010mm; Load Dial: .00142kN/Div)

DEFORMATION DIAL READING	LOAD DIAL READING	SAMPLE DEFORMATION,ΔL(mm)	STRAIN ($\varepsilon=\Delta L/L_O$)	% STRAIN(ε*100)	CORRECTED AREA,($A'=A_0/1-\varepsilon$)	LOAD (P) (KN)	STRESS ($\sigma_c=P/A'$)(kpa)
0	0.0	0.000	0.000	0.000	11.341	0.000	0.000
20	25.0	0.200	0.003	0.263	11.371	0.036	31.220
40	38.0	0.400	0.005	0.526	11.401	0.054	47.329
60	49.0	0.600	0.008	0.789	11.431	0.070	60.868
80	62.0	0.800	0.011	1.053	11.462	0.088	76.813
100	75.0	1.000	0.013	1.316	11.492	0.107	92.671
120	87.0	1.200	0.016	1.579	11.523	0.124	107.212
140	98.0	1.400	0.018	1.842	11.554	0.139	120.445
160	109.0	1.600	0.021	2.105	11.585	0.155	133.605
180	124.0	1.800	0.024	2.368	11.616	0.176	151.582
200	138.0	2.000	0.026	2.632	11.648	0.196	168.242
220	154.0	2.200	0.029	2.895	11.679	0.219	187.241
240	165.0	2.400	0.032	3.158	11.711	0.234	200.071
260	174.0	2.600	0.034	3.421	11.743	0.247	210.411
280	182.0	2.800	0.037	3.684	11.775	0.258	219.486
300	188.0	3.000	0.039	3.947	11.807	0.267	226.102
320	190.0	3.200	0.042	4.211	11.840	0.270	227.881
340	190.0	3.400	0.045	4.474	11.872	0.270	227.255
360	188.0	3.600	0.047	4.737	11.905	0.267	224.243
380	184.0	3.800	0.050	5.000	11.938	0.261	218.866
400	175.0	4.000	0.053	5.263	11.971	0.249	207.584
420	162.0	4.200	0.055	5.526	12.004	0.230	191.630

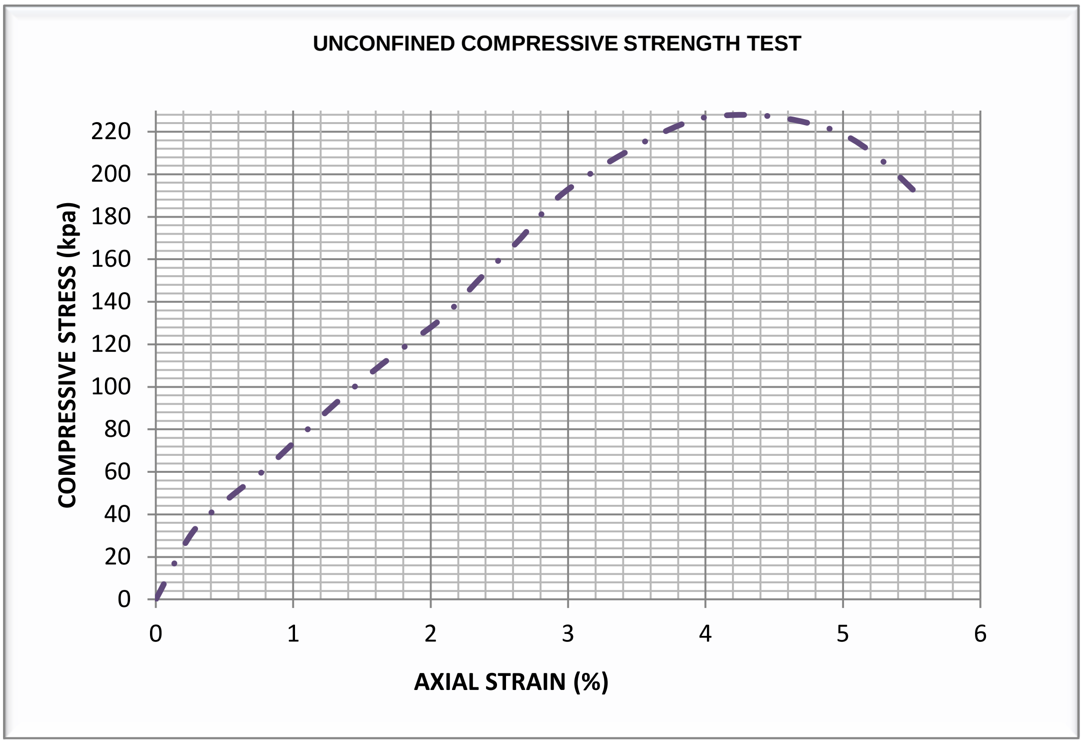

Figure F-4: Unconfined Compression Strength curve for 40%CS+60%ES

Unconfined Compressive Strength (q_u) :	**228kpa**
Cohesion (C=qu/2) :	**114kpa**

Table F-5: Unconfined Compression Strength Test for 50%CS+50%ES

Diameter of the test specimen (d) - cm : 3.80

Initial length of the test specimen (L_o) - cm: 7.80 at 15% strain , ΔL 11.7

Weight of Soil Sample (gm) : 167.8

A) MOISTURE CONTENT DETERMINATION

	SPECIMEN NUMBER	
1	MOISTURE CAN AND LID NUMBER(TOP/SIDE)	C1
2	M_C=MASS OF EMPTY,CLEAN CAN + LID (M_C) (gram)	21.9
3	M_{CMS}=MASS OF CAN,LID, AND MOIST SOIL (M_{CMS})(gram)	44.6
4	M_{CDS}=MASS OF CAN,LID, AND DRY SOIL (M_{CDS})(gram)	41.2
5	M_S=MASS OF SOIL SOLIDS (M_S)(gram)=M_{CDS}-M_C	19.3
6	M_W=MASS OF PORE WATER(M_W) (gram) = M_{CDS}-M_{Cms}	3.40
7	WATER CONTENT,W% = M_W/M_S	17.62

Initial Area of the test specimen (A_o) - cm^2: 11.341

Initial Volume of the test specimen (V_o) - cm^3: 88.46

Wet/Bulk density (g/cm^3) : 1.897

Water Content (W %): 17.62

Dry density(g/cm3): 1.613

B) UNCONFINED COMPRESSION TEST DATA (Deformation Dial: 1 unit = 0.010mm; Load Dial: .00142kN/Div)

DEFORMATION DIAL READING	LOAD DIAL READING	SAMPLE DEFORMATION, ΔL(mm)	STRAIN ($\varepsilon=\Delta L/L_O$)	% STRAIN(ε*100)	CORRECTED AREA,($A'=A_0/1-\varepsilon$)	LOAD (P) (KN)	STRESS ($\sigma_c=P/A'$)(kpa)
0	0.0	0.000	0.000	0.000	11.341	0.000	0.000
20	20.0	0.200	0.003	0.256	11.370	0.028	24.978
40	31.0	0.400	0.005	0.513	11.399	0.044	38.616
60	41.0	0.600	0.008	0.769	11.429	0.058	50.941
80	50.0	0.800	0.010	1.026	11.459	0.071	61.963
100	61.0	1.000	0.013	1.282	11.488	0.087	75.399
120	73.0	1.200	0.015	1.538	11.518	0.104	89.997
140	84.0	1.400	0.018	1.795	11.548	0.119	103.288
160	98.0	1.600	0.021	2.051	11.579	0.139	120.188
180	110.0	1.800	0.023	2.308	11.609	0.156	134.552
200	128.0	2.000	0.026	2.564	11.639	0.182	156.159
220	142.0	2.200	0.028	2.821	11.670	0.202	172.783
240	156.0	2.400	0.031	3.077	11.701	0.222	189.317
260	170.0	2.600	0.033	3.333	11.732	0.241	205.761
280	181.0	2.800	0.036	3.590	11.763	0.257	218.494
300	200.0	3.000	0.038	3.846	11.795	0.284	240.787
320	205.0	3.200	0.041	4.103	11.826	0.291	246.149
340	208.0	3.400	0.044	4.359	11.858	0.295	249.083
360	209.0	3.600	0.046	4.615	11.890	0.297	249.610
380	209.0	3.800	0.049	4.872	11.922	0.297	248.939
400	208.0	4.000	0.051	5.128	11.954	0.295	247.080
420	204.0	4.200	0.054	5.385	11.986	0.290	241.673
440	202.0	4.400	0.056	5.641	12.019	0.287	238.656
460	195.0	4.600	0.059	5.897	12.052	0.277	229.759

