EFFECT OF NONPLASTIC-SILT AND SAND CONTENT ON PLASTICITY AND CBR OF CLAY SOIL

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It is hereby declared that this project or any part of it has not been submitted elsewhere for the award of any degree or diploma.

Fakir Mohammed Ali Taufique

TABLE OF CONTENTS

List of Figures

List of Tables

NOTATION

LL = Liquid Limit

PL = Plastic Limit

PI = Plasticity Index

CBR = California Bearing Ratio

OMC = Optimum Moisture Content

C_u = Uniformity Coefficient

Cc = Coefficient of Curvature

AASHTO = American Association of State Highway Transportation Officials

ASTM = American Society for Testing and Materials

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ABSTRACT

Recently it has been observed that many national highways and other important roads are seriously damaged after one/two years of construction. Therefore it is necessary to investigate the causes of damage of roads. In most places soils are silty clay or clayey silt. During rainy season and flooding, these soils become saturated and soft. Suitability of subgrade material should be investigated using Liquid limit, Plastic Limit and soaked CBR values. The study aims to investigate the suitability of sand-clay and silt-clay mixtures as subgrade material for highways and roadways construction.

From the study it was found that with the increase of sand content in sand-clay mixture and silt content in silt-clay mixture, maximum dry density increased but optimum moisture content decreased. Soaked CBR values increased with the increase of the percentage of sand while they reduced gradually with the reduction of sand in the Clay-Sand mixture. When a comparison is drawn between Soaked CBR of Clay-Sand mixture and that of Clay-Silt mixture, it has been found that soaked CBR values in the Clay-Sand mixture are much higher. It can thus be concluded that Clay-Sand mixture is a good subgrade material while Clay-Silt mixture is a poor subgrade material for the purpose of road construction.

CHAPTER ONE
INTRODUCTION

1.1 General

It has been observed that highways and other important roads are seriously damaged after one or two years of construction. Therefore it is necessary to investigate the causes of damage of roads. During construction of roads it is usually observed that soils available on two sides of road are used in the construction of embankment. In most places soils are clay or clayey silt. Suitability of subgrade material should be investigated using Liquid limit, Plastic Limit and soaked CBR values.

Clay-sand and Clay-silt mixtures are currently used as engineered filling when constructing roads or embankment dams (Fukue et al. 1986). For larger embankment dams or road, their use is typically confined to the construction of a low permeability dam core, which is often used in conjunction with an engineered soil filter (Jafari and Shafiee, 2004). It is also feasible to use a mixture of highly plastic clay with sand to construct liner systems or other types of impervious buffer zones for waste disposal projects (Chapuis, 1990). In these cases, the plasticity and the compaction behavior of the engineered clay-sand and clay-silt mixtures are dependent upon the Atterberg limits and compaction process. For Geotechnical engineers that are designing these types of engineered fill systems, it is useful to have an understanding of this behavior as a function of the plasticity and compaction process that is used. A review of past studies has revealed that the majority of previous research in this area has focused on the behavior of pure sands or clays, while research on clay-sand and clay-silt mixtures has been limited.

This particular study investigated the plasticity behavior by doing Atterberg limits test and compaction characteristics of laboratory compacted lean clay-sand and clay-silt mixtures. Here, plasticity is an important feature in the case of fine-grained soils, the term plasticity describing the ability of a soil to undergo unrecoverable deformation at constant volume without cracking or crumbling. It is due to the presence of clay minerals or organic material. However, compaction is the process of increasing the density of a soil by packing the particles closer together with a reduction in the volume of air, there is no significant change in the volume of water in the soil. In addition, the California Bearing Ratio test, or soaked CBR test is performed which was a laboratory testing method to estimate the bearing value and the mechanical strength of highway sub-bases and sub- grades.

The ultimate purpose of this research was to obtain data which can be used by engineers to predict the compaction properties including change of maximum dry density & change of moisture content with percentage of sand, soaked CBR value of soil mixture to know the best possible for roadway construction, and plasticity phenomena including change of plastic limit, change of liquid limit, change of plasticity index with percentage of Sand and silt in the context of clay-sand and clay-silt mixtures. This will make it easier for engineers to better design earthen levees, embankment dams, containment barrier systems, highway that utilize these mixtures in their construction. And finally, the test results that are presented herein also provide useful insight into the fundamental principles of soil behavior that affect the mechanical behavior of lean clay-sand and clay-silt mixtures.

1.2 Background of the Study

There are many researches that have been implemented to know the undrained shear strength, consolidation, and the stress - strain relationship of clay-sand and clay-silt mixtures. Moreover, the clay-sand and clay-silt mixture was attributed on a few researches to witness the plasticity and compaction characteristics. But not a single dissertation is done on clay-sand clay-silt mixture. So it is a real hard opportunity to do this kind of job. Sand-clay and silt-clay mixtures are found in several parts of the world. It is generally believed that addition of clay or introduction of plasticity to the silt increases the resistance of silts against liquefaction (Puri, 1984). It was demonstrated by Sandoval (1989), and Prakash and Sandoval (1992), that for plasticity index (PI) in the range of 2-4%, the liquefaction resistance of silt decreases with increasing Plasticity.

Moreover, if someone knows the plasticity and compaction characteristics of the clay-sand and clay-silt mixture, it would create a lot of opportunity to the geologists, soil scientists and foundation engineers to be accustomed of these phenomena.

1.3 Objectives of the Study

The objectives of the study are as follows:

i) To study the effect of nonplastic silt content on plasticity behavior of clay soil.
ii) To study the effect of sand content on plasticity behavior of clay soil.
iii) To study the effect of nonplastic silt content on soaked CBR of clay soil.
iv) To study the effect of sand content on soaked CBR of clay soil.

1.4 Methodology

To cognize the plasticity and compaction behavior of Lean Clay-Sand and Lean Clay-Silt mixture, several tests have been performed including grain size analysis test, Atterberg limit test, and compaction test and soaked CBR test finally.

A specific gravity test was done to know the specific gravity of sand and lean clay and nonplastic silt. Lean clay and nonplasic silt was gone through hydrometer test in order to know the grain size distribution as its particle size is less than .075 mm. approximately; we have been completed five LL and PL test and nine compaction test. At the beginning of the project, 10% Sand and 10% Silt were mixed with 90% Lean Clay and performed Atterberg limits test and compaction test both. After that, 20% Sand and 20% Silt were mixed with 80% Lean Clay and again performed Atterberg limits test and compaction test. By following this order, at the end 90% Sand and 90% Silt were mixed with 10% Lean Clay and performed these two tests. Because of inadequate soil sample, It had done eight soaked CBR test having Clay100%, Lean Clay 90% + Sand 10%, Lean Clay 80% + Sand 20%, Lean Clay 70% + Sand 30%, Similarly, Lean Clay 0% + Silt 100%, Lean Clay 90% + Silt 10%, Lean Clay 80% + Silt 20%, Lean Clay 70% + Silt 30% Mixing ratios were strictly maintained in the overall project work.

1.5 Organization of the thesis

The thesis is arranged into five chapters and one appendix. In Chapter One, background and objectives of the research are presented. Chapter Two contains the literature review where an elaborate description of Specific Gravity, Hydrometer Analysis, Plasticity and Compaction behavior of soil mixture have been presented briefly. The testing program is mentioned in Chapter Three. Chapter Four is analyzed about results and discussion about the topic of this dissertation. Chapter Five contains the conclusions and recommendations for further research. Graphs and testing results are presented in the appendix A.

CHAPTER TWO
LITERATURE REVIEW

2.1 Introduction

It is clear from the Geotechnical point of view that relationships between two or more soil types are very essential in practical application. The mixture of sand, nonplastic silt and clay will help to predict the behavior of such similar composition of other types. For Geotechnical engineers that are designing these types of engineered fill systems, it is useful to have an understanding of the engineering behavior of these mixtures as a function of the soil mixture and compaction process that is utilized. This chapter deals with the definition and implementation of Atterberg's limit to understand the plasticity of Lean Clay-Sand mixture and Lean Clay-Silt mixture, and compaction characteristics including soaked CBR.

2.2 Plasticity and Compaction Behavior of Sand-Clay mixture

Soil makes it more workable by reducing the water content; besides a drastic decrease in plasticity index is generally observed. This decrease in plasticity index is brought about from either an increase or decrease in liquid limit and a definite increase in plastic limit of the modified soil in comparison with the natural soil. Although most researchers report a decrease in plasticity index, the issue still, remains contentious. Diamond and Kinter (1965) found that excess amounts of lime would result in an incremental increase in plasticity index. Bell (1996) tested the changes in plasticity index of pure clay minerals. Significant reduction in plasticity index was observed in case of laminated clay. The optimum moisture content increases and maximum dry unit weight decreases in comparison with the natural soil. Sweeney et al. (1988) reported that the short term reactions which take place before compaction will result in the cementation of particles into a loose structure and cementation that has developed at the points of contact between the edges and faces of adjacent clay particles will cause the soil to offer greater resistance to compaction. Therefore, for a given compactive effort, a lower unit weight would be expected as the time available for this reaction increases. The impact of compaction energy on the strength of stabilized soil was studied by Thompson (1969). He reported that a 5% increase in compaction energy can result in an increase in unconfined compressive strength as much as 60%. Uddin et al. (1996) studied the effect of amount of lime on the strength of lime stabilized soils.

Compacted aggregate-clay and sand-clay mixtures are currently successfully used as the cores of embankment dams. These materials, called composite clays by Jafari and Shafiee (2004), are usually broadly graded and are composed of clay as the main body with sand & gravel in the clay matrix. Miboro and Ohshirakawa dams in Japan (Asao, 1963), Taguaza dam in Venezuela (Sherard, 1981) and Karkheh dam in Iran are some examples of dams with cores composed of aggregate-clay & sand-clay mixtures. It is also a current practice to employ a mixture of high plastic clay with sand & aggregates as impervious blankets for waste disposal projects (Lundgren, 1981; Abeele, 1986; Chapuis, 1990; Pandian et al., 1995). It is generally assumed that the coarser fraction of such soils imparts a relatively higher shear strength, high compacted density and low compressibility while the permeability of the soil is governed by the proportion and nature of the finer fraction. This generally results in a relatively serviceable and trouble free fill (Garga and Madureira, 1985). A review of the published literature (Hall,1951; Holtz and Willard,1956; Miller and Sowers,1957; Holtz and Ellis,1961; Patwardhan et al.,1970; Shakoor and Cook,1990; Vallejo and Zhou,1994; Muir Wood and Kumar,2000; Jafari and Shafiee,2004) reveals that experimental studies on Sand-clay mixtures.

2.3 Plasticity and Compaction Behavior of Silt-Clay mixture

Silts and silt-clay mixtures are found in several parts of the world. However it has been observed that, if a sand-fines mixture has the same standard penetration value (N1) 60 as the clean sands, the addition of fines increase its liquefaction resistance (Seed et al. 1985). This has led to an erroneous belief that the addition of fines to sands increases their liquefaction resistance and therefore, addition of clays to silt will also increase their liquefaction resistance. However, Troncoso (1990) established that if fines are added to sands, their resistance to liquefaction decreases if the soils are tested at the same void ratio. Neither Troncoso (1990) nor Seed et al. (1985) describe the plasticity characeristics of fines in the sand. It is generally believed that addition of clay or introduction of plasticity to the silt increases the resistance of silts against liquefaction (Puri 1984).

Past research has debated the effect of non-plastic silt content on the liquefaction behavior of sand. Many studies suggest that liquefaction resistance increases with increasing silt content (Seed et al. 1983; Tokimatsu and Yoshimi 1983; Robertson and Campanella 1985; Seed et al. 1985; Kuerbis et al. 1988; Kuerbis 1989; Pitman et al. 1994; Salgado et al. 2000; Amini and Qi 2000; Polito and Martin 2001). However, other studies conclude that loose silty sands are more prone to liquefaction (Sladen and Hewitt 1989; Verdugo and Ishihara 1996; Lade and Yamamuro 1997; Yamamuro and Lade 1997; Zlatovic and Ishihara 1997). Silty sand can be deposited into dense configurations that result in more dilatant behavior than clean sand (Kuerbis 1989). However, silty sand also has been show to have a greater potential for exhibiting much more volumetrically contractive behavior when deposited in very loose states (Lade and Yamamuro 1997). Yamamuro and Lade (1997, 1998) and Yamamuro and Covert (2001) concluded that complete static liquefaction (zero effective confining pressure and zero effective stress difference) in laboratory testing is most easily achieved in silty sands at very low pressures. Kramer and Seed (1988) also observed that liquefaction resistance increased with increasing confining pressure. Numerous studies (Oda 1972, 1972; Ladd 1974; Mulilis et al. 1977; Tatsuoka et al.1979; Miura and Toki 1982; Tatsuoka et al. 1986; Zlatovic and Ishihara 1997; Jang and Frost 1998; Vaid et al. 1999; Wood and Yamamuro 1999; Høeg et al. 2000) have reported that the behavior of sands can be greatly influenced by specimen reconstitution method. However, experimental data related to the effect of depositional method on the behavior of sand with non-plastic silt content is very limited because most prior studies have focused their efforts on clean sands.

2.4 Importance of Plasticity Behavior on Soil Mixture

The liquid limit, plastic limit and plasticity index of soils are also used extensively, either individually or together, with other soil properties to correlate with the engineering behavior such as compressibility, permeability, compatibility, shrink swollen and shear strength. However, adding water to clay can turn it from a solid into a fluid state. Heavy rains that saturate clay on a steep slope can suddenly turn into a liquid, resulting in a landslide. A low plasticity soil like sand is subject to erosion by prolonged rains. Steep slopes of clay in areas of heavy rainfall may have to be held in place by ground cover to prevent landslides Thus these tests are used widely in the preliminary stages of building any structure to ensure that the soil will have the correct amount of shear strength and not too much change in volume as it expands and shrinks with different moisture contents. The importance of the liquid limit test is to classify soils. Different soils have varying liquid limits. Also, one must use the plastic limit to determine its plasticity index. Besides, the knowledge of the soil consistency is important in defining or classifying a soil type or predicting soil performance when used a construction material. The soil consistency is a practical and an inexpensive way to distinguish between silts and clays. To the improvement of soil characteristics for maintaining its allowable load sustainability, deformation and stability understanding of soils nature of creation of model by mixing soil technique is a first requirement at design of soil foundation.

2.5 Compaction Behavior of soil mixture

Soils are used as subgrades for the construction of roadway pavements or other structures and transported soils used in embankments or as leveling material for various types of construction projects are usually compacted to improve their density and other properties. Increasing the soil's density improves its strength, lowers its permeability, and reduces future settlement. Seed and Chan (1959) discussed the effect of soil structure in compacted clays on shrinkage, swelling, swell pressures, stress-deformation characteristics, undrained strength, pore-water pressures, and effective strength characteristics. The increase of water content from dry to wet of optimum was believed to play an important role in producing an increased degree of particle orientation and clay particle dispersion, which then had a significant effect on the associated clay behavior. Wilson (1952) investigated the effect of compaction water content on the compressibility of compacted clayey sand. Lambe (1958) attributed the compressibility behavior of compacted clays in large part to the particle rearrangement that occurs under application of a load. Hodek and Lovell (1978) presented convincing evidence of a strong relationship between pore size distribution and the compressibility characteristics of a compacted clayey soil shown in Fig 2.1. They concluded that the dry of optimum samples consisted mostly of large pores.

There are some controlling factors affecting the extent of compaction:

(1) Compaction effort

(2) Soil type and gradation

(3) Moisture content.

2.5.1 Compaction of fine-grained soils.

The compaction method for a fine-grained soil is entirely different than that for a coarse-grained soil. The reason is that fine-grained soils possess cohesion. It should be remembered that the finer fraction of the fine-grained soils exists in a colloidal state, and all colloids possess cohesion. The mineral grains of a cohesive soil are not in physical contact, as they are in a coarse-grained soil. Every grain is surrounded by a blanket of water, whose molecules are electrically bonded to the grains. This blanket of water isolates the grains and prevents them from being in physical contact with adjacent grains (Duncan 1992). The degree to which a fine-grained soil can be compacted is almost wholly dependent on the in-situ moisture content of the soil. The moisture content that corresponds to the maximum degree of compaction (under a given compaction energy) is called the optimum moisture content. The approximate optimum moisture content of several soil groups is given in Table 2.1.