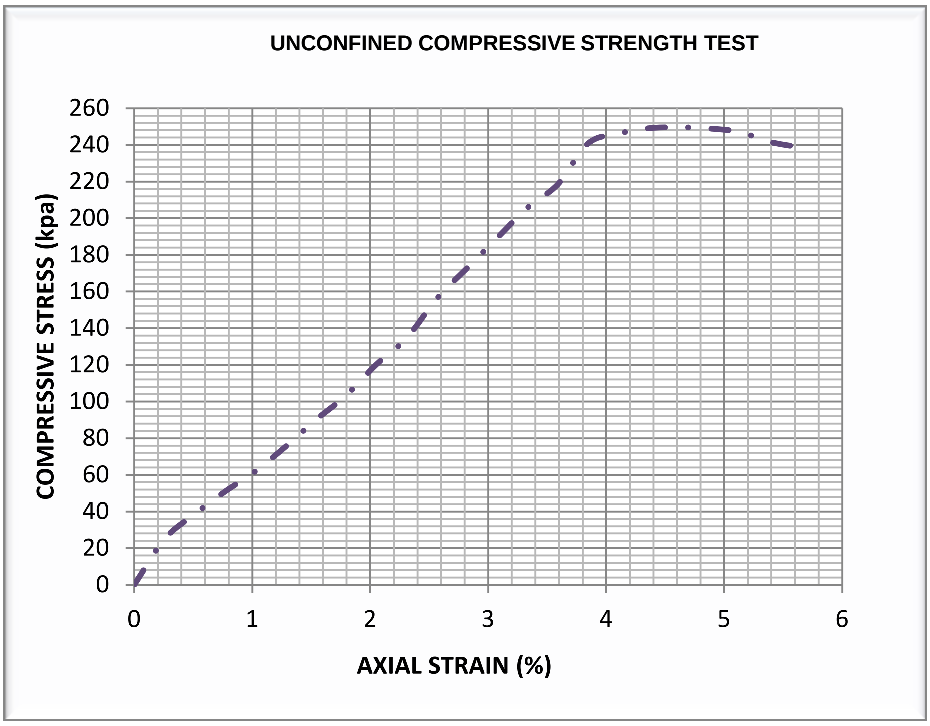

Figure F-5: Unconfined Compression Strength curve for 50%CS+50%ES

Unconfined Compressive Strength (q_u) :	**250kpa**
Cohesion (C=qu/2) :	**125kpa**

Table F-6: Unconfined Compression Strength Test for 20%NS+80%ES

Diameter of the test specimen (d) - cm : 3.50

Initial length of the test specimen (L_o) - cm: 8.30 at 15% strain , ΔL 12.45

Weight of Soil Sample (gm) : 157.1

A) MOISTURE CONTENT DETERMINATION

	SPECIMEN NUMBER	1
1	MOISTURE CAN AND LID NUMBER(TOP/SIDE)	p-1,l-1,s2
2	M_C=MASS OF EMPTY,CLEAN CAN + LID (M_C) (gram)	22
3	M_{CMS}=MASS OF CAN,LID, AND MOIST SOIL (M_{CMS})(gram)	40.7
4	M_{CDS}=MASS OF CAN,LID, AND DRY SOIL (M_{CDS})(gram)	35.8
5	M_S=MASS OF SOIL SOLIDS (M_S)(gram)=M_{CDS}-M_C	13.8
6	M_W=MASS OF PORE WATER(M_W) (gram) = M_{CDS}-M_{Cms}	4.90
7	WATER CONTENT,W% = M_W/M_S	35.51

Initial Area of the test specimen (A_o) - cm^2: 9.621

Initial Volume of the test specimen (V_o) - cm^3: 79.854

Wet/Bulk density (g/cm^3) : 1.967

Water Content (W %): 35.51

Dry density(g/cm3): 1.452

B) UNCONFINED COMPRESSION TEST DATA (Deformation Dial: 1 unit = 0.010mm; Load Dial: .00142kN/Div)

DEFORMATION DIAL READING	LOAD DIAL READING	SAMPLE DEFORMATION ,ΔL(mm)	STRAIN ($\varepsilon=\Delta L/L_O$)	% STRAIN(ε* 100)	CORRECTED AREA,($A'=A_0/1-\varepsilon$)	LOAD (P) (KN)	STRESS ($\sigma_c=P/A'$)(kpa)
0	0.0	0.000	0.000	0.000	9.621	0.000	0.000
20	40.0	0.200	0.002	0.241	9.644	0.057	58.895
40	64.0	0.400	0.005	0.482	9.668	0.091	94.005
60	82.0	0.600	0.007	0.723	9.691	0.116	120.152
80	93.0	0.800	0.010	0.964	9.715	0.132	135.939
100	100.0	1.000	0.012	1.205	9.738	0.142	145.816
120	105.0	1.200	0.014	1.446	9.762	0.149	152.733
140	110.0	1.400	0.017	1.687	9.786	0.156	159.615
160	113.0	1.600	0.019	1.928	9.810	0.160	163.566
180	115.0	1.800	0.022	2.169	9.834	0.163	166.052
200	117.0	2.000	0.024	2.410	9.859	0.166	168.524
220	118.0	2.200	0.027	2.651	9.883	0.168	169.544
240	118.0	2.400	0.029	2.892	9.907	0.168	169.125
260	117.0	2.600	0.031	3.133	9.932	0.166	167.275
280	104.0	2.800	0.034	3.373	9.957	0.148	148.319
300	96.0	3.000	0.036	3.614	9.982	0.136	136.569
320	82.0	3.200	0.039	3.855	10.007	0.116	116.361

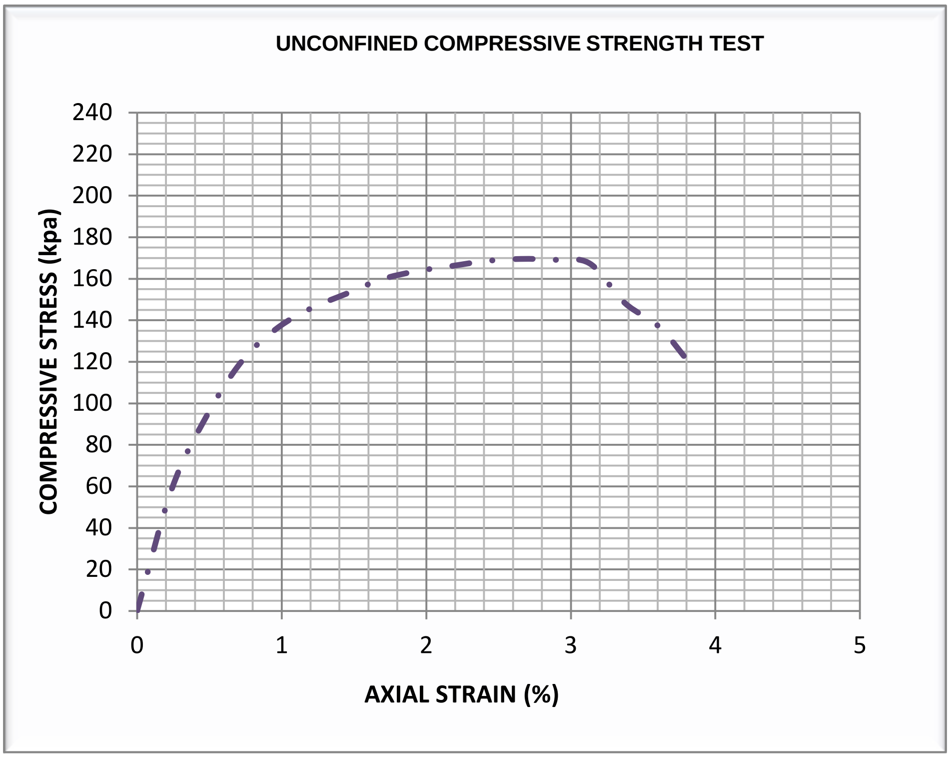

Figure F-6: Unconfined Compression Strength curve for 20%NS+80%ES

Unconfined Compressive Strength (q_u) :	**170kpa**
Cohesion (C=qu/2) :	**85kpa**

Table F-7: Unconfined Compression Strength Test for 30%NS+70%ES

Diameter of the test specimen (d) - cm : 3.80

Initial length of the test specimen (L_o) - cm: 7.60   at 15% strain , ΔL 1

Weight of Soil Sample (gm) : 137.5

A) MOISTURE CONTENT DETERMINATION

	SPECIMEN NUMBER	1
1	MOISTURE CAN AND LID NUMBER(TOP/SIDE)	p-1,l-1,s2
2	M_C=MASS OF EMPTY,CLEAN CAN + LID (M_C) (gram)	22.9
3	M_{CMS}=MASS OF CAN,LID, AND MOIST SOIL (M_{CMS})(gram)	49.9
4	M_{CDS}=MASS OF CAN,LID, AND DRY SOIL (M_{CDS})(gram)	45
5	M_S=MASS OF SOIL SOLIDS (M_S)(gram)=M_{CDS}-M_C	22.1
6	M_W=MASS OF PORE WATER(M_W) (gram) = M_{CDS}-M_{Cms}	4.90
7	WATER CONTENT,W% = M_W/M_S	22.17

Initial Area of the test specimen (A_o) - cm^2: 11.341

Initial Volume of the test specimen (V_o) - cm^3: 86.192

Wet/Bulk density (g/cm^3) : 1.595

Water Content (W %): 22.17

Dry density(g/cm3): 1.306

B) UNCONFINED COMPRESSION TEST DATA (Deformation Dial: 1 unit = 0.010mm; Load Dial: .00142kN/Div)