2.5.2 Compaction of coarse-grained soils.

The method behind why compaction works for a coarse-grained soil is entirely different than that for a fine-grained soil. Coarse-grained soils exist by their very nature in inter-granular contact, much like a bucket of marbles. The way these grains are arranged within the mass and the distribution of particle size throughout the mass, will ultimately determine the density, stability, and load-bearing capacity of that particular soil (Duncan 1992). The honeycombed structure shown in Fig. 2.4a is representative of very poor inter-granular seating. Such a structure is inherently unstable and can collapse suddenly when subjected to shock or vibration. The stability and load-bearing capacity of this type of soil will be improved by compaction because of the resulting rearrangement in inter-granular seating. With sufficient compaction, this structure will take on the characteristics of the arrangement shown in Fig. 2.4c. The arrangement of particles shown in Fig. 2.4b provides maximum inter-granular contact, but there are insufficient fines to lock the larger particles in place. Compaction of this type of arrangement is ineffective, since neither additional particle contact nor additional stability can be achieved. This soil is inherently stable, however, when it is laterally restrained, and demonstrates good load-bearing characteristics. When insufficiently restrained, however, this soil will be free to move laterally, in which case there is a pronounced loss in stability and load-bearing characteristics. The arrangement of particles shown in Fig. 2.4c not only provides maximum inter-granular contact, but also inherent stability. This very important property of stability is due to the inclusion of fines in the spaces between the larger particles. One cautionary note must be made concerning fines: too many fines are detrimental to the mix because they may separate the larger grains, thereby destroying the inter-granular contact between them. In this instance, the larger grains are more or less floating in a sea of fines.

2.5.3 Importance of Compaction on Soil mixture

Compacted Clay-Sand mixtures are currently used as engineered fills when constructing earthen levees or embankment dams and roads (Fukue et al. 1986). For larger embankment dams and roads, their use is typically confined to construction of a low permeability dam core, which is often used in conjunction with an engineered soil filter (Jafari and Shafiee 2004). It is also feasible to use a mixture of highly plastic clay with sand to construct liner systems or other types of impervious buffer zones for waste disposal projects (Chapuis 1990). In these cases, the compaction and compressibility behavior of the engineered Clay-Sand mixtures are dependent upon the soil compaction process. For geotechnical engineers that are designing these types of engineered fill systems, it is useful to have an understanding of the compressibility behavior of these mixtures as a function of the compaction process and compaction energy that is used. A review of past studies has revealed that the majority of previous research in this area has focused on the behavior of pure sands or clays, while research on Clay-sand mixtures has been very limited. The ultimate purpose of this research was to obtain data which can be used by engineers to predict the compaction properties, compressibility characteristics of partially saturated compacted clay-sand mixtures at different compaction conditions (compaction energy, water content). This will make it easier for engineers to better design embankment dams, roads and containment barrier systems that utilize these mixtures in their construction.

2.6 Moisture-density relationships for soils

Compaction is the densification of soils by mechanical manipulation. Soil densification entails expelling air out of the soil, which improves the strength characteristics of soils, reduces compressibility, and reduces permeability. Using a given energy, the density of soil varies as a function of moisture content shown in Table 2.1. This relationship is known as the moisture-density curve, or the compaction curve shown in Fig. 2.1 and 2.3. The energy inputs to the soil have been standardized and are generally defined by Standard Proctor (ASTM D 698 and AASHTO T 99) and Modified Proctor (ASTM D 1557 and AASHTO T 180) tests. These tests are applicable for cohesive soils. For cohesionless soils, the relative density test should be used (ASTM D 4253 and ASTM D 4254). The information below describes the compaction results of both cohesive and cohesionless soils.

2.6.1 Fine-grained soils.

The moisture-density relationship for fine-grained soils (silts and clays) is determined using Standard or Modified Proctor tests. Typical results of Standard Proctor tests are shown in Fig. 2.3 which represents the relationship between the moisture content and the dry density of the soil. At the peak point of the curve, moisture content is called the optimum moisture content, and the density is called the maximum dry density. If the moisture content exceeds the optimum moisture content, the soil is called wet of optimum. On the other hand, if the soil is drier than optimum, the soil is called dry of optimum. Soils compacted on the dry side of optimum have higher strength, stability and less compressibility than the same soil compacted on the wet side of optimum. However, soils compacted on the wet side of optimum have less permeability and volume change due to change in moisture content. The question of whether to compact the soil on the dry side of optimum or on the wet side of optimum depends on the purpose of the construction and construction equipment. For example, when constructing an embankment, strength and stability are the main concern (not permeability); therefore, moisture content on the dry side of optimum should be used. For contractors, compacting the soil on the wet side of optimum is more economical, especially if it is within 2% of the optimum moisture content. However, if the soil is too wet, the specified compaction density will not be reached.

2.6.2. Coarse-grained soils.

When coarse-grained, soils (sands and gravels) are compacted using standard or modified Proctor procedures, the moisture-density curve is not as distinct as that shown for cohesive soils in Fig. 2.1., 2.2 and Fig. 2.3 shows a typical curve for cohesionless materials, exhibiting what is often referred to as a hump back or camel back shape. It can be seen that the granular material achieves its densest state at 0% moisture, then decreases to a relative low value, and then increases to a relative maximum, before decreasing again with increasing water content. A better way of representing the density of cohesionless soils is through relative density. Tests can be conducted to determine the maximum density of the soil at its densest state and the minimum density at its loosest state (ASTM D 4253 and D 4254). The relative density of a field soil, *Dr*, can be defined using the density measured in the field, through a ratio to the maximum and the minimum density of the soil.

2.7 Effect of CBR value on Clay-Sand and Clay-Silt Mixtures

The performance of a pavement depends on the quality of its subgrade and subbase layers shown in figure 2.5 and Table 2.2 and 2.3. As the foundation for the pavement's upper layers, the subgrade and subbase layers play a key role in mitigating the detrimental effects of climate and the static and dynamic stresses generated by traffic. Therefore, building a stable subgrade and a properly drained subbase is vital for constructing an effective and long lasting pavement system. The subgrade, the layer of soil on which the subbase or pavement is built, provides support to the remainder of the pavement system. It is crucial for highway engineers to develop a subgrade with a California Bearing Ratio soaked CBR value of at least 10. The most significant applications of soil reinforcement are in road construction. Sub grade soil and its properties are very important in the design of road pavement structure.

Its main function is to give adequate support to the pavement from beneath. Therefore, it should have a sufficient load carrying capacity. Conventional laboratory soaked CBR test has been widely used for predicting bearing capacity of subgrade layer for pavement design. In unsaturated soil, suction is one of the key parameters for understanding the soil behavior. The analysis of soaked CBR is commonly presented in soaked CBR-water content relation. The standard compacted test on various proportions of sand-clay mixtures starting from 0% (pure sand), 5%, 10%, and 20% of sand were used. The tests were performed with different value of water content in both soaked conditions. Kumar and Tabor (2003) studied the strength behavior of silty clay for varying degree of compaction. Shale as described by De-Graft Johnson et al. (1973) is the product of highly consolidated clay silt and sand or a mixture of all the three fractions of soil derived from the weathering of rocks. To improve the performance of soil with low bearing capacity, cement has been used by many researchers (Ismail, 2002; Baisha, 2005; Kolias, 2005; Chen, 2009). Some studies have also been carried out by researchers (Yetimoglu, 2004; Park, 2005; Tang, 2007; Sivakumar Babu, 2008) to study the influence of fiber inclusion on the mechanical behavior of cemented soil. Soaked CBR value is used as an index of soil strength and bearing capacity. This value is broadly used and applied in design of the base and the sub-base material for pavement. Soaked CBR value is a familiar indicator test used to evaluate the strength of soils for these applications (Nicholson et al., 1994). Soaked soaked CBR values of untreated compacted soils need to be interpreted in the context of the general relationship between the soaked CBR-values and the consistency (quality) of the soils used in pavement applications (Bowles, 1992). Soaked CBR values ranging from 3 to 7% are considered as a poor to fair consistency.

2.7.1 Flexible Pavement System

The performance of pavements depends upon the quality of subgrades and subbases. A stable subgrade and properly draining subbase help produce a long-lasting pavement. A high level of spatial uniformity of a subgrade and subbase in terms of key engineering parameters such as shear strength, stiffness, volumetric stability, and permeability is vital for the effective performance of the pavement system. Pavement system consists of the pavement and foundation materials, which are layers of surface course, Base Course, Subbase and Subgrade (Compacted embankment fill) Shown in Fig. 2.5. Subgrade (Compacted embankment fill). Consists of the naturally occurring material on which the road is built, or the imported fill material used to create an embankment on which the road pavement is constructed. Subgrades are also considered layers in the pavement design, with their thickness assumed to be infinite. Pavement systems generally start to deteriorate from the bottom (subgrade), which often determines the service life of a road. Subgrade (Compacted embankment fill) performance generally depends on two interrelated characteristics:

1. Load-bearing capacity. The ability to support loads is transmitted from the pavement structure, which is often affected by degree of compaction, moisture content, and soil type.

2. Volume changes of the subgrade. The volume of the subgrade may change when exposed to excessive moisture or freezing conditions.

In determining the suitability of a subgrade (Compacted embankment fill), the following factors should be considered:

- General characteristics of the subgrade soil
- Depth to water table
- Compaction that can be attained in the subgrade
- Soaked CBR values of compacted subgrades
- Presence of weak or soft layers or organics in the subsoil

2.7.2 Suitable subgrade (Compacted embankment fill) soil

Suitable soils are used throughout the fill and under the prepared subgrade. Suitable soils may be used in the prepared subgrade if they meet the requirements of select subgrade soils or are stabilized to meet those requirements CBR ≥ 10 shown in Table 2.3. Suitable soils must meet all of the following condition:

A. Standard Proctor Density ≥ 95 pcf

Table 2.1 Maximum dry density and optimum moisture content (typical for standard compaction energy).

AAHSTO Classification	Maximum Dry Density (pcf)	Moisture Content (%)
A-1	115-135	7-15
A-2	110-135	9-18
A-3	110-115	10-18
A-4	95-130	10-20
A-5	85-100	15-30
A-6	95-120	10-25
A-7	85-115	15-30

Table 2.2 AASHTO Soil Classification (Yoder & Witczak, "Principles of Pavement Design", 1975)

AAHSTO Symbol	Soaked CBR Range
A-7-6	1 – 5
A-7-5	2 – 8
A-6	5 – 15
A-5	8 – 16
A-4	10 – 20
A-3	15 – 35
A-2-7	10 – 20
A-2-6	10 – 25
A-2-5	15 – 30
A-2-4	20 – 40
A-1-b	35 – 60
A-1-a	60 – 80

Table: 2.3: Suitability of soils for subgrade applications

Subgrade Soils for Design	Unified Soil Classifications	CBR Range
Crushed Stone	GW, GP, and GU	30 to 80
Gravel	GW, GP, and GU	30 to 80
Silty gravel	GW-GM, GP-GM, and GM	20 to 60
Sand	SW, SP, GP-GM, and GM	10 to 40
Silty sand	SM, non-plastic and >35% silt	5 to 30
Silt	ML, >50% silt, liquid limit <40, and PI <10	1 to 15
Clay	CL, liquid limit >40 and PI >10	1 to 15

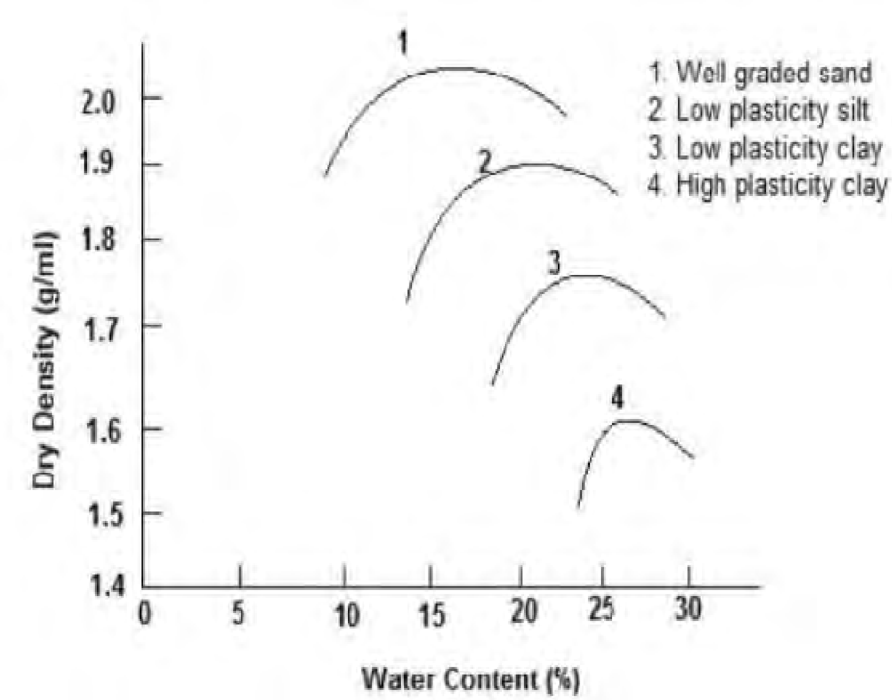

Fig. 2.1 Dry density vs water content (Spangler and Handy, 1982)

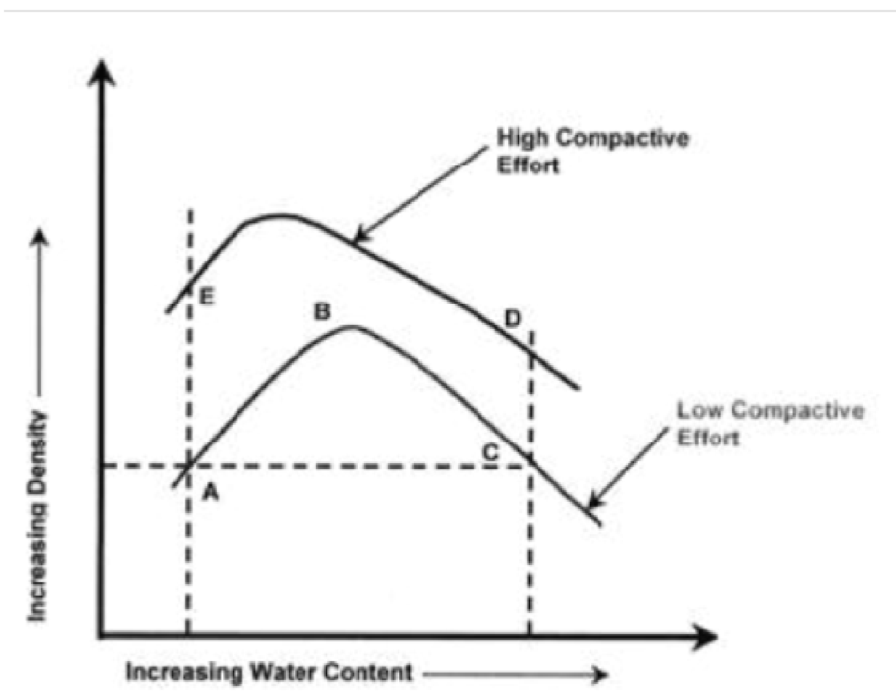

Fig. 2.2 Dry density vs water content (Spangler and Handy, 1982)

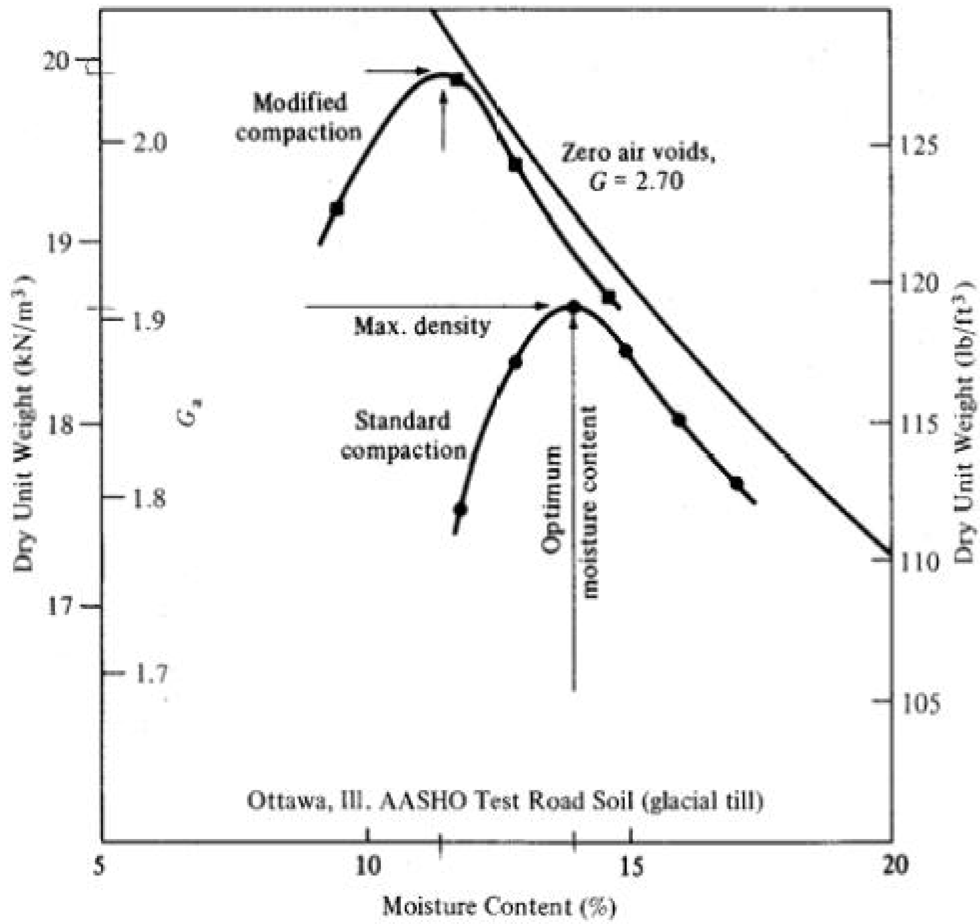

Fig. 2.3 An example of standard and modified Proctor moisture-density curves for the same soil (Spangler and Handy 1982)

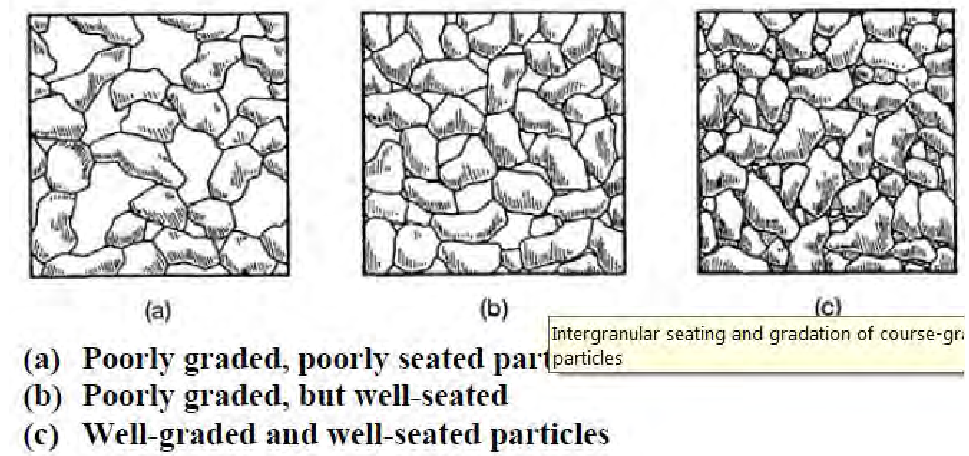

Fig 2.4 Inter-granular seating and gradation of coarse-grained particles

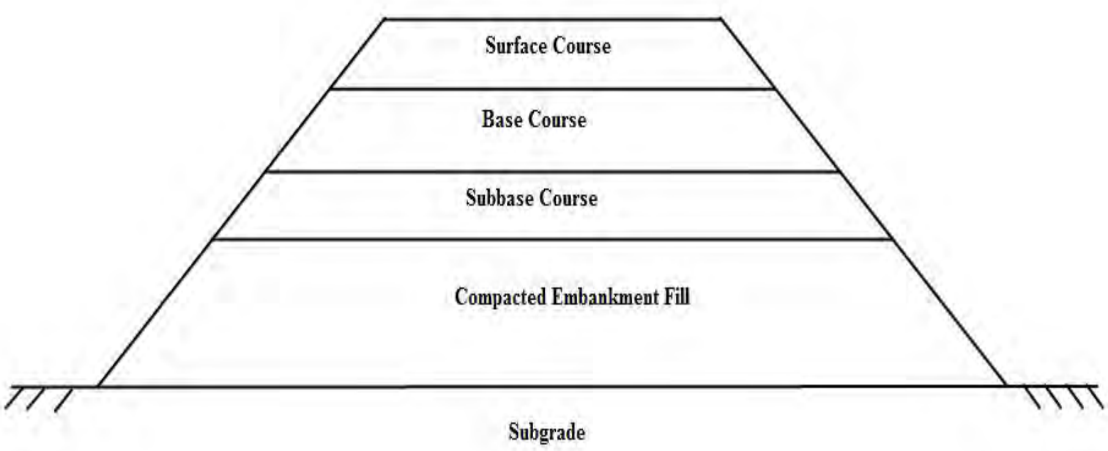

Fig 2.5 Typical section for a flexible pavement

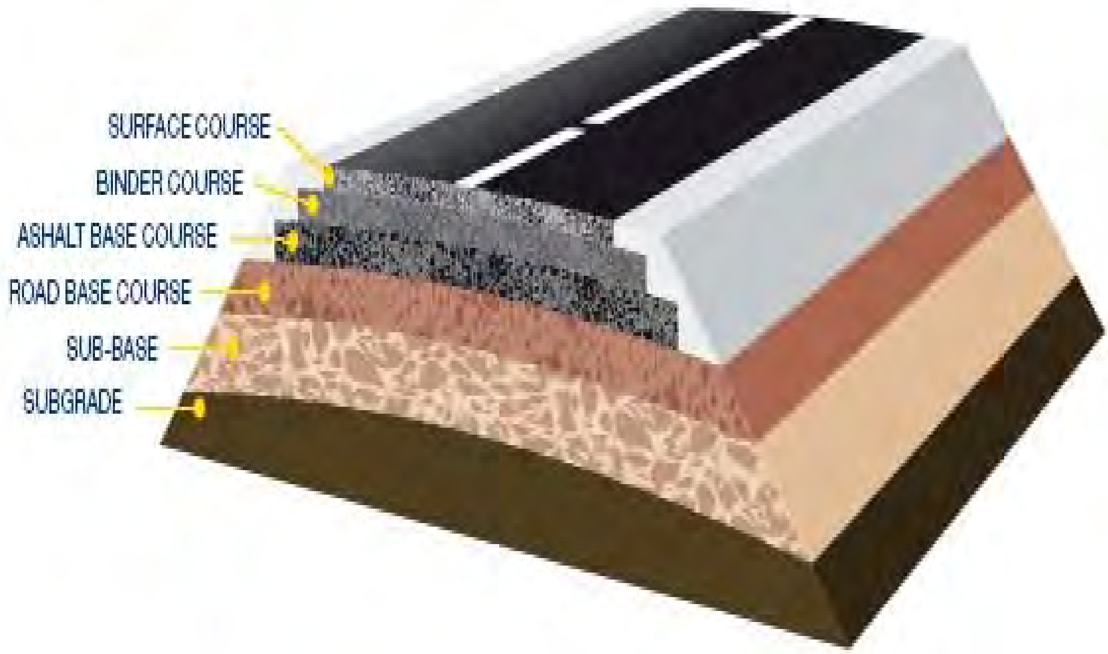

Fig 2.6 A Flexible pavement structure

CHAPTER THREE
LABARATORY INVESTIGATION

3.1 General

This chapter describes the laboratory testing program. To know the effect of Sand and Silt content on the plasticity and compaction behavior of Clay, several tests have been performed including specific gravity test of Sand, Silt and Clay, hydrometer test of Lean Clay and Silt it has shown in Fig. 3.1, Atterberg limits test, compaction test and Soaked CBR test of Lean Clay soil. Atterberg limits test, compaction test and Soaked CBR test Lean Clay-Sand mixture and Lean Clay-Silt mixture shown in Table 3.1. Three soil samples were collected for the study. Sand was collected from Maghna River, Lean Clay was collected from Mirpur region and Nonplastic Silt sample was collected from Keraniganj have been shown in Fig. 3.1 and Table 3.2. Finally, the test results were compared.

3.2 Preparation of Test Sample

During first test pertaining 10% Sand + 90% Lean Clay and second test pertaining 10% Silt + 90% Lean Clay, the plastic limit test was performed using material left over from the thoroughly mixed portion of the soil prepared for the liquid limit test, which normally was at moisture content higher than the plastic limit. It sets the sample aside and allowed to air dry until the liquid limit test has been completed. However, if the sample was too dry to permit rolling to a ⅛ in. (3 mm) thread, water had been added, thoroughly remixed and seasoned in air prior to doing the test. After performing this Sand-Lean Clay mixture, the above procedure on 20% Sand + 80% Lean Clay as well as from 30% Sand + 70% Lean Clay mixture to 50% Sand + 50% Lean Clay mixture. Similarly, Silt-Lean Clay mixture, the above procedure on 20% Silt + 80% Lean Clay as well as from 30% Silt + 70% Lean Clay mixture to 30% Lean Clay mixture. Secondly, same procedure was maintained for 20% Sand to 80% Lean Clay as well as from 30% Sand + 70% Lean Clay to 90% Sand + 10% Lean Clay. Similarly, was maintained for 20% Silt to 80% Lean Clay as well as from 30% Silt + 70% Lean Clay to 90% Silt + 10% Lean Clay.

3.3 Laboratory Tests

All the tests performed at the geotechnical laboratory of BUET were to determine the Liquid limit, Plastic limit of the collected samples. In order to identify the grain size distribution and specific gravity (G_s) were determined. Besides, water content (w_n), liquid limit (w_L), plastic limit (w_p), and grain size distributions were determined using collected soil samples. Soaked CBR tests were performed on different percentage of mixture soil samples for the determination of soaked CBR values for comparing different soil according to USCS. The laboratory test programs of different soil samples are shown in Table 3.1.

3.4 Compaction Test of Lean Clay-Sand and Lean Clay-Silt Mixture

The Standard proctor compaction test was a laboratory method of experimentally determining the optimal moisture content at which a given soil type will become denser and achieve its maximum dry density. The compaction effort depends on the amount of water the soil contains during soil compaction. The test was most commonly referred to as the standard Proctor compaction test. After determining the dry density and optimum moisture content from tests, the graphical relationship of the dry density to moisture content was then plotted to establish the compaction curve. The maximum dry density was finally obtained from the peak point of the compaction curve and its corresponding moisture content, also known as the optimal moisture content.

3.5 Soaked CBR Test

This test method covers the determination of the soaked CBR (California Bearing Ratio) of pavement subgrade, subbase, and base course materials from laboratory compacted specimens. The test method was primarily intended for (but not limited to) evaluating the strength of materials having maximum particle sizes less than 3/4 in. (19 mm). Standard References ASTM D 1883 Load was applied on the penetration piston so that the rate of penetration is approximately 0.05 in. (1.27 mm)/min. Record the load readings at penetrations of 0.025 in. (0.64mm), 0.050 in. (1.27mm), 0.075 in. (1.91 mm), 0.100 in. (2.54 mm), 0.125 in. (3.18 mm), 0.150 in. (3.81 mm), 0.175 in.

(4.45 mm), 0.200 in. (5.08 mm), 0.300 in. (7.62 mm), 0.400 in. (10.16 mm) and 0.500in. (12.70 mm). Note the maximum load and penetration if it occurs for a penetration of less than 0.500 in. (12.70 mm). With manually operated loading devices, it may be necessary to take load readings at closer intervals to control the rate of penetration. Measure the depth of piston penetration into the soil by putting a ruler into the indentation and measuring the difference from the top of the soil to the bottom. If the depth does not closely match the depth of penetration gauge, determine the cause and test a new sample.

3.5.1 CBR Determination

Using corrected stress values taken from the stress penetration curve for 0.100 in. (2.54 mm) and 0.200 in. (5.08 mm) penetrations, calculate the soaked CBR values for each by dividing the corrected stresses by the standard stresses of 1000 psi (6.9 MPa) and 1500 psi (10.3 MPa) respectively, and multiplying by 100. Also, calculate the soaked CBR values for the maximum stress, if the penetration is less than 0.200 in. (5.08 mm) interpolating the standard stress. The bearing ratio reported for the soil is normally the one at 0.100 in. (2.54 mm) penetration. When the ratio at 0.200 in. (5.08 mm) penetration is greater, rerun the test. If the check test gives a similar result, use the bearing ratio at 0.200 in. (5.08 mm) penetration.

Table 3.1 Laboratory Testing Program

Item	Laboratory test performed
Lean Clay	Specific gravity test, Hydrometer test, Atterberg limits, Compaction test, Soaked CBR test
Sand	Specific gravity test, grain size distribution, Soaked CBR test
Nonplastic Silt	Specific gravity test, Hydrometer test, Soaked CBR test
Sand-Clay Mixture	Liquid limit test & Plastic limit test
Silt-Clay Mixture	Liquid limit test & Plastic limit test
Sand-Clay Mixture	Compaction test
Silt-Clay Mixture	Compaction test
Sand-Clay Mixture	Soaked CBR test
Silt-Clay Mixture	Soaked CBR test

Table 3.2 Collected three soil samples

Type of Soil	Collected from	Procedure of collection
Lean Clay	Mirpur	Passing # 200 sieve
Nonplastic Silt	Karanigonj	Passing # 200 sieve
Fine Sand	Meghna	Passing # 200 sieve

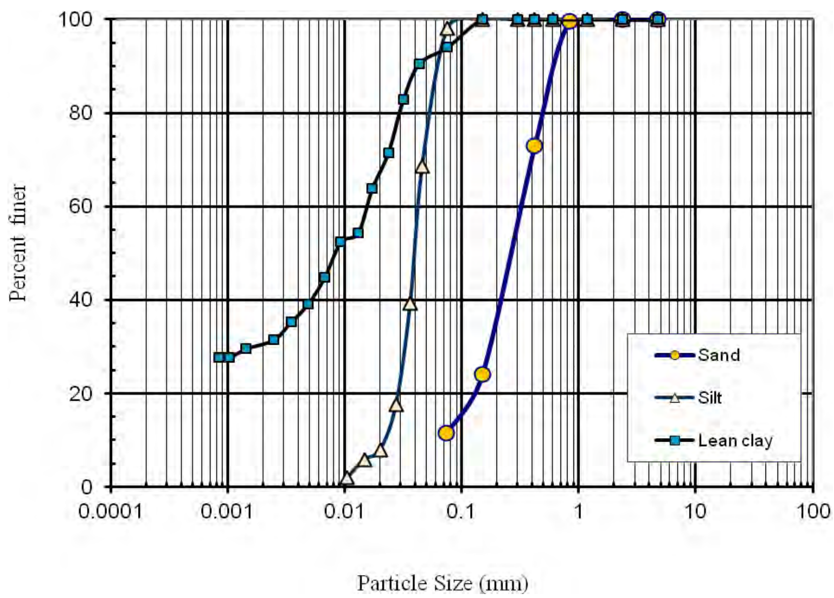

Fig. 3.1 Grain Size Distribution Curve of Soil Samples

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 General

Laboratory test data on soil samples obtained from three samples were analyzed to develop soil profile along the study area**.** Effect of Sand content on Lean Clay and effect of Silt content of Lean Clay can be determined by adding a relative percentage of Sand and Silt samples with Lean Clay sample or vice-versa. Also, additional knowledge that had been learned throughout this dissertation will be analyzed from geotechnical point of view as well as natural circumstances. The laboratory test results on different mixed soil samples were presented in this chapter. This chapter mainly covers the broad discussion on results of the tests related with this dissertation.

4.2 Laboratory Test Results and Discussions

Three soil samples were collected for the study (Nonplastic Silt, Sand and Lean clay), all samples were taken to laboratory and following test were performed.

4.2.1 Specific gravity Test of Silt and Lean Clay

Firstly Silt and Lean Clay soil sample were passing #200 sieves. Then the laboratory work to determine the specific gravity of soil using the volumetric flask is a somewhat an indirect method. Data of Specific Gravity were mentioned in Appendix A. It was found the value of specific gravity of Silt 2.62 and Lean Clay 2.63. This was well-known which value should be within 2.65 and 2.67 for Silt and Lean Clay

4.2.2 Grain size distribution of Soil Sample

A sieve analysis consists of shaking the soil through a stack of wire screens with openings of known size; the definition for particle diameter for a sieve test was, therefore, the side dimensions a square hole. The data was plotted on a semi log plot of percent finer vs. grain diameters in Fig. 4.1. This test was performed using mechanical method of grain size analysis. When tested at the laboratory the amount of material passing #200 sieves was not greater than 1.5% by weight. The value of

Uniformity Coefficient (C_u) and Coefficient of Curvature (C_c) of sand were 6.18 and 1.73 respectively for the sand used in the study.

4.2.3 Identification of Lean Clay

It was collected a clay sample but didn't know its type. Its color was radish brown. Its physical appearance made thinking on the possibility of lean clay. Firstly this soil sample was passing #200 then LL and PL test were performed in laboratory to ensure about its true type. Those results were adorned in Table 4.1 it was found the value of LL, PL & PI were 37, 15 & 22 respectively. As it was known Lean Clay of LL should be less than fifty and PI should be greater than seven. Above results meet all the requisite quality to confirm this clay sample as Lean Clay.

4.2.4 Atterberg Limits

ASTM D4318-86 described method of Atterberg Limits Test was performed on Clay-Sand mixture and Clay-Silt mixture to determine liquid limit (LL), plastic limit (PL), and plasticity index (PI). The liquid limit test was performed using Casagrande's apparatus. A summary of the liquid limit (w_L), plastic limit (w_p) and Plasticity Index were shown in Table 4.2 and 4.4.