DEFORMATION DIAL READING	LOAD DIAL READING	SAMPLE DEFORMATION,ΔL(mm)	STRAIN ($\varepsilon=\Delta L/L_O$)	% STRAIN(ε*100)	CORRECTED AREA,($A'=A_0/1-\varepsilon$)	LOAD (P) (KN)	STRESS ($\sigma_c=P/A'$)(kpa)
0	0.0	0.000	0.000	0.000	11.341	0.000	0.000
20	18.0	0.200	0.003	0.263	11.371	0.026	22.478
40	31.0	0.400	0.005	0.526	11.401	0.044	38.611
60	41.0	0.600	0.008	0.789	11.431	0.058	50.931
80	52.0	0.800	0.011	1.053	11.462	0.074	64.424
100	61.0	1.000	0.013	1.316	11.492	0.087	75.373
120	70.0	1.200	0.016	1.579	11.523	0.099	86.263
140	80.0	1.400	0.018	1.842	11.554	0.114	98.322
160	89.0	1.600	0.021	2.105	11.585	0.126	109.090
180	100.0	1.800	0.024	2.368	11.616	0.142	122.244
200	110.0	2.000	0.026	2.632	11.648	0.156	134.106
220	119.0	2.200	0.029	2.895	11.679	0.169	144.686
240	126.0	2.400	0.032	3.158	11.711	0.179	152.782
260	136.0	2.600	0.034	3.421	11.743	0.193	164.459
280	140.0	2.800	0.037	3.684	11.775	0.199	168.835
300	145.0	3.000	0.039	3.947	11.807	0.206	174.387
320	150.0	3.200	0.042	4.211	11.840	0.213	179.906
340	151.0	3.400	0.045	4.474	11.872	0.214	180.608
360	152.0	3.600	0.047	4.737	11.905	0.216	181.303
380	151.0	3.800	0.050	5.000	11.938	0.214	179.613
400	147.0	4.000	0.053	5.263	11.971	0.209	174.371
420	140.0	4.200	0.055	5.526	12.004	0.199	165.606

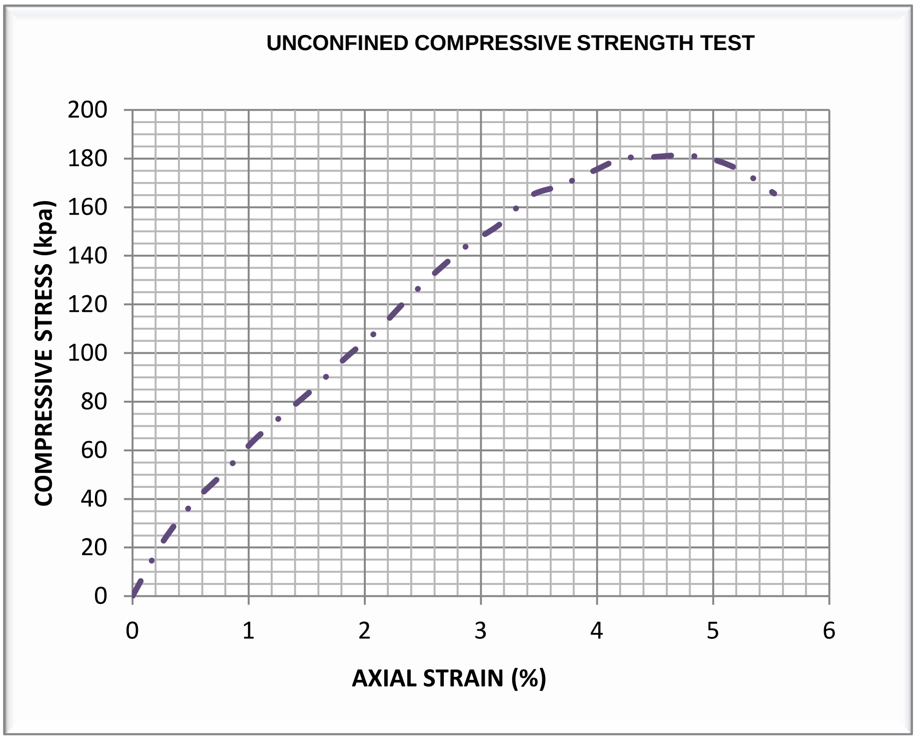

Figure F-7: Unconfined Compression Strength curve for 30%NS+70%ES

Unconfined Compressive Strength (q_u) :	**181kpa**
Cohesion (C=qu/2) :	**91kpa**

Table F-8: Unconfined Compression Strength Test for 40%NS+60%ES

Diameter of the test specimen (d) - cm : 3.80

Initial length of the test specimen (L_o) - cm: 7.60 at 15% strain , ΔL 11

Weight of Soil Sample (gm) : 156.5

A) MOISTURE CONTENT DETERMINATION

	SPECIMEN NUMBER	1
1	MOISTURE CAN AND LID NUMBER(TOP/SIDE)	p-1,l-1,s2
2	M_C=MASS OF EMPTY,CLEAN CAN + LID (M_C) (gram)	22
3	M_{CMS}=MASS OF CAN,LID, AND MOIST SOIL (M_{CMS})(gram)	59.7
4	M_{CDS}=MASS OF CAN,LID, AND DRY SOIL (M_{CDS})(gram)	52.4
5	M_S=MASS OF SOIL SOLIDS (M_S)(gram)=M_{CDS}-M_C	30.4
6	M_W=MASS OF PORE WATER(M_W) (gram) = M_{CDS}-M_{Cms}	7.30
7	WATER CONTENT,W% = M_W/M_S	24.01

Initial Area of the test specimen (A_o) - cm^2: 11.341

Initial Volume of the test specimen (V_o) - cm^3: 86.192

Wet/Bulk density (g/cm^3) : 1.816

Water Content (W %): 24.01

Dry density(g/cm3): 1.464

B) UNCONFINED COMPRESSION TEST DATA (Deformation Dial: 1 unit = 0.010mm; Load Dial: .00142kN/Div)

DEFORMATION DIAL READING	LOAD DIAL READING	SAMPLE DEFORMATION,ΔL(mm)	STRAIN ($\varepsilon=\Delta L/L_O$)	% STRAIN(ε *100)	CORRECTED AREA,($A'=A_0/1-\varepsilon$)	LOAD (P) (KN)	STRESS ($\sigma_c=P/A'$)(kpa)
0	0.0	0.000	0.000	0.000	11.341	0.000	0.000
20	31.0	0.200	0.003	0.263	11.371	0.044	38.713
40	49.0	0.400	0.005	0.526	11.401	0.070	61.030
60	64.0	0.600	0.008	0.789	11.431	0.091	79.501
80	79.0	0.800	0.011	1.053	11.462	0.112	97.874
100	94.0	1.000	0.013	1.316	11.492	0.133	116.148
120	108.0	1.200	0.016	1.579	11.523	0.153	133.091
140	120.0	1.400	0.018	1.842	11.554	0.170	147.484
160	133.0	1.600	0.021	2.105	11.585	0.189	163.023
180	144.0	1.800	0.024	2.368	11.616	0.204	176.031
200	152.0	2.000	0.026	2.632	11.648	0.216	185.310
220	162.0	2.200	0.029	2.895	11.679	0.230	196.968
240	163.0	2.400	0.032	3.158	11.711	0.231	197.646
260	163.0	2.600	0.034	3.421	11.743	0.231	197.109
280	162.0	2.800	0.037	3.684	11.775	0.230	195.366
300	159.0	3.000	0.039	3.947	11.807	0.226	191.224
320	155.0	3.200	0.042	4.211	11.840	0.220	185.903
340	148.0	3.400	0.045	4.474	11.872	0.210	177.020

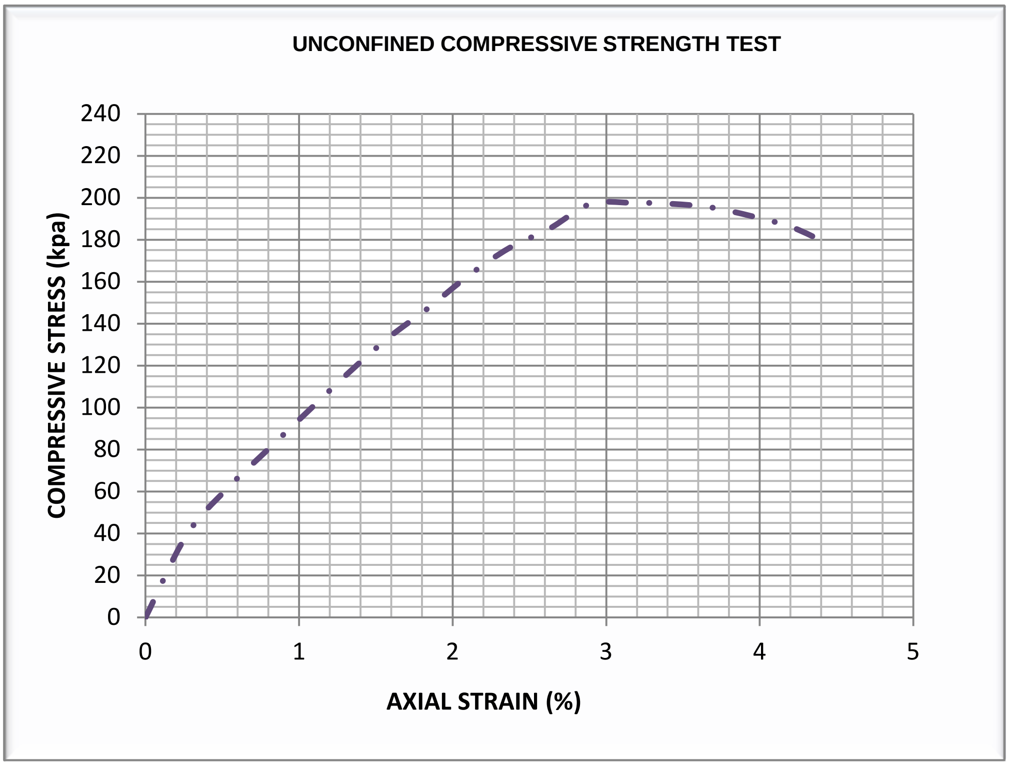

Figure F-8: Unconfined Compression Strength curve for 40%NS+60%ES

Unconfined Compressive Strength (q_u) :	**198kpa**
Cohesion (C=qu/2) :	**99kpa**

Table F-9: Unconfined Compression Strength Test for 50%NS+50%ES

Diameter of the test specimen (d) - cm : 3.80

Initial length of the test specimen (L_o) - cm: 7.60 at 15% strain , ΔL

Weight of Soil Sample (gm) : 159.3

A) MOISTURE CONTENT DETERMINATION

	SPECIMEN NUMBER	1
1	MOISTURE CAN AND LID NUMBER(TOP/SIDE)	p-1,l-1,s2
2	M_C=MASS OF EMPTY,CLEAN CAN + LID (M_C) (gram)	22
3	M_{CMS}=MASS OF CAN,LID, AND MOIST SOIL (M_{CMS})(gram)	59.2
4	M_{CDS}=MASS OF CAN,LID, AND DRY SOIL (M_{CDS})(gram)	52.6
5	M_S=MASS OF SOIL SOLIDS (M_S)(gram)=M_{CDS}-M_C	30.6
6	M_W=MASS OF PORE WATER(M_W) (gram) = M_{CDS}-M_{Cms}	6.60
7	WATER CONTENT,W% = M_W/M_S	21.57