4.3 Effect of Sand Content on Plasticity Behavior of Lean Clay

Atterberg limits test describes the plasticity behavior of any sort of soil mixture. Here, when sand content is just enough to fill the voids of the granular portion at its maximum porosity, the structure of the mixture changes and the linear relationship between the Atterberg limits (plastic and liquid limit) and the clay content was no more valid and soil changed its behavior from clay to sand. In order to notice the change, it was charted the value of LL, PL & PI with Sand in Table 4.2 and from this test data, it was drawn Fig. 4.2, 4.3 and 4.4 to witness the plasticity behavior of those soil mixture.

4.3.1 Variation of Liquid Limit on Sand-Lean Clay Mixture

In liquid limit test, Casagrande observes the number of drops from the cup necessary to close the groove depended upon the water content of the soil in the cup and that when the results of a series of determinations for anyone soil was plotted water content versus the number of drops of the cup, which the point fell on a straight line. Such a curve is named as flow curve. The data and corresponding flow curve of lean clay-sand mixture was furnished in Appendix A. It has only been able to perform five test i.e. Sand 10% + Clay 90% mixture to Sand 50% + Clay 50% mixture. Beyond this proportion, it was impossible to determine liquid limit as cohesion was found to be ignorable. In that case, some soils tended to slide on the surface of the cup instead of flowing. More water was added to the sample and remixed. But after doing this, soil continued to slide on the cup at a lesser number of blows than 25. So, liquid limit could not be determined. At that time, effect of lean clay is minimized on mixture and the soil converted into sandy lean clay. In Fig. 4.2 and Table 4.2 it was observed that there was a decrease in liquid limit with the increase of sand content. The curve was slight upward slightly in 40% sand content. Again it was decrease until the end of the 50% sand content.

4.3.2 Variation of Plastic Limit on Sand-Lean Clay Mixture

The results found from the PL test of sand- lean clay mixture. Similar to liquid limit test, it were able to do only five test i.e. from Clay 90% + Sand 10% to Clay 50% + Sand 50%. As the diameter of the thread should be 3mm, but when it mixed 40% Clay with 60% Sand, it was unable to form a 3mm thread of that sample. It clearly indicated that the mixture turned into sandy condition and the effect of lean clay was gradually decreased and the soil mixture converted into sandy lean clay. Then, it was stopped the project of doing plastic limit test after the result of 50% Clay and 50% Sand. It was plotted plastic limit vs. percentage of sand content in Fig. 4.3 and Table 4.2. In that figure the curve was almost steady with the increases of sand content. But slightly decreased that was 20% to 40% with the increases of sand content.

4.3.3 Variation of Plasticity Index on Lean Sand-Lean Clay Mixture

A graph was plotted as PI vs. percentage of Sand content in Fig. 4.4. In that graph, the curve was steady from 0% to 20% of sand content. After that, PI values decreased with the increase of sand content PI is an important factor to know the AASHTO soil classification. It was classified sol mixture according to the values of PI. 90% lean clay and 10% sand mixture was classified as lean clay where 50% clay and 50% sand was classified as sandy lean clay. These data were given in Table 4.6

4.4 Effect of Silt Content on Plasticity behavior of Lean Clay

Atterberg limits test describes the plasticity behavior of any sort of soil mixture. It was the value of LL, PL & PI with Silt content shown in Table 4.4. And from the test data, it was drawn Fig. 4.7, 4.8 and 4.9 to witness the plasticity behavior of those soil mixtures.

4.4.1 Variation of Liquid Limit on Silt-Lean Clay Mixture

The data and corresponding flow curve of silt-lean clay mixtures were furnished in Appendix A. It had only been able to perform seven test i.e. Silt 10% + Clay 90% mixture to Silt 70% + Clay 30% mixture. Beyond this proportion, it was impossible to determine liquid limit as cohesion was found to be ignorable. In that case, some soils tended to slide on the surface of the cup instead of flowing. More water was added to the sample and remixed. But after doing this, soil continued to slide on the cup at a lesser number of blows than 25. So, liquid limit could not be determined. At that time, effect of lean clay was minimized on mixture and the soil converted into silty clay. It was plotted as liquid limit vs. Percentage of Silt Content in Fig. 4.7. It was observed that there was a downward trend in liquid limit with the increases of

4.4.2 Variation of Plastic Limit on Silt-Lean Clay Mixture

Similar to liquid limit test, i.e. from Clay 90% + Silt 10% to Clay 30% + Silt 70%. As the diameter of the thread should be 3mm, but when mixed 20% Clay with 80% Silt, it was unable to form a 3mm thread of that sample. It clearly indicated that the mixture turned into non-plastic silty condition and the effect of lean clay was gradually decreased and the soil mixture converted into silty lean clay. It was plotted Plastic limit vs. percentage of silt content in Fig. 4.8 and found that plastic limit was increased up to 10% silt content. After that it was almost steady curve with some fluctuations.

4.4.3 Variation of Plasticity Index on Silt-Lean Clay Mixture

A graph was plotted as PI vs. percentage of Silt in Fig. 4.9. In this graph, a gradual declined trend was observed. The value of PI increases from 10% to 20% silt .content. After that, PI value was decreased with the increases of silt content. PI is an important factor to know the AASHTO soil classification. These data were given in Table 4.7

4.5 Effect of Sand Content on Compaction behavior of Lean Clay

Nearly all soils exhibit a similar relationship between moisture content and dry density when subjected to a given compactive effort. A typical compaction curve presents different densification stages when the soil was compacted with the same apparent energy input but different water contents. The water content at the peak of the curve was called optimum water content and represents the water content in which dry density was maximized for a given compaction energy. When the moisture content was less than optimum, the soil was more difficult to compact. Beyond optimum, most soils were not as dense under a given effort because the water interferes with the close packing of the soil particles. Beyond optimum and for the stated conditions, the air content of most soils remains essentially the same, even though the moisture content was increased. The moisture-density relationship was indicative of the workability of the soil over a range of water contents for the compactive effort used. The relationship is valid for laboratory and field compaction. The test data collected from compaction tests on various proportions of lean clay and sand and Fig. are shown in appendix A.

Each of those tests designates same curve as like as conventional Moisture-Density Relationship curve. But it was hard to find more than three values when soil mixture As the sand percentage increased in soil mixture, weight of compacted soil changed quickly. It was well-known that, the reading should be stopped when the weight of compacted soil was just decreased from its previous reading

4.5.1 Variation of Maximum Dry Density on Sand-Lean Clay Mixture

Table 4.3 shows the increases in Maximum Dry Density with the increase of percentage of sand content. As well as, Fig. 4.5 represents Maximum Dry density vs. Percentage of Sand Content. In this figure, there was a upward trend observed from 10% to 100% of sand content. A gradually increased curve has been found in Maximum Dry Density vs. Percentage of Sand Content graph. It indicates that the soil sample will be getting hardened with the adding of water. The more moisture was absorbed the more it will strong.

4.5.2 Variation of OMC on Sand-Lean Clay Mixture

A graph mentioning optimum moisture content vs. percentage of sand content was drawn in Fig. 4.6. In the graph it was found that OMC decreasing from 10% to 50% sand content. But the OMC suddenly increased with the increases of sand content until 70%. Then, it was steady. This was because soil mixture was bound to absorb more sand content as the project was moving on and by the time being lean clay turned into sandy lean clay, there was an unusual change occurred. But a good observation was found that OMC always decreased with the increases of percentage of sand content by avoiding some abnormal situation.

4.6 Effect of Silt Content on Compaction behavior of Lean Clay

The mixture was started containing from 10% Clay + 100% Silt to 100% clay + 0% silt as the silt percentage increased in soil mixture, weight of compacted soil changed quickly. It was well-known that, the reading should be stopped when the weight of compacted soil is just decreased from its previous reading.

4.6.1 Variation of Maximum Dry Density on Silt-Lean Clay Mixture

Table 4.5 shows the increase in Maximum Dry Density with the increases percentage of silt content. As well as, Fig. 4.10 represents Maximum Dry density vs. Percentage of Silt Content. In the figure, there was a slight upward trend have been observed in the graph. A gradually increased curve has been found in Maximum Dry Density vs. Percentage of Silt Content graph. It indicates that the soil sample will be getting hardened with the adding of water. The more moisture is absorbed the more it will strong.

4.6.2 Variation of OMC on Silt-Lean Clay Mixture

A graph mentioning optimum moisture content vs. percentage of silt content was drawn in Fig. 4.11. Abrupt curve was found where no clear indication of change of soil mixture with optimum moisture content was achieved. After decreasing OMC from 30% silt to 100% silts. The curve suddenly increases from 60% with the increase in moisture content until 80% silt content arrives. Then, it was downward and steady. But a good observation was found that by doing this test was that OMC will always decrease with the increases percentage of silt content by avoiding some abnormal situation.

4.7 Effect of Soaked CBR value on Soil Sample

This test method was used to evaluate the potential strength of subgrade, subbase, and base course material, including recycled materials for use in road and airfield pavements. A soaked CBR of 3 equates to tilled farmland, a soaked CBR of 4.75 equates to turf or moist clay, while moist sand may have a soaked CBR of 10. High quality crushed rock has a CBR over 80. CBR values for different soil according to USCS are categorized in Table 2.2 and 2.3. This Table signifies the lower CBR for fine-grained soil and higher CBR for coarse-grained soil. It has done CBR on soaked samples.

4.7.1 Effect of CBR value on Lean Clay-Sand Mixture

The sample was prepared on the basis of mixing ratio of lean clay & sand as mentioned earlier. But due to shortage of samples, only three tests were done (Compacted by Standard Effort) and six tests were done for the study. The mixing ratios are: Lean Clay 90% + Sand 10%, Lean Clay 80% +Sand 20%, Lean Clay 70%

+Sand 30% (Compacted by Standard Effort). And the mixing ratios are Lean Clay 90% + Sand 10%, Lean Clay 70% + Sand 30%, Lean Clay 50% + Sand 50%, Lean Clay 30% + Sand 70%, Lean Clay 10% + Sand 90%, Lean Clay 0% + Sand 100% both test results were shown in Table 4.8 and Table 4.9 and in Fig. 4.12 and Fig. 4.13. In the graph it has been observed that the soaked CBR values were high when the percentage of sand was high and soaked CBR value reduces gradually with the reduction of sand in the Clay-Sand mixture. So, a decision can be made that the samples having higher percentage of sand will be used as good subgrade

4.7.2 Effect of CBR value on Lean Clay-Silt Mixture

Due to shortage soil samples, only three tests were done (Compacted by Standard Effort) and six tests were done for the study. The mixing samples (Lean Clay-Silt) are: Lean Clay 0% + Silt 100%, Lean Clay 90% + Silt 10%, Lean Clay 80% + Silt 20%. And the mixing ratios are Lean Clay 90% + Silt 10%, Lean Clay 70% + Silt 30%, Lean Clay 50% + Silt 50%, Lean Clay 30% + Silt 70%, Lean Clay 10% + Silt 90%, Lean Clay 0% + Silt 100% the both tests results were shown in Table 4.10 and Table 4.11 and in Fig.4.14 and Fig. 4.15. In the figure it has been observed that the percentage of silt was high then soaked CBR values were high. Soaked CBR value reduces gradually with the reduction of silt content in the Clay-Silt mixture. So, a decision can be made that the samples having higher percentage of silt content will be used as not good subgrade soil.

Table 4.1 Identifications of Soil Sample

Type of Soil	Collected from	Procedure of collection	LL	PL	PI	Sand (%)	Silt (%)	Clay (%)
Lean Clay	Mirpur	Passing # 200 sieve	37	15	22	4	6	90
Silt	Keraniganj	Passing # 200 sieve				2	97	1
Sand	Maghna River	Passing # 200 sieve				100		

Table 4.2 Value of LL, PL and PI of Sand-Clay Mixture

Sand %	LL	PL	PI
0	37	15	22
10	37	15	22
20	35	13	22
30	31	14	17
40	32	14	18
50	28	15	13

Table 4.3 Value of Maximum Dry Density & OMC Sand-Clay Mixture

Sand %	Max Dry density KN/m³	OMC %
0	16.12	18.7
10	15.76	15.9
20	16.45	16.2
30	16.77	13.9
40	16.86	12.7
50	17.11	9.8
60	16.50	9.5
70	17.23	10.9
80	17.39	8.7
90	17.49	8.7
100	17.16	8.3

Table 4.4 Value of LL, PL and PI Silt-Clay Mixture

Silt (%)	LL	PL	PI
0	37	15	22
10	37	21	16
20	36	20	16
30	34	19	15
40	33	19	14
50	32	18	14
60	28	18	10
70	27	18	9

Table 4.5 Value of Maximum Dry Density & OMC Silt-Clay Mixture

Silt (%)	Maximum dry density KN/m³	OMC
0	16.12	18.7
10	15.41	14.50
20	14.80	15.09
30	14.83	14.85
40	15.47	15.41
50	15.37	12.13
60	14.94	13.98
70	15.62	12.32
80	15.44	13.64
90	16.41	10.12
100	16.62	10.0

Table 4.6 AASHTO classification of Sand-Clay mixture

Sand %	Liquid Limit	Plasticity Index	AASHTO Classification	Group Index (GI)
0	37	22	A-6	22
10	37	22	A-6	19
20	35	22	A-6	16
30	31	17	A-6	9
40	32	18	A-6	7
50	28	13	A-6	3
60	NA	NA	A-4	0
70	NA	NA	A-4	0
80	NA	NA	A-3	0
90	NA	NA	A-3	0
100	NA	NA	A-3	0

Table 4.7 AASHTO classification of Silt-Clay mixture

Silt %	Liquid Limit	Plasticity Index	AASHTO Classification	Group Index (GI)
0	37	22	A-6	22
10	37	16	A-6	17
20	36	16	A-6	17
30	34	15	A-6	15
40	33	14	A-6	14
50	32	14	A-6	14
60	28	10	A-6	9
70	27	9	A-5	8
80	NA	NA	A-4	0
90	NA	NA	A-4	0
100	NA	NA	A-4	0