Initial Area of the test specimen (A_o) - cm^2: 11.341

Initial Volume of the test specimen (V_o) - cm^3: 86.192

Wet/Bulk density (g/cm^3) : 1.848

Water Content (W %): 21.57

Dry density(g/cm3): 1.52

B) UNCONFINED COMPRESSION TEST DATA (Deformation Dial: 1 unit = 0.010mm; Load Dial: .00142kN/Div)

DEFORMATION DIAL READING	LOAD DIAL READING	SAMPLE DEFORMATION, ΔL(mm)	STRAIN ($\varepsilon=\Delta L/L_O$)	% STRAIN(ε*100)	CORRECTED AREA,($A'=A_0/1-\varepsilon$)	LOAD (P) (KN)	STRESS ($\sigma_c=P/A'$)(kpa)
0	0.0	0.000	0.000	0.000	11.341	0.000	0.000
20	19.0	0.200	0.003	0.263	11.371	0.027	23.727
40	28.0	0.400	0.005	0.526	11.401	0.040	34.874
60	36.0	0.600	0.008	0.789	11.431	0.051	44.720
80	45.0	0.800	0.011	1.053	11.462	0.064	55.751
100	55.0	1.000	0.013	1.316	11.492	0.078	67.959
120	63.0	1.200	0.016	1.579	11.523	0.089	77.636
140	74.0	1.400	0.018	1.842	11.554	0.105	90.948
160	89.0	1.600	0.021	2.105	11.585	0.126	109.090
180	100.0	1.800	0.024	2.368	11.616	0.142	122.244
200	110.0	2.000	0.026	2.632	11.648	0.156	134.106
220	123.0	2.200	0.029	2.895	11.679	0.175	149.549
240	136.0	2.400	0.032	3.158	11.711	0.193	164.907
260	147.0	2.600	0.034	3.421	11.743	0.209	177.761
280	157.0	2.800	0.037	3.684	11.775	0.223	189.336
300	166.0	3.000	0.039	3.947	11.807	0.236	199.643
320	175.0	3.200	0.042	4.211	11.840	0.249	209.891
340	178.0	3.400	0.045	4.474	11.872	0.253	212.902
360	177.0	3.600	0.047	4.737	11.905	0.251	211.123
380	168.0	3.800	0.050	5.000	11.938	0.239	199.834
400	162.0	4.000	0.053	5.263	11.971	0.230	192.164
420	155.0	4.200	0.055	5.526	12.004	0.220	183.349

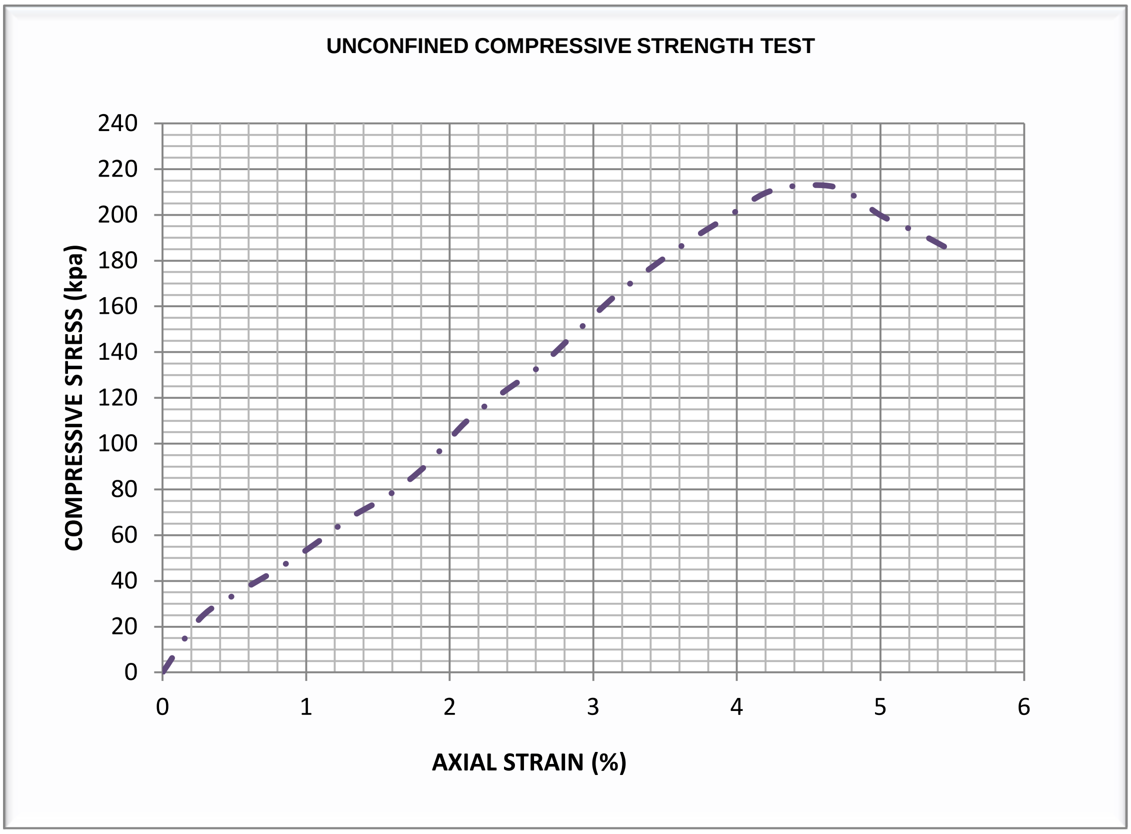

Figure F-9: Unconfined Compression Strength curve for 50%NS+50%ES

Unconfined Compressive Strength (q_u) :	**213kpa**
Cohesion (C=qu/2) :	**106kpa**

Annex G: California Bearing Ratio (CBR) Test Results

Table G-1: CBR Test for ES

SOAKED CBR Computation Table ***Sample type: Natural Soil***

Blow/ Layer	56/5
Swell, %	0.08
CBR Value, %	**0.77**

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.0	0	0.00		
0.64	1.0	26	0.01		
1.27	2.0	51	0.03		
1.91	3.0	77	0.04		
2.54	4.0	103	0.05	6.9	0.77
3.18	4.0	103	0.05		
3.81	5.0	129	0.07		
4.45	6.0	154	0.08	10.3	0.77
5.08	6.0	154	0.08	10.3	0.77
7.62	8.0	206	0.11		
10.16	10.0	257	0.13		
12.70	12.0	308	0.16		

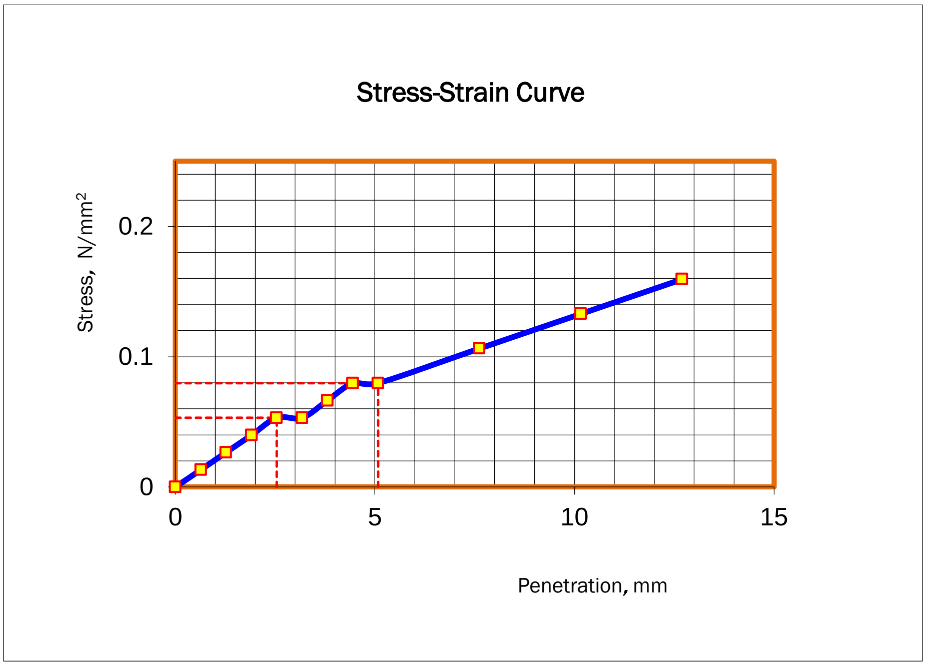

Figure G-1: Load-Penetration Curve for ES

Elapse time (Day)	Gauge reading	Swell	
		mm	%
0(Initial)	11	0.0958	0.08
4(Final)	20.58		