Table 4.8 Soaked CBR of Sand-Clay Mixture

Sand %	Soaked CBR
100	15.0
90	11.5
70	8.5
50	7.5
30	3.0
10	2.3
0	1.1

Table 4.9 Soaked CBR of Sand-Clay Mixture Compacted by Standard Effort

Sand %	Soaked CBR
0	1.8
10	4.9
20	8.8
30	10.8

Table 4.10 Soaked CBR of Silt-Clay Mixture

Silt %	Soaked CBR
100	3.5
90	3.0
70	2.0
50	1.8
30	1.6
10	1.2
0	1.1

Table 4.11 Soaked CBR of Silt-Clay Mixture Compacted by Standard Effort

Silt (%)	Soaked CBR
0	1.8
10	2.6
20	3.0
100	4.9

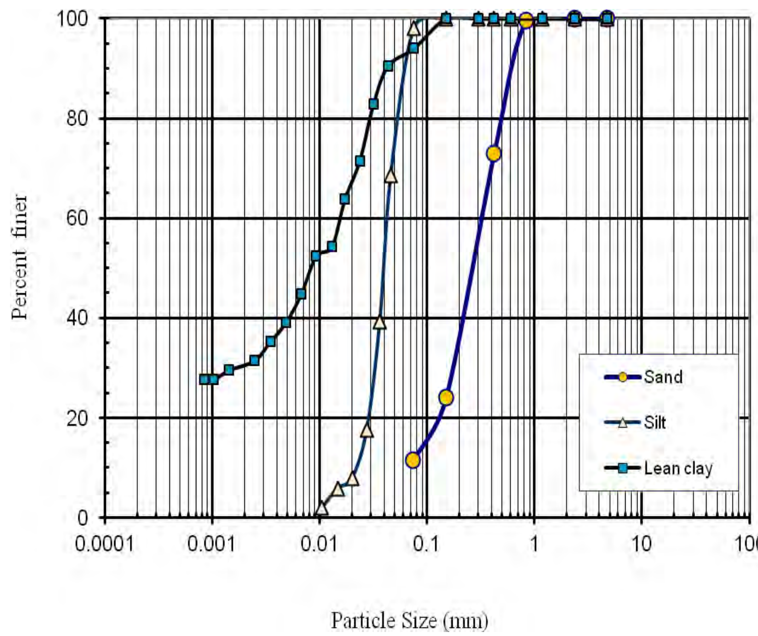

Fig. 4.1 Grain Size Distribution Curve of Soil Samples

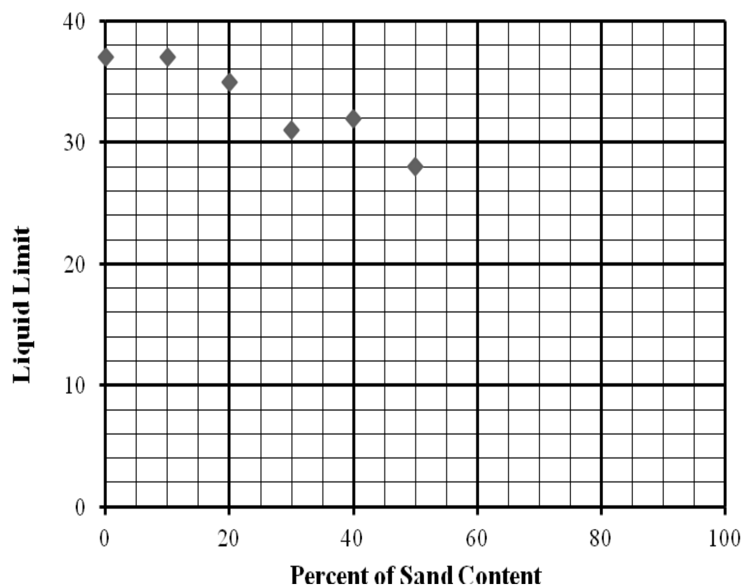

Fig 4.2 Liquid limit vs. Percentage of Sand Content in Sand-Lean Clay mixture

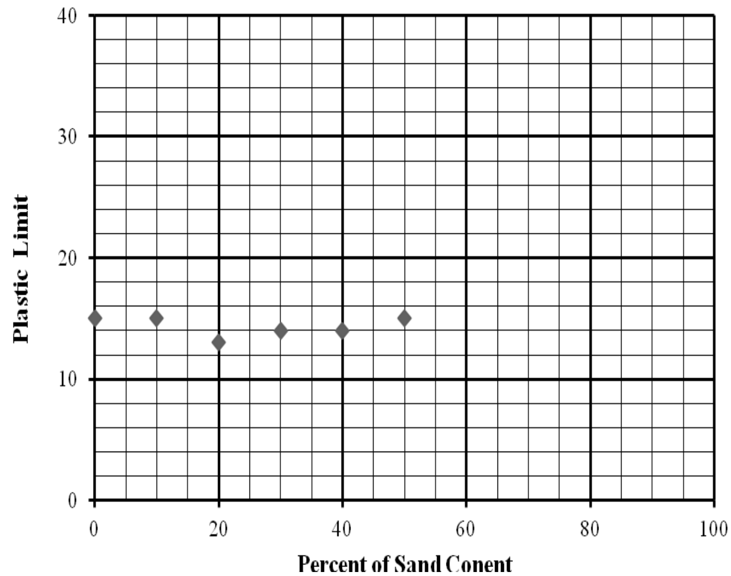

Fig 4.3 Plastic limit vs. Percentage of Sand Content in Sand-Lean Clay mixture

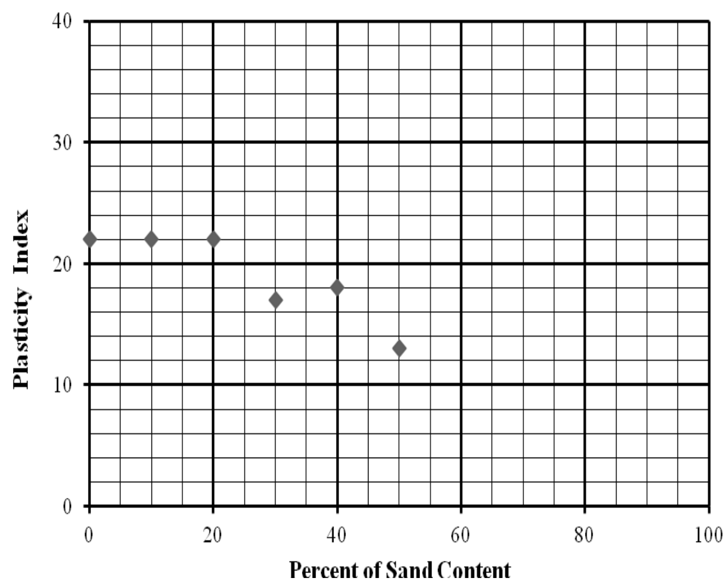

Fig 4.4 Plasticity Index vs. Percentage of Sand Content in Sand-Lean Clay mixture

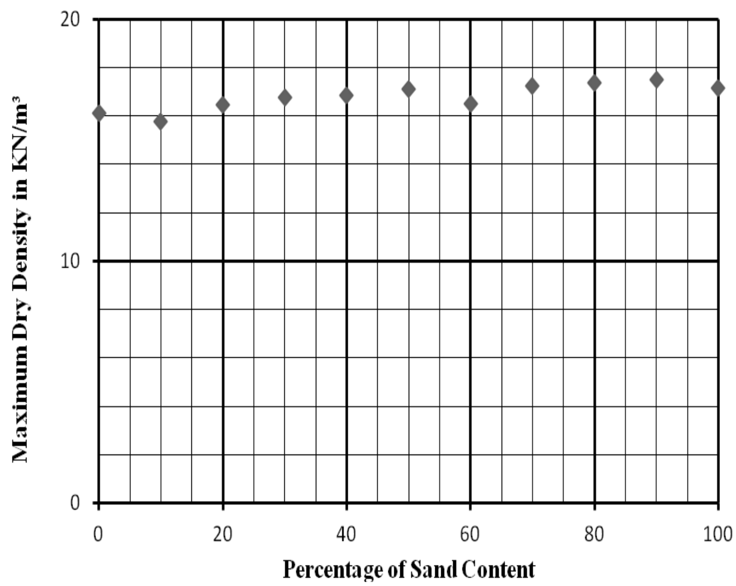

Fig 4.5 Maximum Dry density vs. Percentage of Sand Content in Clay-Sand mixture

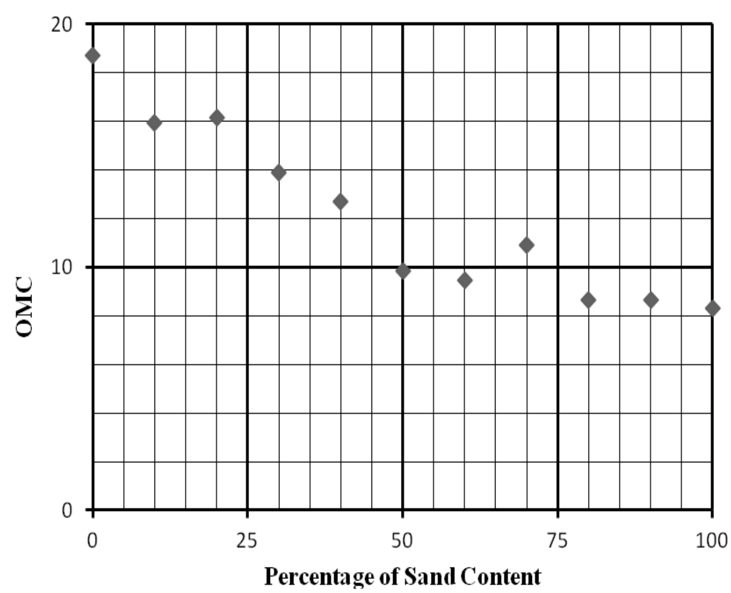

Fig 4.6 OMC vs. Percentage of Sand Content in Clay-Sand mixture

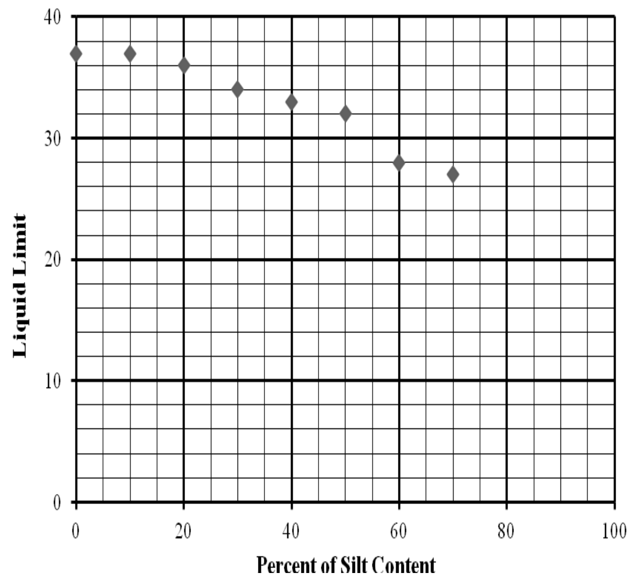

Fig 4.7 Liquid limit vs. Percentage of Silt Content in Clay-Silt mixture

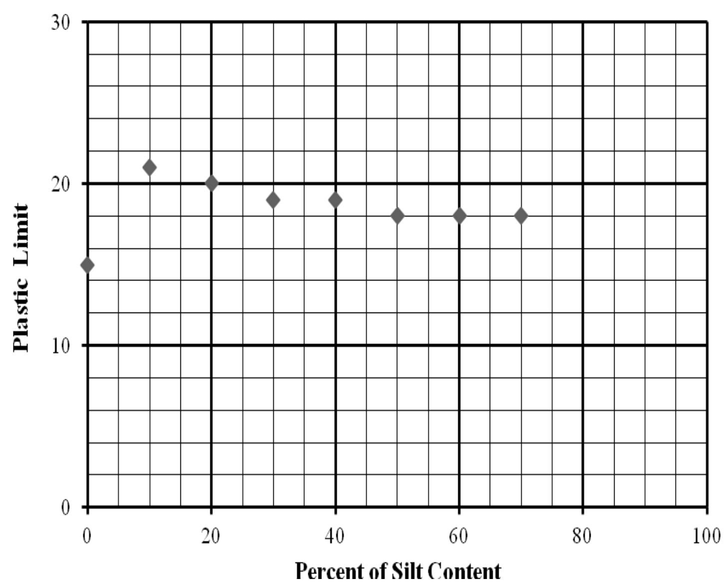

Fig 4.8 Plastic limit vs. Percentage of Silt Content in Clay-Silt mixture

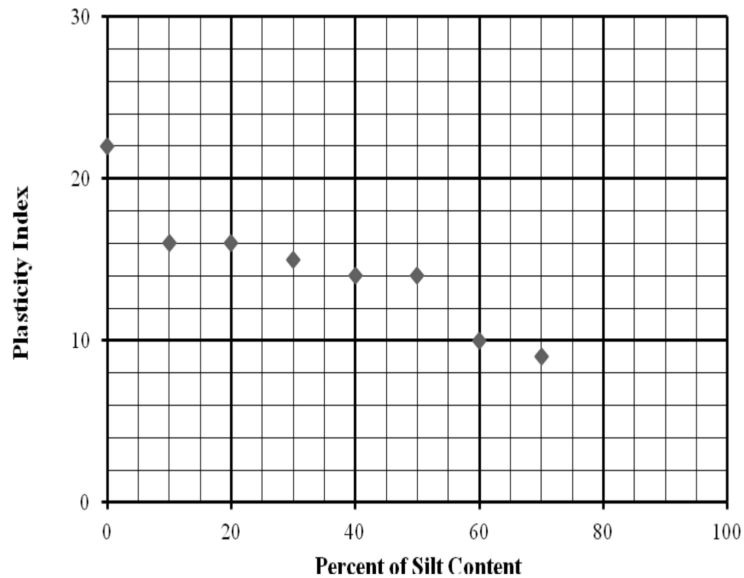

Fig 4.9 Plasticity Index vs. Percentage of Silt Content in Clay-Silt mixture

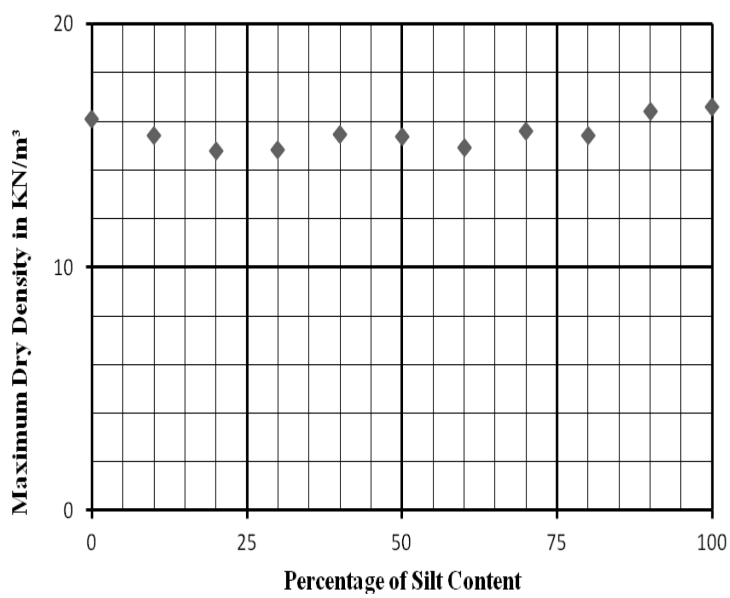

Fig 4.10 Maximum Dry density vs. Percentage of Silt Content in Clay-Silt mixture

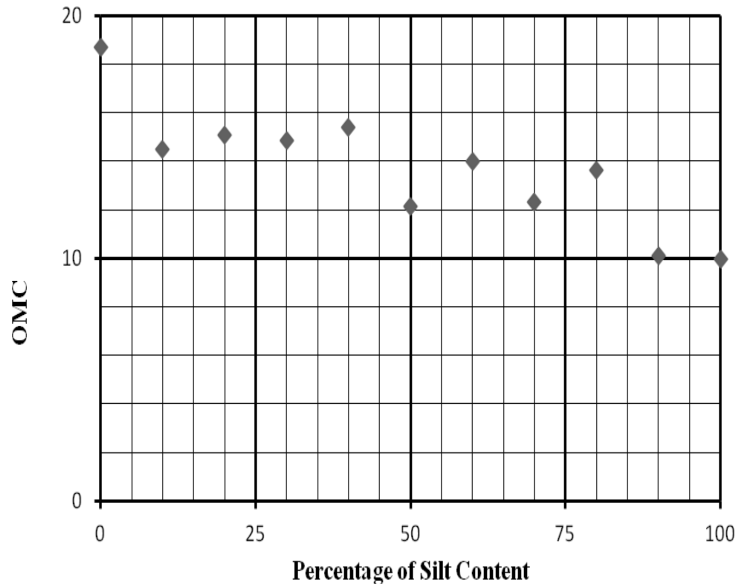

Fig 4.11 OMC vs. Percentage of Silt Content in Clay-Silt mixture

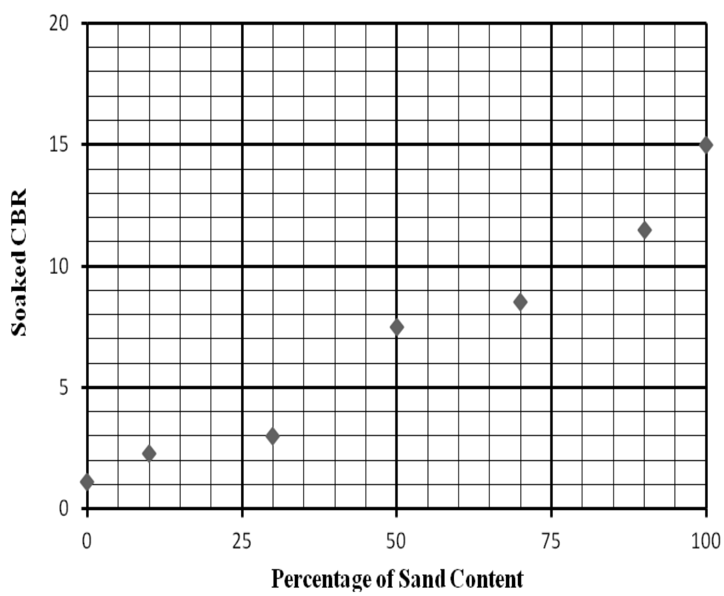

Fig. 4.12 CBR vs. Percentage of Sand Content in Clay-Sand mixture

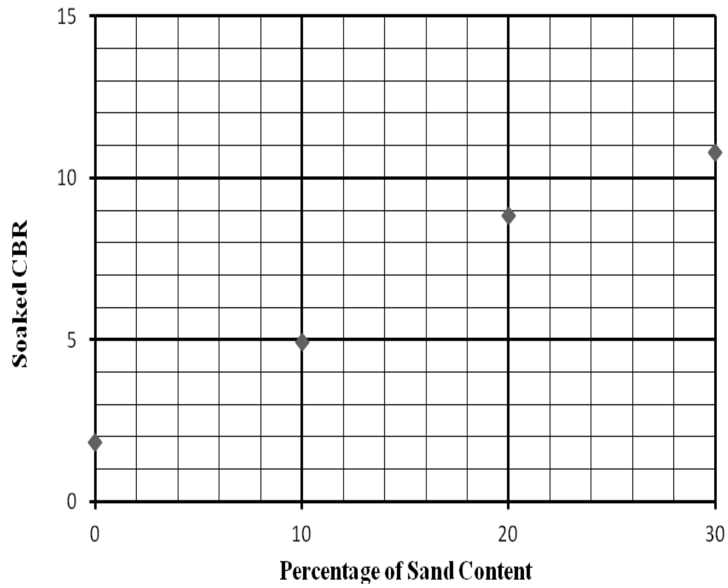

Fig. 4.13 CBR vs. Percentage of Sand Content in Clay-Sand mixture (Compacted by Standard Effort)

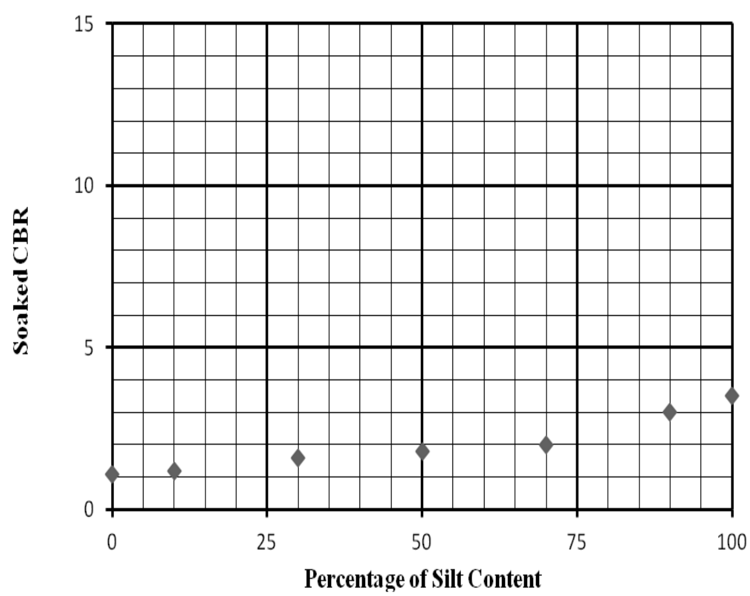

Fig. 4.14 CBR vs. Percentage of Silt Content in Clay-Silt mixture

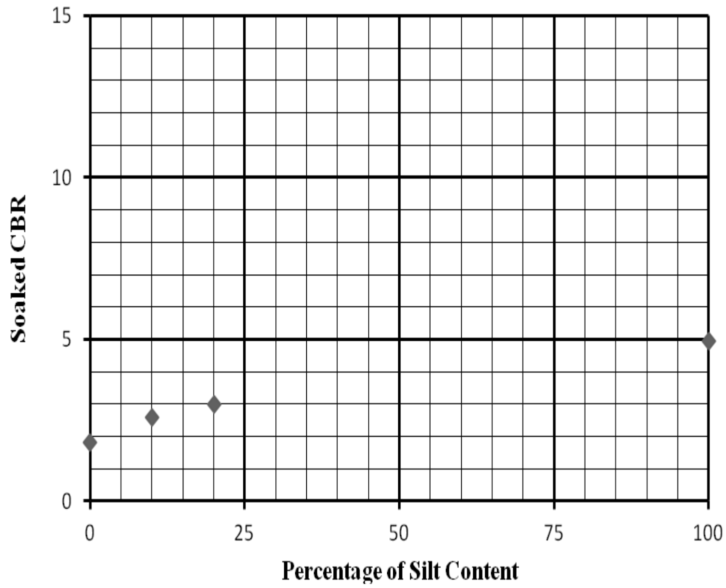

Fig. 4.15 CBR vs. Percentage of Silt Content in Clay-Silt mixture (Compacted by Standard Effort)

CHAPTER FIVE
CONCLUSION AND RECOMMENDATION

5.1 General

The goal of this dissertation is the behavior of the plasticity and compaction in soil mixture constituting clay- sand and clay-silt. In this project work, three soil samples, namely fine sand, nonplastic silt and clay, were collected and all the tests were performed. Sand-Clay and Silt-Clay mixtures were prepared with various proportions. Atterberg Limits, Soaked CBR and Compaction tests were performed on all the mixed soils. Thus suitable subgrade material for road construction was suggested.

5.2 Conclusions

The following conclusions may be drawn with respect to this experimental study:

i. With the increase of sand content in sand-clay mixture and silt content in silt clay mixture, maximum dry density increased but optimum moisture content decreased.

ii. CBR values increased with the increase of the percentage of sand while they reduced gradually with the reduction of sand in the Clay-Sand mixture.

iii. Similarly, CBR values increased with the increase of the percentage of silt while they reduced gradually with the reduction of silt in the Clay-Silt mixture.

iv. When a comparison is drawn between Soaked CBR of Clay-Sand mixture and that of Clay-Silt mixture, it is found that CBR values in the Clay-Sand mixture are much higher. It can thus be concluded that Clay-Sand mixture is a good subgrade material while Clay-Silt mixture is a poor subgrade material for the purpose of road construction.

5.3 Recommendations for future study

From the lessons of the present study, the recommendations for future study may be summarized as follows:

i. Sand-silt-clay mixture can be used to know the suitable proportions for a good subgrade material.

ii. Soil samples from existing roads may be collected and tested to identify the problems of current road construction practices.

iii. Considering climate change, what type of material is more suitable for road and embankment construction would be an interesting topic of research.

REFERENCES

Atterberg, A. (1909). “Die Bestandteile der mineralboden, die analyses, classification und haupteigenschaften der tonartigen boden.” Comptes Renders de la Premiere Conference International Agrogeologique, pp. 289-301

Atterberg, A. (1911). “Relation of clays to water, their degrees of plasticity and plastic limits”, Documents and Journal No. 2 of the Royal Swedish Academy of Agriculture and Forestry, Stockholm, pp.133–139.

Casagrande, A. (1932). “Research on the Atterberg limits of soils”, Public Roads, Vol.13, No.8, pp.121–130.

ASTM D 421-58 (2006) “Standard Method for dry preparation of soil samples for particle-size analysis and determination of soil constants,” Designation: ASTM D 421-58, Annual Book of ASTM Standard.

ASTM D 422-63 (2006), “Standard Test Method for Particle Size Analysis of Soils”, Designation: D 422-63(Reapproved 1998), Annual Book of ASTM Standards, Vol. 04.08. pp. 10-17.

ASTMD 698-07 (2006), “Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort,” Designation: D 698 – 07, Annual Book of ASTM Standards. Vol. 04.08. pp. 1-4

ASTM D 4318-84 (2006), “Standard Method for liquid limit, plastic limit and plasticity index of soils,” Philadelphia, USA. Carter, M., S. Bentley. 1991. “Correlations of Soil Properties,” Pentech Press, London, England.

Casagrande, A. (1948), “Classification and Identification of Soils,” Transactions of the American Society of Civil Engineer, Vol. 113, pp. 901.

Casagrande, A. (1931), “The Hydrometer Method for Mechanical Analysis of Soils and Other Granular Materials,” Cambridge, mass, June.

Chapuis, R. P. (1990). “Sand-Bentonite Liners: Predicting Permeability from Laboratory Tests.” Canadian Geotechnical Journal 27, pp. 47-57.

Fukue, M., Okusa, S., and Nakamura, T. (1986). “Consolidation of Sand-Clay Mixtures.” Consolidation of Soils: Testing and Evaluation, ASTM STP 892. R.

Jafari, M. K., and Shafiee, A. (2004). “Mechanical Behavior of Compacted Composite Clays.” Canadian Geotechnical Journal, 41, pp. 1152-1167.

Shoop, S., P. Seman, D. Diemand, G. Mason, and L. Danyluk. (2008). “California bearing ratio database for soil strength prediction”, Transportation Systems Workshop, April 2008, Phoenix, AZ.

U.S. Army. 1997. *Military soils engineering*. Chapter 5, “Soil Classification.” Field Manual 5-410, Change 1. Washington, DC: U.S. Army.

ASTM D1883 (2011). “Standard test method for CBR (California bearing ratio) of laboratory-compacted soils”, In *Annual Book of ASTM Standards*, Vol. 04.08. West Conshohocken, PA: ASTM.

ASTM D4429 – 04 (2011). “Standard test method for CBR (California bearing ratio) of soils in place”, ASTM International, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA, 19428-2959 USA.

ASTM D2487–11 (2011). “Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)”, ASTM International, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA, 19428-2959 USA.

ASTM D422 – 63 (2007). “Standard Test Method for Particle-Size Analysis of Soils”, ASTM International, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA, 19428-2959 USA.

ASTM D2487–11 (2011). “Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)”, ASTM International, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA, 19428-2959 USA.

APPENDIX A

TEST DATA AND GRAPH

Table A.1 Data of Specific Gravity of Lean Clay

Wt. Of bottle + water+ soil (W_1), gm	372.1
Temperature of test (^oC)	27
Wt. Of bottle + water (W_2), gm	341.4
Wt. Of soil (W_s), gm	51
Specific gravity of water at T =27	0.9965451
Specific gravity of soil solids	2.63

Table A.2 Data of Liquid Limit Test for identifying Lean Clay

Sample no.	2	3	4	5
No. of blows	32	45	41	15
Container no.	778	879	174	407
Wt. of container	10.4	7.23	7.24	7.2
Wt. of container + wet soil	30.3	25.5	28.6	26.3
Wt. of container + dry soil	25.4	21.2	23.4	21.1
Wt. of water, W_w (gm).	4.9	4.3	5.2	5.2
Wt. of dry soil, W_s (gm).	15	13.97	16.16	13.9
Water content, w (%)	32.67	30.78	32.18	37.41

Table A.3 Data of Plastic Limit Test for identifying Lean Clay

Sample no.	1	2	3	4
Container no.	880	869	300	760
Wt. of container in gm.	10.74	11.2	7	11.3
Wt. container + wet soil (gm)	22.9	21.2	17.8	23.1
Wt. container + dry soil (gm)	21.3	19.9	16.4	21.6
Wt. of water in gm.	1.6	1.3	1.4	1.5
Wt. of dry soil in gm.	10.56	8.7	9.4	10.3
Water content, w in %	15.15	14.94	14.89	14.56

Table A.4 Data of Sieve Analysis of Sand

Sieve No	Sieve Opening (mm)	Wt of sieve (gm)	Wt of sieve + soil (gm)	Wt of soil retained (gm)	% of soil retained	Cumulative % retained	% Finer
# 4	4.76	525.3	525.3	0	0	0	100
# 8	2.38	491.7	491.7	0	0	0	100
# 20	0.84	420.2	420.5	0.3	0.36	0.36	99.64
# 40	0.42	400.3	422.2	21.9	26.64	27.01	72.99
#100	0.15	355	395.2	40.2	48.91	75.91	24.09
#200	0.074	342	352.4	10.4	12.65	88.56	11.44
Pan	0	364.4	373.8	9.4	11.44	100	0.00

Table A.5 Data of Specific Gravity of silt

Wt. Of bottle + water+ soil (W_1), gm	372.1
Temperature of test (^oC)	27
Wt. Of bottle + water (W_2), gm	341.4
Wt. Of soil (W_s), gm	51
Specific gravity of water at T =27	0.9965451
Specific gravity of soil solids	2.62

Table A.6 Compaction Test Result of Lean Clay

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moisture in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m^3	Dry density KN/m^3
01	403	6.7	17.2	16.1	9.4	1.1	11.7	12.5	4260	5850	1590	16.53	14.69
	9012	7.4	21.0	19.4	12.0	1.6	13.3						
02	405	6.8	17.5	16.1	9.3	1.4	15.1	15.5	4260	5970	1710	17.77	15.39
	736	7.3	19.7	18	10.7	1.7	15.9						
03	870	7.1	23.1	20.7	13.6	2.4	17.6	17.9	4260	6080	1820	18.92	16.04
	907	7.5	20.5	18.5	11.0	2.0	18.2						
04	77	7.7	28.1	24.9	17.2	3.2	18.6	18.7	4260	6100	1840	19.12	16.12
	818	7.2	25.6	22.7	15.5	2.9	18.7						
05	214	7.3	28.3	24.6	17.3	3.7	21.4	21.3	4260	6070	1810	18.81	15.51
	856	7.7	30.6	26.6	18.9	4.0	21.2						

Table A.7 Compaction Test Result of Lean Clay-Sand Mixture

(Clay=90% & Sand=10%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moistuere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m^3	Dry density KN/m^3
01	870	7.2	23.4	22.4	15.2	1.0	6.8	6.4	4260	5680	1420	14.74	13.85
	736	7.3	24.8	23.8	16.5	1.0	6.1						
02	818	7.3	27.6	25.6	18.3	2.0	10.9	10.4	4260	5710	1450	15.06	13.64
	839	10.9	28.8	27.2	16.3	1.6	9.8						
03	77	7.6	27.2	25.5	17.9	1.7	9.5	10.6	4260	5820	1560	16.20	14.64
	907	7.4	30.2	27.8	20.4	2.4	11.8						
04	403	6.7	27.7	25.2	18.5	2.5	13.5	13.6	4260	5920	1660	17.24	15.17
	405	6.7	24.1	22.0	15.3	2.1	13.7						
05	745	7.2	33.4	29.8	22.6	3.6	15.9	15.9	4260	6020	1760	18.27	15.76
	214	7.3	34.9	31.1	23.8	3.8	16.0						
06	9012	7.4	36.6	32.2	24.8	4.4	17.7	17.7	4260	6010	1750	18.17	15.44
	845	7.7	30.4	27.0	19.3	3.4	17.6						

Table A.8 Compaction Test Result of Lean Clay-Sand Mixture

(Clay=80% & Sand=20%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moistuere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m^3	Dry density KN/m^3
01	309	11.1	23.4	22.4	11.3	1.0	8.8	8.0	4260	5700	1440	14.95	13.84
	883	11.2	29.0	27.8	16.6	1.2	7.2						
02	718	7.2	23.2	21.8	14.6	1.4	9.6	9.5	4260	5850	1590	16.51	15.08
	832	11.2	29.8	28.2	17.0	1.6	9.4						
03	607	7.0	24.0	22.3	15.3	1.7	11.1	12.0	4260	6000	1740	18.07	16.13
	212	7.3	30.0	27.4	20.1	2.6	12.9						
04	12	11.4	27.3	25.4	14.0	1.9	13.6	14.1	4260	6050	1790	18.59	16.29
	765	10.6	32.6	29.8	19.2	2.8	14.6						
05	152	7.0	30.0	26.8	19.8	3.2	16.2	16.2	4260	6100	1840	19.10	16.45
	145	7.1	29.4	26.3	19.2	3.1	16.1						
06	175	7.3	25.9	23.2	15.9	2.7	17.0	17.3	4260	6040	1780	18.48	15.76
	107	7.3	30.0	26.6	19.3	3.4	17.6						

Table A.9 Compaction Test Result of Lean Clay-Sand Mixture

(Clay=70% & Sand=30%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moist uere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m^3	Dry density KN/m^3
01	107	7.7	28.7	26.9	19.2	1.8	9.4	9.5	4260	5800	1540	15.99	14.61
	175	8.0	29.8	27.9	19.9	1.9	9.5						
02	212	9.0	25.6	23.8	14.8	1.8	12.2	12.2	4260	5850	1590	16.51	14.72
	883	11.2	28.7	26.8	15.6	1.9	12.2						
03	765	10.6	33.0	30.4	19.8	2.6	13.1	13.4	4260	5950	1690	17.55	15.48
	309	11.1	30.3	28.0	16.9	2.3	13.6						
04	145	7.0	28.8	26.2	19.2	2.6	13.5	13.9	4260	6100	1840	19.10	16.77
	12	11.4	34.6	31.7	20.3	2.9	14.3						
05	832	11.3	34.1	30.9	19.6	3.2	16.3	16.5	4260	5950	1690	17.55	15.07
	607	7.0	29.5	26.3	19.3	3.2	16.6						

Table A.10 Compaction Test Result of Lean Clay-Sand Mixture

(Clay=60% & Sand=40%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moistuere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m³	Dry density KN/m³
01	838	7.1	31.3	29.2	22.1	2.1	9.5	9.0	4260	5900	1640	17.03	15.63
	768	10.7	33.9	32.1	21.4	1.8	8.4						
02	146	7.3	31.1	28.8	21.5	2.3	10.7	11.0	4260	5960	1700	17.65	15.91
	800	6.4	33.1	30.4	24.0	2.7	11.3						
03	848	7.2	34.8	31.7	24.5	3.1	12.7	12.7	4260	6090	1830	19.00	16.86
	714	6.9	32.5	29.6	22.7	2.9	12.8						
04	162	6.9	30.6	27.6	20.7	3.0	14.5	14.6	4260	6070	1810	18.79	16.40
	739	7.1	27.4	24.8	17.7	2.6	14.7						

Table A.11 Compaction Test Result of Lean Clay-Sand Mixture

(Clay=50% & Sand=50%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moist uere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m^3	Dry density KN/m^3
01	214	7.4	30.3	28.9	21.5	1.4	6.5	5.8	4260	5900	1640	17.03	16.09
	856	11.4	34.1	33.0	21.6	1.1	5.1						
02	2	7.5	29.5	27.9	20.4	1.6	7.8	7.6	4260	5980	1720	17.86	16.60
	712	7.3	29.4	27.9	20.6	1.5	7.3						
03	901	7.8	31.0	28.9	21.1	2.1	10.0	9.8	4260	6070	1810	18.79	17.11
	803	11.1	34.8	32.7	21.6	2.1	9.7						
04	909	7.4	31.5	29.0	21.6	2.5	11.6	11.6	4260	6050	1790	18.59	16.66
	764	11.0	30.3	28.3	17.3	2.0	11.6						