Initial height of sample, mm =	127

Table F-2: CBR Test for 20%CS+80%ES

SOAKED CBR Computation Table ***Sample type: 20% Crushed Sand***

Blow/ Layer	56/5
Swell, %	0.0685
CBR Value, %	**1.16**

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.0	0	0.00		
0.64	1.0	26	0.01		
1.27	3.0	77	0.04		
1.91	4.0	103	0.05		
2.54	6.0	154	0.08	6.9	1.16
3.18	6.0	154	0.08		
3.81	7.0	180	0.09		
4.45	8.0	206	0.11	10.3	1.03
5.08	8.0	206	0.11	10.3	1.03
7.62	9.0	231	0.12		
10.16	10.0	257	0.13		
12.70	11.0	283	0.15		

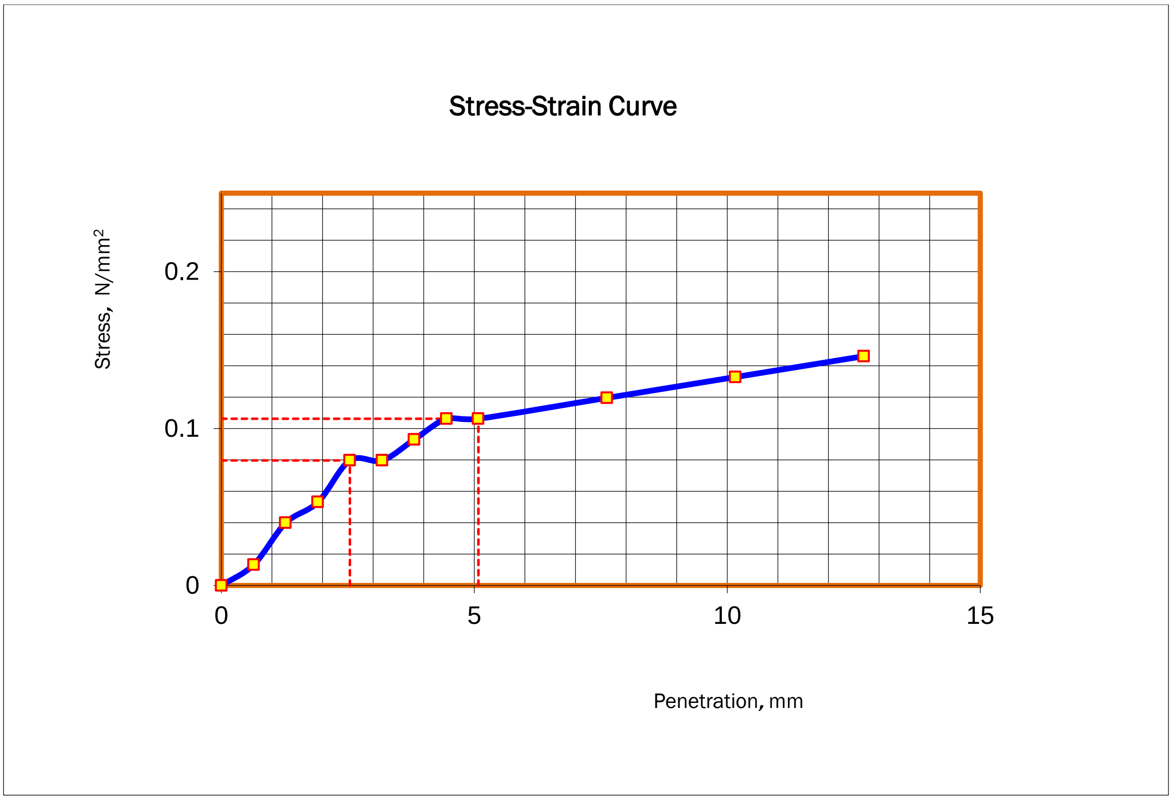

Figure G-2: Load-Penetration Curve for 20%CS+80% ES

Elapse time (Day)	Gauge reading	Swell	
		mm	%
0(Initial)	13.8	0.087	0.0685
4(Final)	22.5		

Initial height of sample, mm =	127

Table G-3: CBR Test for 30%CS+70%ES

SOAKED CBR Computation Table ***Sample type: 30% Crushed Sand***

Blow/ Layer	56/5
Swell, %	0.0566
CBR Value, %	**1.35**

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.0	0	0.00		
0.64	4.0	103	0.05		
1.27	5.0	129	0.07		
1.91	6.0	154	0.08		
2.54	7.0	180	0.09	6.9	1.35
3.18	7.0	180	0.09		
3.81	8.0	206	0.11		
4.45	8.0	206	0.11	10.3	1.03
5.08	8.0	206	0.11	10.3	1.03
7.62	9.0	231	0.12		
10.16	10.0	257	0.13		
12.70	12.0	308	0.16		

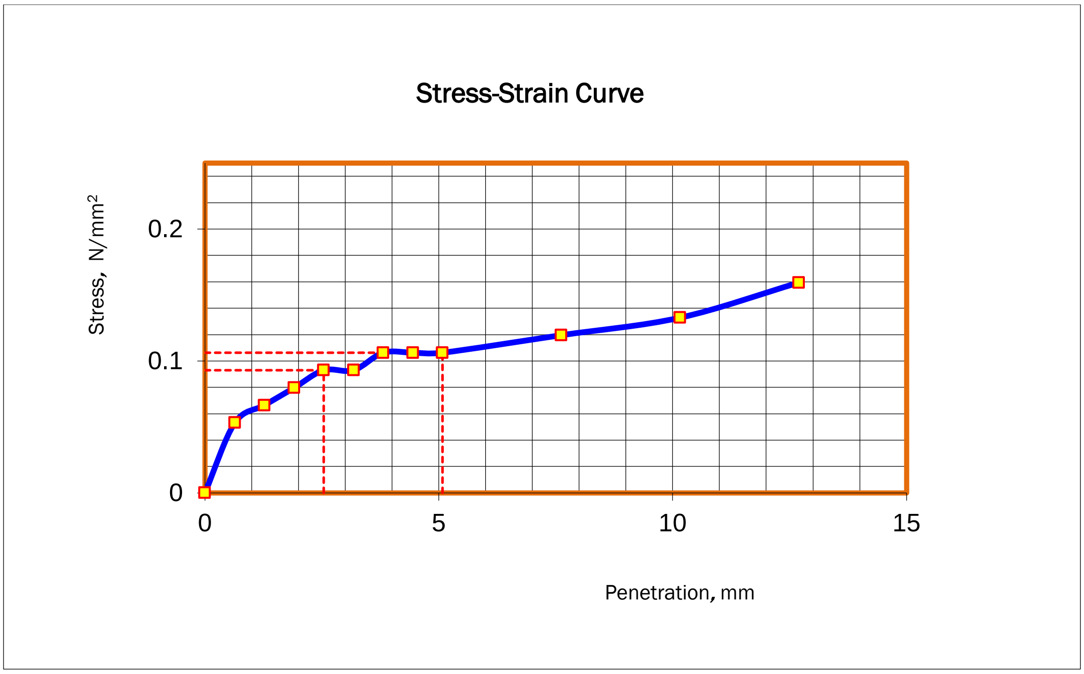

Figure G-3: Load-Penetration Curve for 30%CS+70% ES

Elapse time (Day)	Gauge reading	Swell	
		mm	%
0(Initial)	13.61	0.0719	0.0566
4(Final)	20.8		

Initial height of sample, mm =	127

Table G-4: CBR Test for 40%CS+60%ES

SOAKED CBR Computation Table ***Sample type: 40% Crushed Sand***

Blow/ Layer	56/5
Swell, %	0.0546
CBR Value, %	**1.54**

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.0	0	0.00		
0.64	3.0	77	0.04		
1.27	5.0	129	0.07		
1.91	6.0	154	0.08		
2.54	8.0	206	0.11	6.9	1.54
3.18	8.0	206	0.11		
3.81	9.0	231	0.12		
4.45	10.0	257	0.13	10.3	1.29
5.08	10.0	257	0.13	10.3	1.29
7.62	11.0	283	0.15		
10.16	12.0	308	0.16		
12.70	12.0	308	0.16		

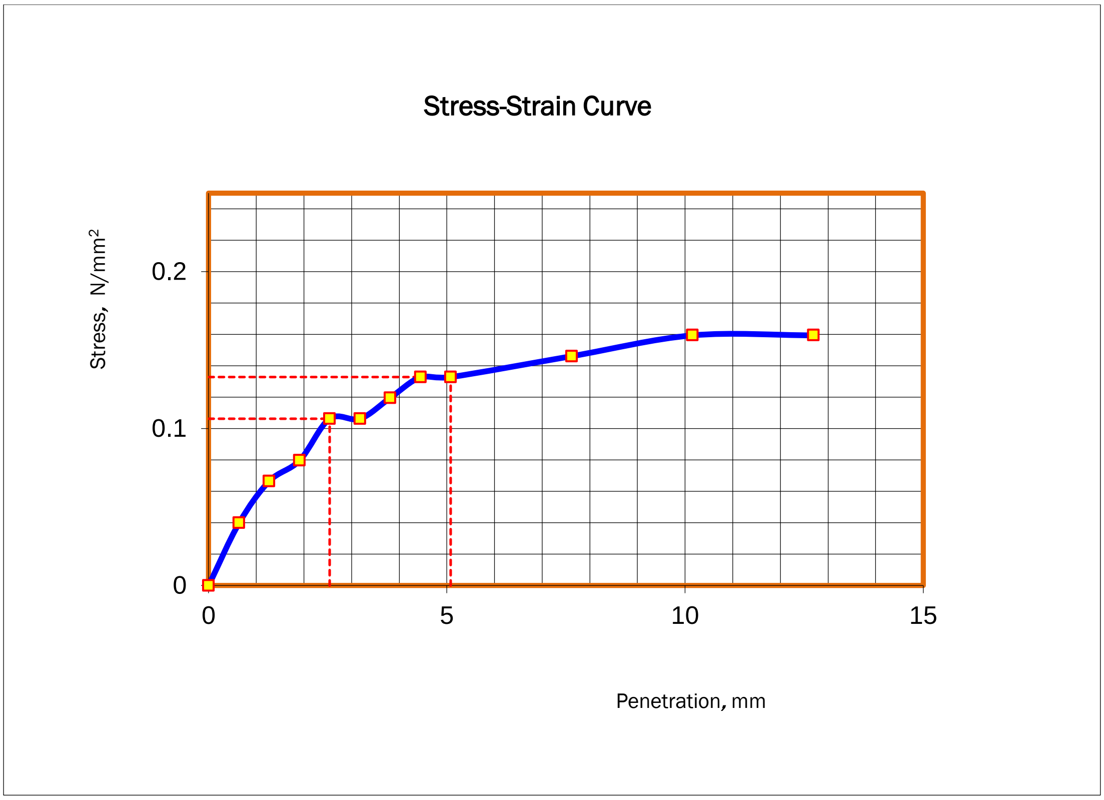

Figure G-4: Load-Penetration Curve for 40%CS+60% ES

Elapse time (Day)	Gauge reading	Swell	
		mm	%
0(Initial)	12.2	0.0693	0.0546
4(Final)	19.13		

Initial height of sample, mm =	127

Table G-5: CBR Test for 50%CS+50%ES

SOAKED CBR Computation Table ***Sample type: 50% Crushed Sand***

Blow/ Layer	56/5
Swell, %	0.0491
CBR Value, %	**1.73**

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.0	0	0.00		
0.64	4.0	103	0.05		
1.27	6.0	154	0.08		
1.91	7.0	180	0.09		
2.54	9.0	231	0.12	6.9	1.73
3.18	9.0	231	0.12		
3.81	10.0	257	0.13		
4.45	11.0	283	0.15	10.3	1.42
5.08	11.0	283	0.15	10.3	1.42
7.62	12.0	308	0.16		
10.16	14.0	360	0.19		
12.70	14.0	360	0.19		

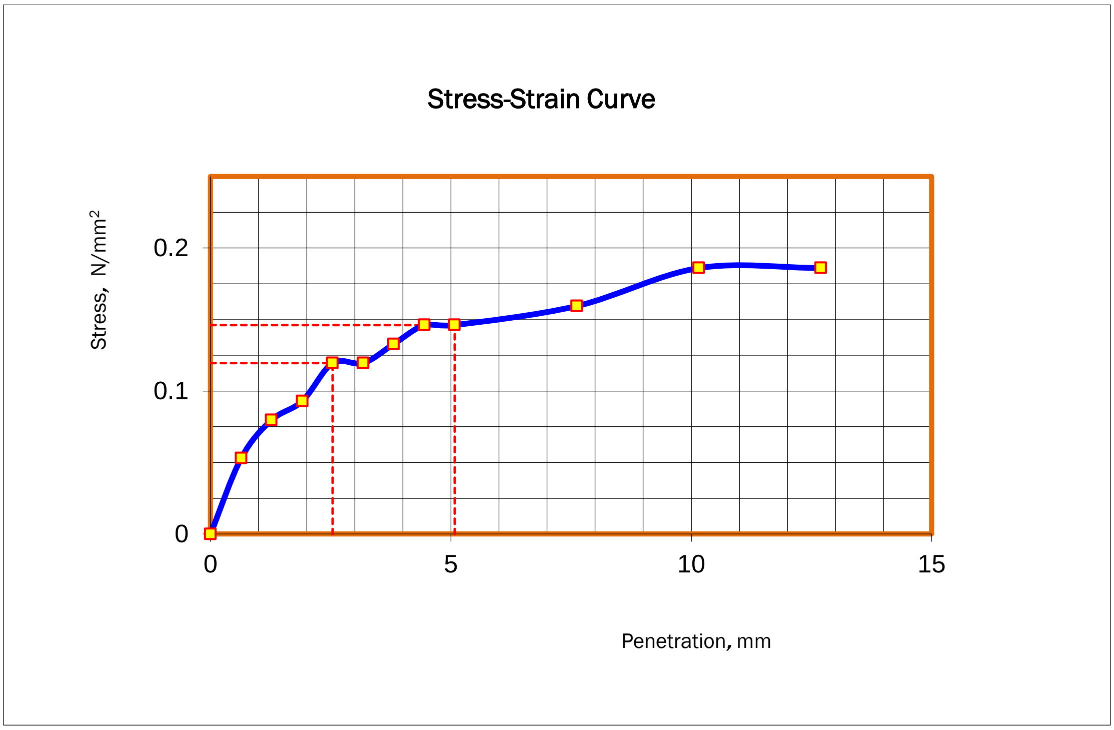

Figure G-5: Load-Penetration Curve for 50%CS+50% ES

Elapse time (Day)	Gauge reading	Swell	
		mm	%
0(Initial)	10.97	0.0623	0.0491
4(Final)	17.2		

Initial height of sample, mm =	127

Table G-6: CBR Test for 20%NS+80%ES

SOAKED CBR Computation Table ***Sample type: 20% Natural Sand***

Blow/ Layer	56/5
Swell, %	0.0709
CBR Value, %	**1.16**

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.0	0	0.00		
0.64	1.0	26	0.01		
1.27	3.0	77	0.04		
1.91	4.0	103	0.05		
2.54	6.0	154	0.08	6.9	1.16
3.18	7.0	180	0.09		
3.81	8.0	206	0.11		
4.45	8.0	206	0.11	10.3	1.03
5.08	9.0	231	0.12	10.3	1.16
7.62	12.0	308	0.16		
10.16	14.0	360	0.19		
12.70	17.0	437	0.23		

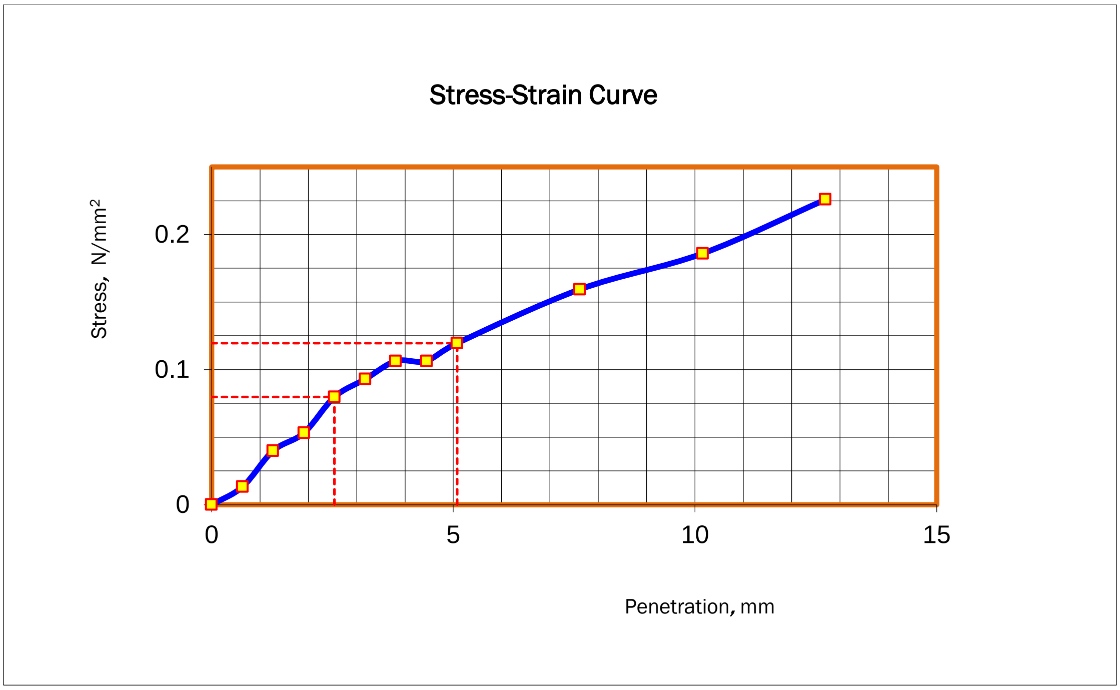

Figure G-6: Load-Penetration Curve for 20%NS+80% ES

Elapse time (Day)	Gauge reading	Swell	
		mm	%
0(Initial)	12.3	0.09	0.0709
4(Final)	21.3		

Initial height of sample, mm =	127

Table G-7: CBR Test for 30%NS+70%ES

SOAKED CBR Computation Table ***Sample type: 30% Natural Sand***

Blow/ Layer	56/5
Swell, %	0.0661
CBR Value, %	**1.35**

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.0	0	0.00		
0.64	3.0	77	0.04		
1.27	4.0	103	0.05		
1.91	6.0	154	0.08		
2.54	7.0	180	0.09	6.9	1.35
3.18	8.0	206	0.11		
3.81	9.0	231	0.12		
4.45	10.0	257	0.13	10.3	1.29
5.08	10.0	257	0.13	10.3	1.29
7.62	12.0	308	0.16		
10.16	14.0	360	0.19		
12.70	14.0	360	0.19		