Table A.12 Compaction Test Result of Lean Clay-Sand Mixture

(Clay=40% & Sand=60%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moist uere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m^3	Dry density KN/m^3
01	214	7.2	34.7	32.7	25.5	2.0	7.8	8.2	4260	5910	1650	17.13	15.84
	856	11.3	38.2	36.1	24.8	2.1	8.5						
02	2	7.4	34.9	32.0	24.6	2.9	11.8	9.5	4260	6000	1740	18.07	16.50
	712	7.2	34.2	32.4	25.2	1.8	7.1						
03	901	7.7	33.7	30.8	23.1	2.9	12.6	12.3	4260	5950	1690	17.55	15.63
	803	11.2	39.3	36.3	25.1	3.0	12.0						

Table A.13 Compaction Test Result of Lean Clay-Sand Mixture

(Clay=30% & Sand=70%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moist uere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m³	Dry density KN/m³
01	837	7.1	33.8	31.4	24.3	2.4	9.9	9.8	4260	6000	1740	18.07	16.46
	509	6.9	37.5	34.8	27.9	2.7	9.7						
02	3333	7.0	36.2	33.3	26.3	2.9	11.0	10.9	4260	6100	1840	19.10	17.23
	154	7.0	31.7	29.3	22.3	2.4	10.8						
03	301	7.3	28.1	25.8	18.5	2.3	12.4	12.8	4260	6080	1820	18.90	16.75
	784	7.6	28.2	25.8	18.2	2.4	13.2						

Table A.14 Compaction Test Result of Lean Clay-Sand Mixture

(Clay=20% & Sand=80%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moist uere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m³	Dry density KN/m³
01	784	7.6	36.8	34.9	27.3	1.9	7.0	7.1	4260	6010	1750	18.17	16.96
	509	6.9	30.4	28.8	21.9	1.6	7.3						
02	837	7.1	35.5	33.3	26.2	2.2	8.4	8.7	4260	6080	1820	18.90	17.39
	3333	7.0	33.8	31.6	24.6	2.2	8.9						
03	154	7.0	35.4	32.6	25.6	2.8	10.9	11.0	4260	6050	1790	18.59	16.75
	301	7.2	32.5	30.0	22.8	2.5	11.0						

Table A.15 Compaction Test Result of Lean Clay-Sand Mixture

(Clay=10% & Sand=90%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moistuere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m³	Dry density KN/m³
01	714	6.9	31.7	30.2	23.3	1.5	6.4	6.5	4260	6020	1760	18.27	17.15
	838	7.0	32.8	31.2	24.2	1.6	6.6						
02	768	10.7	35.0	33.1	22.4	1.9	8.5	8.7	4260	6090	1830	19.00	17.49
	712	7.3	30.7	28.8	21.5	1.9	8.8						
03	733	6.8	35.5	32.9	26.1	2.6	10.0	10.1	4260	6050	1790	18.59	16.88
	856	11.4	38.2	35.7	24.3	2.5	10.3						

Table A.16 Compaction Test Result of Lean Clay-Sand Mixture

(Sand=100%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moistuere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m³	Dry density KN/m³
01	856	6.4	31.2	30.2	23.8	1.0	4.2	4.2	4260	6010	1750	18.17	17.44
	733	6.9	32.2	31.2	24.3	1.0	4.1						
02	712	10.0	34.9	33.1	23.1	1.8	7.8	8.3	4260	6050	1790	18.59	17.16
	768	7.2	30.7	28.8	21.6	1.9	8.8						
03	838	6.2	35.3	32.9	26.7	2.4	9.0	9.2	4260	6030	1770	18.38	16.84
	714	11.0	38.0	35.7	24.7	2.3	9.3						

Table A.17 Compaction Test Result of Lean Clay-Silt mixture (Lean Clay=90% and Silt=10%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moistuere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m³	Dry density KN/m³
01	607	6.9	34.6	33.1	26.2	1.5	5.7	5.4	4260	5600	1340	13.91	13.20
	175	7.3	30.4	29.3	22.0	1.1	5.0						
02	145	7.0	23.7	22.5	15.5	1.2	7.7	7.7	4260	5690	1430	14.85	13.78
	832	11.3	29.5	28.2	16.9	1.3	7.7						
03	107	7.2	22.3	21.1	13.9	1.2	8.6	9.3	4260	5770	1510	15.68	14.34
	212	7.2	21.5	20.2	13.0	1.3	10.0						
04	309	11.1	28.1	26.5	15.4	1.6	10.4	11.4	4260	5850	1590	16.51	14.82
	883	11.2	27.6	25.8	14.6	1.8	12.3						
05	760	10.5	28.2	26.3	15.8	1.9	12.0	12.4	4260	5910	1650	17.13	15.24
	12	11.4	27.3	25.5	14.1	1.8	12.8						
06	711	7.3	26.0	23.6	16.3	2.4	14.7	14.5	4260	5960	1700	17.65	15.41
	8	7.1	25.5	23.2	16.1	2.3	14.3						
07	14	7.0	24.2	21.8	14.8	2.4	16.2	16.4	4260	5920	1660	17.24	14.81
	132	7.2	25.5	22.9	15.7	2.6	16.6						

Table A.18 Compaction Test Result of Lean Clay-Silt mixture (Lean Clay = 80% and Silt = 20%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moistuere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m³	Dry density KN/m³
01	175	7.4	21.3	20.2	12.8	1.1	8.6	8.6	4260	5650	1390	14.43	13.29
	145	7.0	24.8	23.4	16.4	1.4	8.5						
02	107	7.3	22.1	20.9	13.6	1.2	8.8	10.5	4260	5700	1440	14.95	13.53
	832	11.3	26.9	25.2	13.9	1.7	12.2						
03	765	10.6	27.8	26.0	15.4	1.8	11.7	11.6	4260	5760	1500	15.57	13.96
	883	11.2	23.8	22.5	11.3	1.3	11.5						
04	309	11.1	25.3	23.7	12.6	1.6	12.7	13.4	4260	5800	1540	15.99	14.11
	12	11.5	23.7	22.2	10.7	1.5	14.0						
05	8	7.3	21.2	19.5	12.2	1.7	13.9	15.1	4260	5900	1640	17.03	14.80
	132	7.2	20.8	18.9	11.7	1.9	16.2						
06	711	7.3	23.0	20.6	13.3	2.4	18.0	17.7	4260	5850	1590	16.51	14.03
	14	7.0	22.6	20.3	13.3	2.3	17.3						

Table A.19 Compaction Test Result of Lean Clay-Silt mixture (Lean Clay = 70% and Silt = 30%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moistuere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m³	Dry density KN/m³
01	14	7.0	20.8	19.6	12.6	1.2	9.5	9.5	4260	5700	1440	14.95	13.65
	12	11.5	26.5	25.2	13.7	1.3	9.5						
02	711	7.3	21.8	20.5	13.2	1.3	9.8	11.1	4260	5790	1530	15.89	14.30
	145	6.9	18.8	17.5	10.6	1.3	12.3						
03	175	7.2	19.1	17.6	10.4	1.5	14.4	14.5	4260	5870	1610	16.72	14.60
	107	7.2	20.6	18.9	11.7	1.7	14.5						
04	8	7.1	22.8	20.8	13.7	2.0	14.6	14.9	4260	5900	1640	17.03	14.83
	883	11.2	27.2	25.1	13.9	2.1	15.1						
05	309	11.1	28.9	26.3	15.2	2.6	17.1	18.8	4260	5850	1590	16.51	13.90
	765	10.6	30.6	27.2	16.6	3.4	20.5						

Table A.20 Compaction Test Result of Lean Clay-Silt mixture (Lean Clay = 60% and Silt = 40%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moistuere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m³	Dry density KN/m³
01	12	11.3	25.6	24.2	12.9	1.4	10.9	10.1	4140	5750	1610	16.72	15.18
	309	11.1	24.0	22.9	11.8	1.1	9.3						
02	711	7.4	21.6	20.2	12.8	1.4	10.9	11.6	4140	5800	1660	17.24	15.44
	145	7.1	20.8	19.3	12.2	1.5	12.3						
03	8	7.4	22.0	20.3	12.9	1.7	13.2	12.9	4140	5830	1690	17.55	15.54
	765	10.6	22.2	20.9	10.3	1.3	12.6						
04	175	7.3	23.0	20.9	13.6	2.1	15.4	15.4	4140	5860	1720	17.86	15.47
	883	11.2	27.7	25.5	14.3	2.2	15.4						
05	107	7.3	26.1	23.1	15.8	3.0	19.0	17.1	4140	5850	1710	17.75	15.16
	14	6.9	25.1	22.7	15.8	2.4	15.2						

Table A.21 Compaction Test Result of Lean Clay-Silt mixture (Lean Clay = 50% and Silt = 50%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moistuere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m³	Dry density KN/m³
01	800	6.4	26.3	24.9	18.5	1.4	7.6	7.4	4260	5770	1510	15.68	14.60
	162	6.9	26.4	25.1	18.2	1.3	7.1						
02	768	10.6	34.6	32.7	22.1	1.9	8.6	8.5	4260	5820	1560	16.20	14.93
	739	7.1	31.6	29.7	22.6	1.9	8.4						
03	848	7.3	31.8	29.6	22.3	2.2	9.9	10.2	4260	5900	1640	17.03	15.45
	838	7.0	25.8	24.0	17.0	1.8	10.6						
04	714	7.0	33.4	30.5	23.5	2.9	12.3	12.1	4260	5920	1660	17.24	15.37
	146	7.3	28.9	26.6	19.3	2.3	11.9						
05	733	6.9	28.3	25.8	18.9	2.5	13.2	13.8	4260	5900	1640	17.03	14.96
	762	7.8	28.5	25.9	18.1	2.6	14.4						

Table A.22 Compaction Test Result of Lean Clay-Silt mixture (Lean Clay = 40% and Silt = 60%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moistuere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m³	Dry density KN/m³
01	848	7.2	28.9	26.8	19.6	2.1	10.7	10.7	4260	5800	1540	15.99	14.44
	739	7.1	28.7	26.6	19.5	2.1	10.8						
02	800	6.4	34.1	31.1	24.7	3.0	12.1	11.8	4260	5840	1580	16.40	14.67
	162	7.0	35.1	32.2	25.2	2.9	11.5						
03	838	7.1	39.5	35.7	28.6	3.8	13.3	14.0	4260	5900	1640	17.03	14.94
	768	10.6	40.3	36.5	25.9	3.8	14.7						
04	733	6.9	33.2	29.9	23.0	3.3	14.3	15.0	4260	5850	1590	16.51	14.36
	714	6.9	33.6	30.0	23.1	3.6	15.6						

Table A.23 Compaction Test Result of Lean Clay-Silt mixture (Lean Clay = 30% and Silt = 70%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moistuere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m³	Dry density KN/m³
01	733	6.9	31.4	29.8	22.9	1.6	7.0	7.1	4260	5830	1570	16.30	15.22
	739	7.0	29.2	27.7	20.7	1.5	7.2						
02	714	7.0	28.8	27.1	20.1	1.7	8.5	8.5	4260	5850	1590	16.51	15.21
	162	6.9	30.8	28.9	22.0	1.9	8.6						
03	800	6.5	33.0	30.8	24.3	2.2	9.1	9.6	4260	5890	1630	16.92	15.44
	848	7.3	34.5	32.0	24.7	2.5	10.1						
04	838	7.0	31.2	29.2	22.2	2.0	9.0	12.3	4260	5950	1690	17.55	15.62
	768	10.6	40.2	36.2	25.6	4.0	15.6						
05	712	7.3	34.9	31.2	23.9	3.7	15.5	14.7	4260	5890	1630	16.92	14.76
	856	11.4	32.8	30.2	18.8	2.6	13.8						

Table A.24 Compaction Test Result of Lean Clay-Silt mixture (Lean Clay = 20% and Silt = 80%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moistuere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m³	Dry density KN/m³
01	733	6.8	30.5	28.2	21.4	2.3	10.7	10.3	4260	5880	1620	16.82	15.25
	714	6.9	26.9	25.1	18.2	1.8	9.9						
02	856	11.5	35.6	33.1	21.6	2.5	11.6	11.6	4260	5900	1640	17.03	15.25
	768	10.6	37.3	34.5	23.9	2.8	11.7						
03	712	7.3	37.5	34	26.7	3.5	13.1	13.6	4260	5950	1690	17.55	15.44
	739	7.0	35.2	31.7	24.7	3.5	14.2						
04	838	7.1	36.3	32.4	25.3	3.9	15.4	15.4	4260	5930	1670	17.34	15.02
	162	6.9	35.3	31.5	24.6	3.8	15.4						

Table A.25 Compaction Test Result of Lean Clay-Silt mixture (Lean Clay = 10% and Silt = 90%)

Sl No.	Can No.		Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moistuere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m³	Dry density KN/m³
01	714	6.9	31.7	30.2	23.3	1.5	6.4	6.5	4260	5900	1640	17.03	15.98
	838	7.0	32.8	31.2	24.2	1.6	6.6						
02	768	10.7	35.0	33.1	22.4	1.9	8.5	8.7	4260	5980	1720	17.86	16.44
	712	7.3	30.7	28.8	21.5	1.9	8.8						
03	733	6.8	35.5	32.9	26.1	2.6	10.0	10.1	4260	6000	1740	18.07	16.41
	856	11.4	38.2	35.7	24.3	2.5	10.3						
04	739	7.1	32.5	29.9	22.8	2.6	11.4	11.7	4260	5980	1720	17.86	15.99
	162	7.0	35.1	32.1	25.1	3.0	12.0						

Table A.26 Compaction Test Result of Lean Clay-Silt mixture (Lean Clay = 0% and Silt = 100%)

Sl No.	Can No.	Wt. of Can in gm	Wt. of can + wet soil in gm	Wt. of can + dry soil in gm	Wt. dry soil in gm	Wt. Moistuere in gm	M.C in %	Average M.C in %	Wt. Mold in gm	Wt. Mold + compacted soil in gm	Wt. ompacted soil in gm	Wt. density KN/m³	Dry density KN/m³
01	733	6.2	31.7	30.2	24.0	1.5	6.3	6.4	4260	5930	1670	17.34	16.29
	714	6.9	32.8	31.2	24.3	1.6	6.6						
02	856	10.2	35.0	33.1	22.9	1.9	8.3	8.5	4260	5990	1730	17.96	16.55
	768	7.0	30.7	28.8	21.8	1.9	8.7						
03	712	6.2	35.5	32.9	26.7	2.6	9.7	10.0	4260	6020	1760	18.27	16.62
	739	11.2	38.2	35.7	24.5	2.5	10.2						
04	838	7.0	32.5	29.9	22.9	2.6	11.4	11.6	4260	6010	1750	18.17	16.28
	162	6.8	35.1	32.1	25.3	3.0	11.9						

Table A.27 CBR Test Data (Lean Clay = 0% and Sand = 100%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in^2)	Penetration Stress (psi)
0	0	0	3	0
0.025	35.0	206.27	3	68.76
0.050	50.0	293.81	3	97.94
0.075	65.0	381.35	3	127.12
0.100	80.0	468.89	3	156.30
0.125	85.0	498.07	3	166.02
0.150	95.0	556.43	3	185.48
0.175	100.0	585.61	3	195.20
0.200	110.0	643.97	3	214.66
0.300	120.0	702.33	3	234.11
0.400	125.0	731.51	3	243.84
0.500	130.0	760.69	3	253.56

Table A.28 Soaked CBR Test Data (Lean Clay = 10% and Sand = 90%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in^2)	Penetration Stress (psi)
0	0	0	3	0
0.025	25.0	147.91	3	49.30
0.050	35.0	206.27	3	68.76
0.075	50.0	293.81	3	97.94
0.100	60.0	352.17	3	117.39
0.125	65.0	381.35	3	127.12
0.150	75.0	439.71	3	146.57
0.175	80.0	468.89	3	156.30
0.200	85.0	498.07	3	166.02
0.300	100.0	585.61	3	195.20
0.400	115.0	673.15	3	224.38
0.500	125.0	731.51	3	243.84

Table A.29 Soaked CBR Test Data (Lean Clay = 30% and Sand = 70%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in^2)	Penetration Stress (psi)
0	0	0	3	0
0.025	15.0	89.55	3	29.85
0.050	25.0	147.91	3	49.30
0.075	35.0	206.27	3	68.76
0.100	45.0	264.63	3	88.21
0.125	50.0	293.81	3	97.94
0.150	55.0	322.99	3	107.66
0.175	60.0	352.17	3	117.39
0.200	65.0	381.35	3	127.12
0.300	85.0	498.07	3	166.02
0.400	100.0	585.61	3	195.20
0.500	115.0	673.15	3	224.38