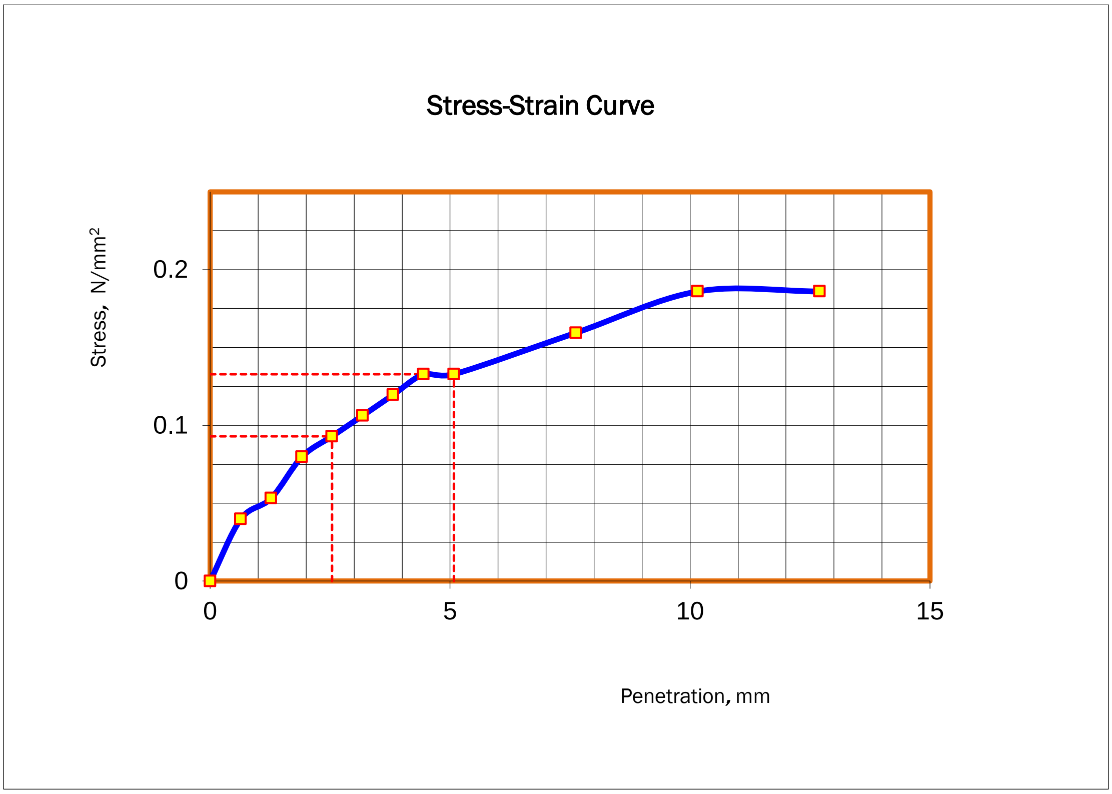

Figure G-7: Load-Penetration Curve for 30%NS+70% ES

Elapse time (Day)	Gauge reading	Swell	
		mm	%
0(Initial)	12.05	0.084	0.0661
4(Final)	20.45		

Initial height of sample, mm =	127

Table G-8: CBR Test for 40%NS+60%ES

SOAKED CBR Computation Table ***Sample type: 40% Natural Sand***

Blow/ Layer	56/5
Swell, %	0.0639
CBR Value, %	**1.73**

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.0	0	0.00		
0.64	3.0	77	0.04		
1.27	5.0	129	0.07		
1.91	7.0	180	0.09		
2.54	9.0	231	0.12	6.9	1.73
3.18	10.0	257	0.13		
3.81	11.0	283	0.15		
4.45	12.0	308	0.16	10.3	1.55
5.08	13.0	334	0.17	10.3	1.68
7.62	15.0	386	0.20		
10.16	17.0	437	0.23		
12.70	19.0	488	0.25		

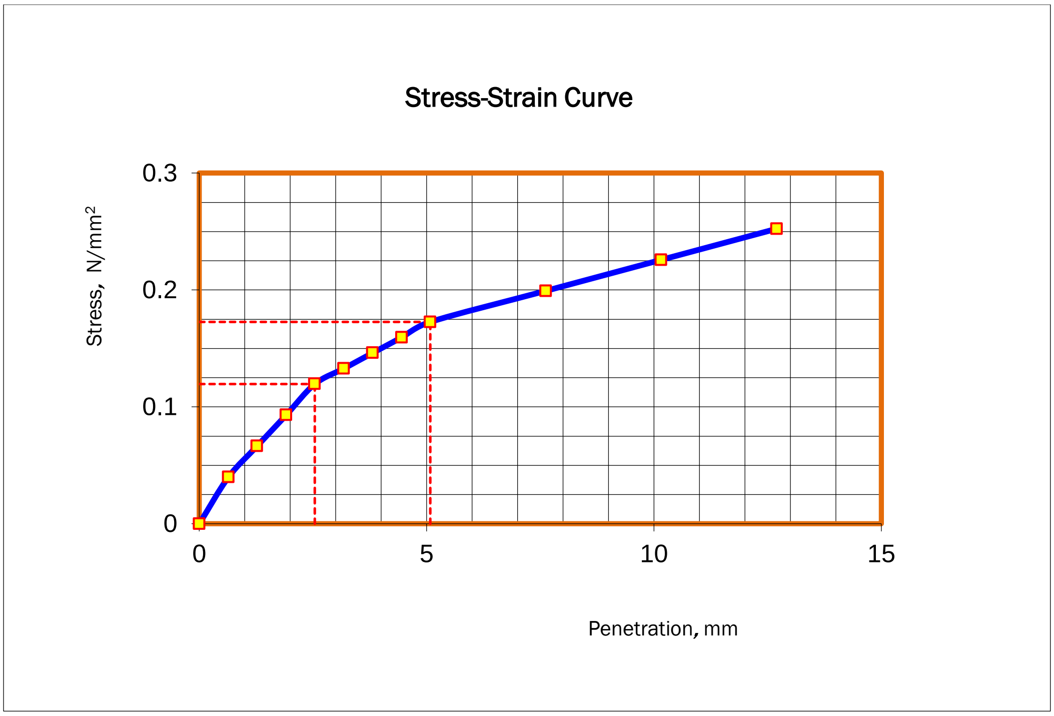

Figure G-8: Load-Penetration Curve for 40%NS+60% ES

Elapse time (Day)	Gauge reading	Swell	
		mm	%
0(Initial)	13.48	0.0812	0.0639
4(Final)	21.6		

Initial height of sample, mm =	127

Table G-9: CBR Test for 50%NS+50%ES

SOAKED CBR Computation Table ***Sample type: 50% Natural Sand***

Blow/ Layer	56/5
Swell, %	0.0541
CBR Value, %	**1.93**

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.0	0	0.00		
0.64	3.0	77	0.04		
1.27	6.0	154	0.08		
1.91	9.0	231	0.12		
2.54	10.0	257	0.13	6.9	1.93
3.18	11.0	283	0.15		
3.81	13.0	334	0.17		
4.45	14.0	360	0.19	10.3	1.81
5.08	17.0	437	0.23	10.3	2.19
7.62	21.0	540	0.28		
10.16	24.0	617	0.32		
12.70	27.0	694	0.36		

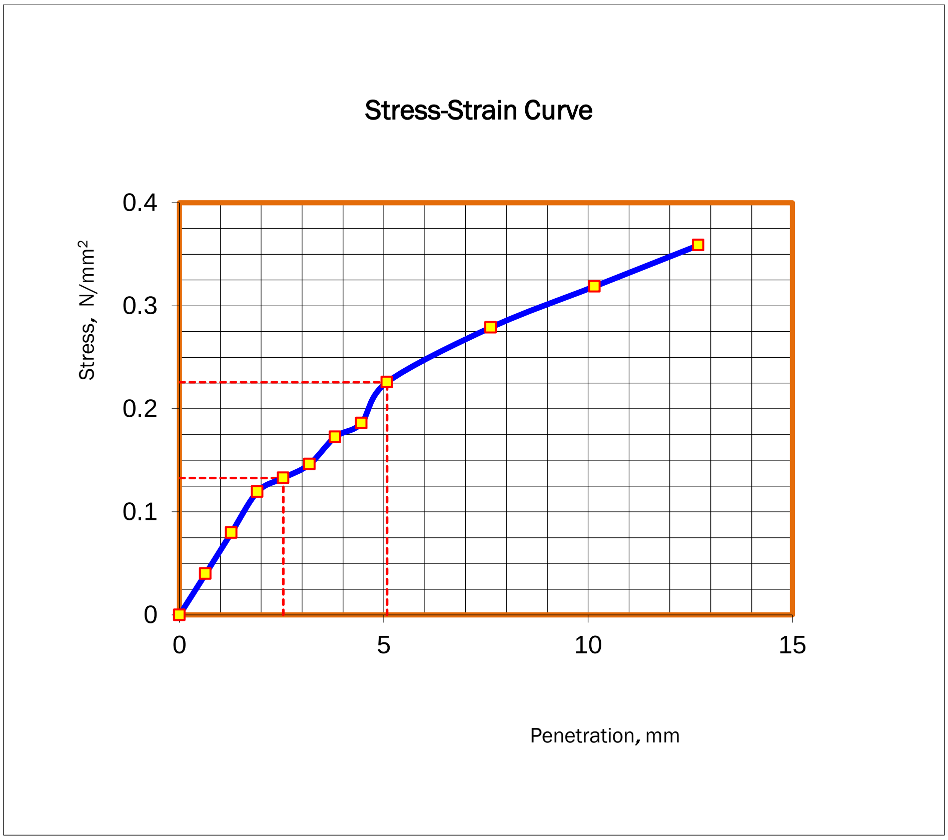

Figure G-9: Load-Penetration Curve for 50%NS+50% ES

Elapse time (Day)	Gauge reading	Swell	
		mm	%
0(Initial)	2.75	0.0687	0.0541
4(Final)	9.62		

Initial height of sample, mm =	127

Table G-10: Un-soaked CBR Test for 40%CS+60%ES

UNSOAKED CBR Computation Table ***Sample type: 40% Crushed Sand***

Blow/ Layer	56/5
Swell, %	-
CBR Value, %	**23.49**

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.0	0	0.00		
0.64	69.0	1774	0.92		
1.27	99.0	2545	1.32		
1.91	107.0	2751	1.42		
2.54	122.0	3136	1.62	6.9	23.49
3.18	127.0	3265	1.69		
3.81	130.0	3342	1.73		
4.45	132.0	3393	1.75	10.3	17.03
5.08	134.0	3445	1.78	10.3	17.28
7.62	136.0	3496	1.81		
10.16	138.0	3548	1.83		
12.70	140.0	3599	1.86		

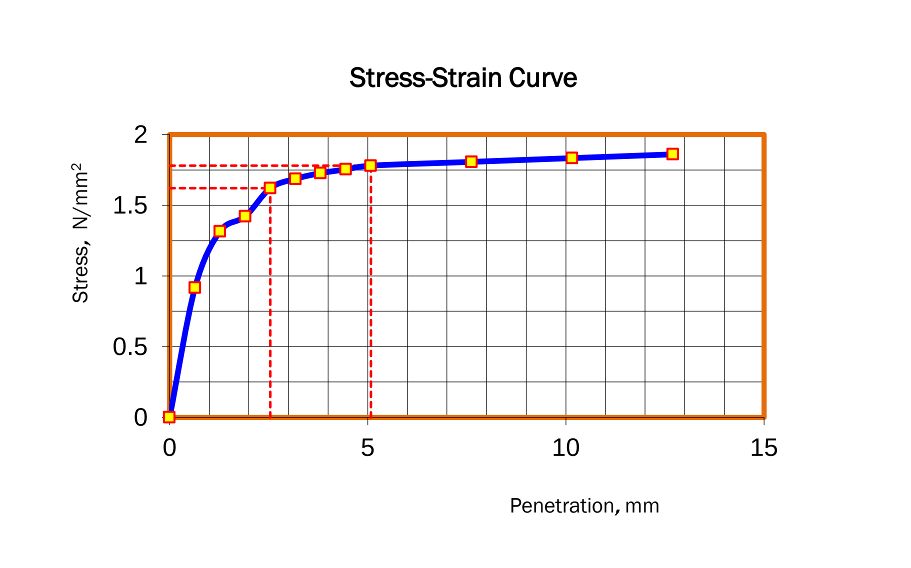

Figure G-10: Load-Penetration Curve for 40%CS+60% ES(Un-soaked)

Table G-11: Un-soaked CBR Test for 40%NS+60%ES

UNSOAKED CBR Computation Table ***Sample type: 40% Natural Sand***

Blow/ Layer	56/5
Swell, %	-
CBR Value, %	**32.35**

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.0	0	0.00		
0.64	95.0	2442	1.26		
1.27	135.0	3470	1.79		
1.91	155.0	3985	2.06		
2.54	168.0	4319	2.23	6.9	32.35
3.18	180.0	4627	2.39		
3.81	185.0	4756	2.46		
4.45	188.0	4833	2.50	10.3	24.25
5.08	191.0	4910	2.54	10.3	24.64
7.62	193.0	4961	2.56		
10.16	194.0	4987	2.58		
12.70	194.0	4987	2.58		

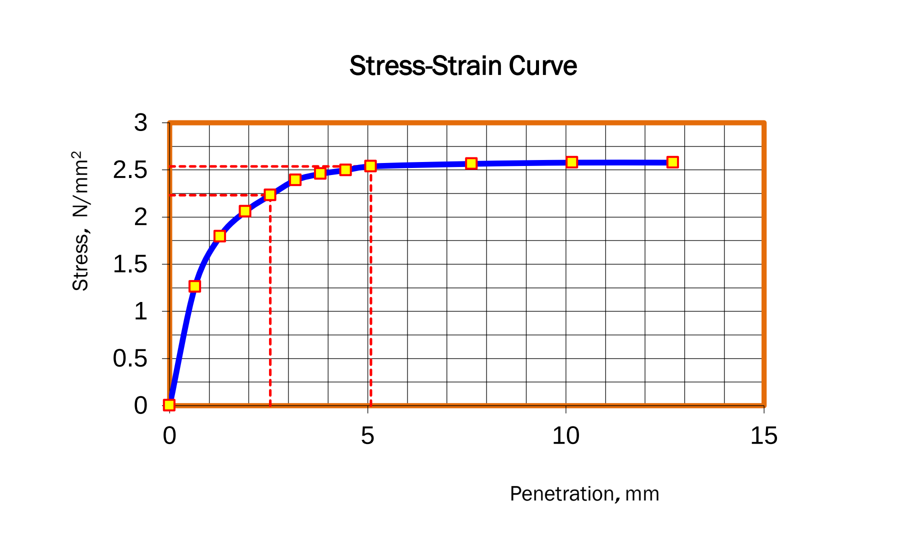

Figure G-11: Load-Penetration Curve for 40%NS+60% ES (Un-soaked)

Annex G: Cost Comparison between Un-stabilized and Stabilized Soil for Typical G+1 Residential Building

Table G-1 cost of sub structure work for G+1 residential building by replacing with non- expansive soil

BILL OF QUANTITY USING NON EXPANSIVE SOIL					
ITEM	DESCRIPTION	UNIT	QTY	RATE	AMOUNT
	<u>A. SUB STRUCTURE</u> <u>1. EXCAVATION & EARTH WORK</u>				
1.1	Site clearing and removing of the top 200mm thick soil	m^2	151.00	10.00	1,510.00
1.2	Bulk excavation in ordinary soil not exceeding 1000mm from NGL.	m^3	61.00	25.00	1,525.00
1.3	Excavate in ordinary soil for isolated footing to a depth exceeding 1500mm but not exceeding 3000mm from reduced level.	m^3	141.00	57.00	8,037.00
1.4	Ditto item 1.3 exceeding 150cm but not exceeding 300cm.	m^3	27.00	68.00	1,836.00
1.5	Trench excavation for perherial masonry wall to a depth of 1500mm starting from reduced ground level.	m^3	38.00	57.00	2,166.00
1.6	Fill with non-expansive granular material around trench, isolated foundation and under hardcore to maintain the desired level and compact in layers not exceeding 200mm until it attains a minimum of 95% proctor density.	m^3	228.00	185.00	42,180.00
1.7	Cart away surplus excavated material to a distance not exceeding 2kms.	m^3	296.00	65.00	19,240.00
1.8	250mm. thick sound basaltic or equivalent stone hard-core finished and blended with crushed stone.	m^2	137.00	100.00	13,700.00
	Total to Summary……………………………..Birr				**90,194.00**

Table G-2 cost of excavation and earth work for G+1 residential building by fill using an expansive soil stabilized by 40% sand

BILL OF QUANTITY WITH EXPANSIVE SOIL STABLIZED BY 40% SAND					
ITEM	**DESCRIPTION**	**UNIT**	**QTY**	**RATE**	**AMOUNT**
	A. SUB STRUCTURE **1. EXCAVATION & EARTH WORK**				
1.1	Site clearing and removing of the top 200mm thick soil	m^2	151.00	10.00	1,510.00
1.2	Bulk excavation in ordinary soil not exceeding 1000mm from NGL.	m^3	61.00	25.00	1,525.00
1.3	Excavate in ordinary soil for isolated footing to a depth exceeding 1500mm but not exceeding 3000mm from reduced level.	m^3	141.00	57.00	8,037.00
1.4	Ditto item 1.3 exceeding 150cm but not exceeding 300cm.	m^3	27.00	68.00	1,836.00
1.5	Trench excavation for perherial masonry wall to a depth of 1500mm starting from reduced ground level.	m^3	38.00	57.00	2,166.00
1.6	Fill with stabilized soil around trench, isolated foundation and under hardcore to maintain the desired level and compact in layers not exceeding 200mm until it attains a minimum of 95% proctor density.	m^3	228.00	173.00	39,444.00
1.7	Cart away surplus excavated material to a distance not exceeding 2kms.	m^3	119.00	65.00	7,735.00
1.8	250mm. thick sound basaltic or equivalent stone hard-core finished and blended with crushed stone.	m^2	137.00	100.00	13,700.00
	Total to Summary……………………………..Birr				**75,953.00**

Table G-3 Summary of cost comparison

OPTION NO	DESCRIPTION	TOTAL SUB STRUCTURE COST IN BIRR
1	Sub structure work by replacing using non-expansive soil.	90,194.00
2	Sub structure work using an expansive soil stabilized by 40% sand	75,953.00