Table A.30 Soaked CBR Test Data (Lean Clay = 50% and Sand = 50%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in^2)	Penetration Stress (psi)
0	0	0	3	0
0.025	10.0	60.37	3	20.12
0.050	15.0	89.55	3	29.85
0.075	20.0	118.73	3	39.58
0.100	40.0	235.45	3	78.48
0.125	45.0	264.63	3	88.21
0.150	50.0	293.81	3	97.94
0.175	55.0	322.99	3	107.66
0.200	60.0	352.17	3	117.39
0.300	85.0	498.07	3	166.02
0.400	100.0	585.61	3	195.20
0.500	110.0	643.97	3	214.66

Table A.31 Soaked CBR Test Data (Lean Clay = 70% and Sand = 30%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in²)	Penetration Stress (psi)
0	0	0	3	0
0.025	8.0	48.70	3	16.23
0.050	12.0	72.04	3	24.01
0.075	15.0	89.55	3	29.85
0.100	16.0	95.39	3	31.80
0.125	17.0	101.22	3	33.74
0.150	18.0	107.06	3	35.69
0.175	20.0	118.73	3	39.58
0.200	22.0	130.40	3	43.47
0.300	28.0	165.42	3	55.14
0.400	30.0	177.09	3	59.03
0.500	35.0	206.27	3	68.76

Table A.32 Soaked CBR Test Data (Lean Clay = 90% and Sand = 10%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in²)	Penetration Stress (psi)
0	0	0	3	0
0.025	5.0	31.19	3	10.40
0.050	7.0	42.86	3	14.29
0.075	9.0	54.54	3	18.18
0.100	12.0	72.04	3	24.01
0.125	13.0	77.88	3	25.96
0.150	14.0	83.72	3	27.91
0.175	15.0	89.55	3	29.85
0.200	16.0	95.39	3	31.80
0.300	18.0	107.06	3	35.69
0.400	20.0	118.73	3	39.58
0.500	25.0	147.91	3	49.30

Table A.33 Soaked CBR Test Data (Lean Clay = 100% and Sand = 0%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in^2)	Penetration Stress (psi)
0	0	0	3	0
0.025	6.0	10.00	3	3.33
0.050	7.0	19.00	3	6.33
0.075	8.0	30.00	3	10.00
0.100	9.0	33.00	3	11.00
0.125	10.0	40.00	3	13.33
0.150	12.0	42.00	3	14.00
0.175	13.0	45.00	3	15.00
0.200	14.0	50.00	3	16.67
0.300	30.0	60.00	3	20.00
0.400	45.0	80.00	3	26.67
0.500	65.0	100.00	3	33.33

Table A.34 Soaked CBR Test Data Compacted by Standard Effort (Lean Clay = 90% and Sand = 10%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in^2)	Penetration Stress (psi)
0	0	0	3	0
0.025	10.0	60.37	3	20.12
0.050	12.0	72.04	3	24.01
0.075	15.0	89.55	3	29.85
0.100	25.0	147.91	3	49.30
0.125	26.0	153.75	3	51.25
0.150	28.0	165.42	3	55.14
0.175	30.0	177.09	3	59.03
0.200	32.0	188.76	3	62.92
0.300	50.0	293.81	3	97.94
0.400	70.0	410.53	3	136.84
0.500	90.0	527.25	3	175.75

Table A.35 Soaked CBR Test Data Compacted by Standard Effort (Lean Clay = 80% and Sand = 20%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in²)	Penetration Stress (psi)
0	0	0	3	0
0.025	25.0	147.91	3	49.30
0.050	30.0	177.09	3	59.03
0.075	35.0	206.27	3	68.76
0.100	45.0	264.63	3	88.21
0.125	46.0	270.47	3	90.16
0.150	48.0	282.14	3	94.05
0.175	50.0	293.81	3	97.94
0.200	55.0	322.99	3	107.66
0.300	70.0	410.53	3	136.84
0.400	100.0	585.61	3	195.20
0.500	115.0	673.15	3	224.38

Table A.36 Soaked CBR Test Data Compacted by Standard Effort (Lean Clay = 70% and Sand = 30%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in²)	Penetration Stress (psi)
0	0	0	3	0
0.025	35.0	206.27	3	68.76
0.050	37.0	217.94	3	72.65
0.075	45.0	264.63	3	88.21
0.100	55.0	322.99	3	107.66
0.125	57.0	334.66	3	111.55
0.150	60.0	352.17	3	117.39
0.175	65.0	381.35	3	127.12
0.200	70.0	410.53	3	136.84
0.300	90.0	527.25	3	175.75
0.400	120.0	702.33	3	234.11
0.500	140.0	819.05	3	273.02

Table A.37 Soaked CBR Test Data (Lean Clay = 0% and Silt = 100%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in^2)	Penetration Stress (psi)
0	0	0	3	0
0.025	10.0	60.37	3	20.12
0.050	12.0	72.04	3	24.01
0.075	14.0	83.72	3	27.91
0.100	18.0	107.06	3	35.69
0.125	20.0	118.73	3	39.58
0.150	22.0	130.40	3	43.47
0.175	25.0	147.91	3	49.30
0.200	27.0	159.58	3	53.19
0.300	45.0	264.63	3	88.21
0.400	55.0	322.99	3	107.66
0.500	65.0	381.35	3	127.12

Table A.38 Soaked CBR Test Data (Lean Clay = 10% and Silt = 90%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in^2)	Penetration Stress (psi)
0	0	0	3.14	0
0.025	8.0	48.70	3.14	15.51
0.050	10.0	60.37	3.14	19.23
0.075	12.0	72.04	3.14	22.94
0.100	15.0	89.55	3.14	28.52
0.125	16.0	95.39	3.14	30.38
0.150	18.0	107.06	3.14	34.10
0.175	20.0	118.73	3.14	37.81
0.200	22.0	130.40	3.14	41.53
0.300	35.0	206.27	3.14	65.69
0.400	40.0	235.45	3.14	74.98
0.500	45.0	264.63	3.14	84.28

Table A.39 Soaked CBR Test Data (Lean Clay = 30% and Silt = 70%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in²)	Penetration Stress (psi)
0	0	0	3.14	0
0.025	4.0	25.36	3.14	8.08
0.050	6.0	37.03	3.14	11.79
0.075	7.0	42.86	3.14	13.65
0.100	10.0	60.37	3.14	19.23
0.125	11.0	66.21	3.14	21.09
0.150	12.0	72.04	3.14	22.94
0.175	13.0	77.88	3.14	24.80
0.200	15.0	89.55	3.14	28.52
0.300	20.0	118.73	3.14	37.81
0.400	30.0	177.09	3.14	56.40
0.500	35.0	206.27	3.14	65.69

Table A.40 Soaked CBR Test Data (Lean Clay = 50% and Silt = 50%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in²)	Penetration Stress (psi)
0	0	0	3.14	0
0.025	3.0	19.52	3.14	6.22
0.050	4.0	25.36	3.14	8.08
0.075	7.0	42.86	3.14	13.65
0.100	9.0	54.54	3.14	17.37
0.125	10.0	60.37	3.14	19.23
0.150	11.0	66.21	3.14	21.09
0.175	12.0	72.04	3.14	22.94
0.200	13.0	77.88	3.14	24.80
0.300	15.0	89.55	3.14	28.52
0.400	17.0	101.22	3.14	32.24
0.500	20.0	118.73	3.14	37.81

Table A.41 Soaked CBR Test Data (Lean Clay = 70% and Silt = 30%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in^2)	Penetration Stress (psi)
0	0	0	3	0
0.025	3.0	19.52	3	6.51
0.050	4.0	25.36	3	8.45
0.075	5.0	31.19	3	10.40
0.100	8.0	48.70	3	16.23
0.125	9.0	54.54	3	18.18
0.150	10.0	60.37	3	20.12
0.175	11.0	66.21	3	22.07
0.200	12.0	72.04	3	24.01
0.300	15.0	89.55	3	29.85
0.400	18.0	107.06	3	35.69
0.500	25.0	147.91	3	49.30

Table A.42 Soaked CBR Test Data (Lean Clay = 90% and Silt = 10%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in^2)	Penetration Stress (psi)
0	0	0	3	0
0.025	2.0	13.68	3	4.56
0.050	3.0	19.52	3	6.51
0.075	5.0	31.19	3	10.40
0.100	6.0	37.03	3	12.34
0.125	7.0	42.86	3	14.29
0.150	8.0	48.70	3	16.23
0.175	9.0	54.54	3	18.18
0.200	10.0	60.37	3	20.12
0.300	12.0	72.04	3	24.01
0.400	15.0	89.55	3	29.85
0.500	20.0	118.73	3	39.58

Table A.43 Soaked CBR Test Data Compacted by Standard Effort (Lean Clay = 100% and Silt = 0%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in^2)	Penetration Stress (psi)
0	0	0	3	0
0.025	6.0	37.03	3	12.34
0.050	7.0	42.86	3	14.29
0.075	8.0	48.70	3	16.23
0.100	9.0	54.54	3	18.18
0.125	10.0	60.37	3	20.12
0.150	12.0	65.00	3	21.67
0.175	13.0	70.00	3	23.33
0.200	14.0	75.00	3	25.00
0.300	30.0	150.00	3	50.00
0.400	45.0	220.00	3	73.33
0.500	65.0	300.00	3	100.00

Table A.44 Soaked CBR Test Data Compacted by Standard Effort (Lean Clay = 0% and Silt = 100%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in^2)	Penetration Stress (psi)
0	0	0	3	0
0.025	10.0	60.37	3	20.12
0.050	15.0	89.55	3	29.85
0.075	20.0	118.73	3	39.58
0.100	25.0	147.91	3	49.30
0.125	27.0	159.58	3	53.19
0.150	30.0	177.09	3	59.03
0.175	32.0	188.76	3	62.92
0.200	35.0	206.27	3	68.76
0.300	55.0	322.99	3	107.66
0.400	70.0	410.53	3	136.84
0.500	90.0	527.25	3	175.75

Table A.45 Soaked CBR Test Data Compacted by Standard Effort (Lean Clay = 90% and Silt = 10%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in^2)	Penetration Stress (psi)
0	0	0	3	0
0.025	5.0	31.19	3	10.40
0.050	7.0	42.86	3	14.29
0.075	10.0	60.37	3	20.12
0.100	13.0	77.88	3	25.96
0.125	15.0	89.55	3	29.85
0.150	18.0	107.06	3	35.69
0.175	20.0	118.73	3	39.58
0.200	23.0	136.24	3	45.41
0.300	45.0	264.63	3	88.21
0.400	65.0	381.35	3	127.12
0.500	80.0	468.89	3	156.30

Table A.46 Soaked CBR Test Data Compacted by Standard Effort (Lean Clay = 80% and Silt = 20%)

Penetration (inch)	Proving Ring Dial Reading	Piston Load (lb)	Area of Piston (in^2)	Penetration Stress (psi)
0	0	0	3	0
0.025	5.0	31.19	3	10.40
0.050	7.0	42.86	3	14.29
0.075	10.0	60.37	3	20.12
0.100	16.0	95.39	3	31.80
0.125	17.0	101.22	3	33.74
0.150	19.0	112.90	3	37.63
0.175	20.0	118.73	3	39.58
0.200	22.0	130.40	3	43.47
0.300	36.0	212.11	3	70.70
0.400	40.0	235.45	3	78.48
0.500	70.0	410.53	3	136.84

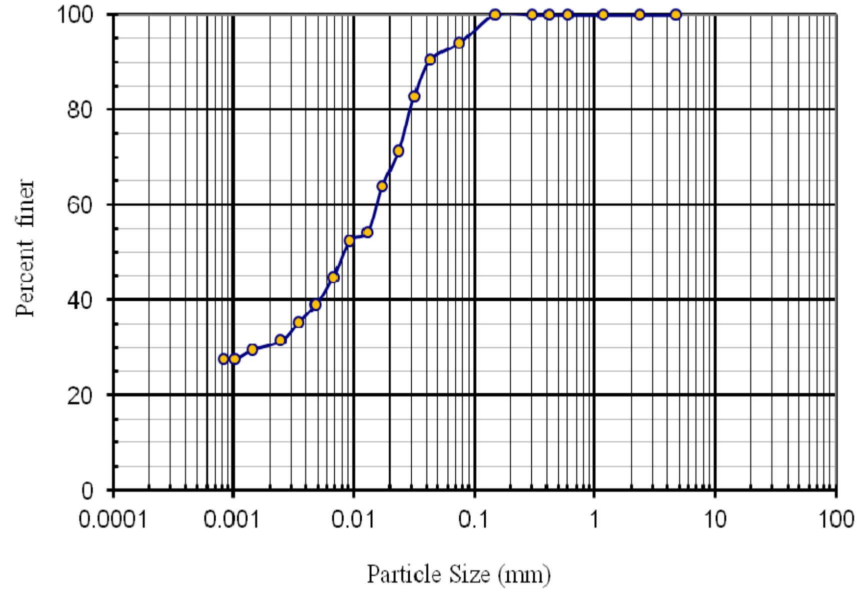

Fig. A.1 Typical Grain size Distribution curve of Lean Clay

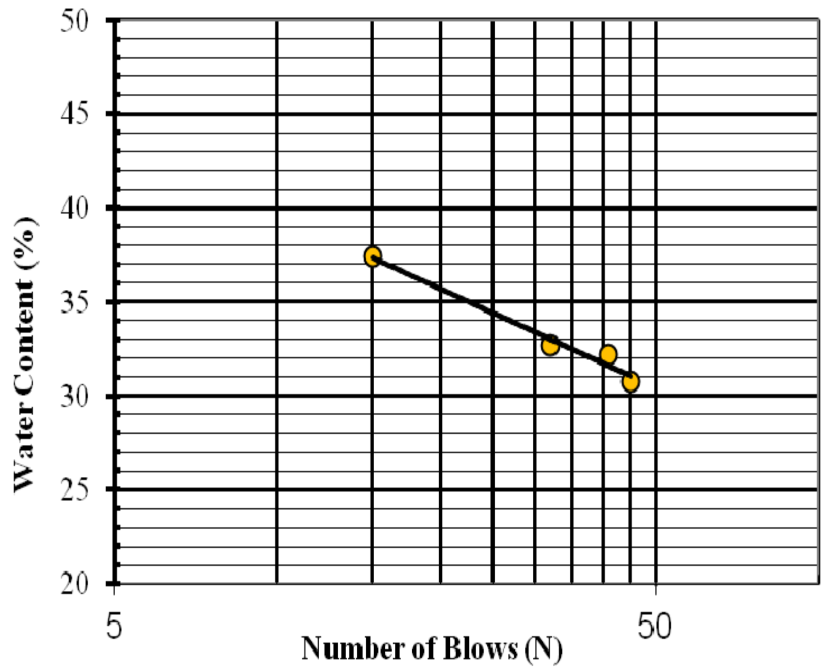

Fig. A.2 Flow curve for identifying Lean Clay

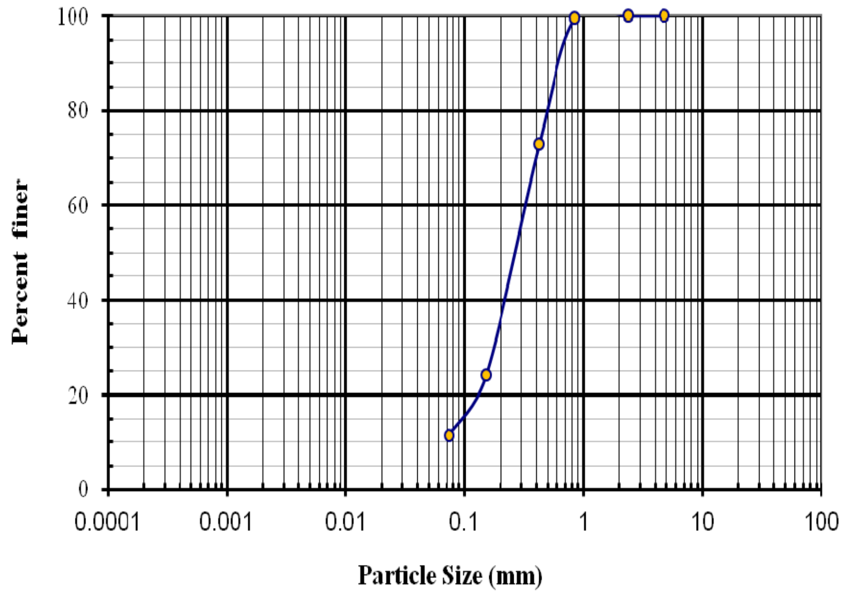

Fig. A.3 Typical Grain size Distribution Curve of sand sample

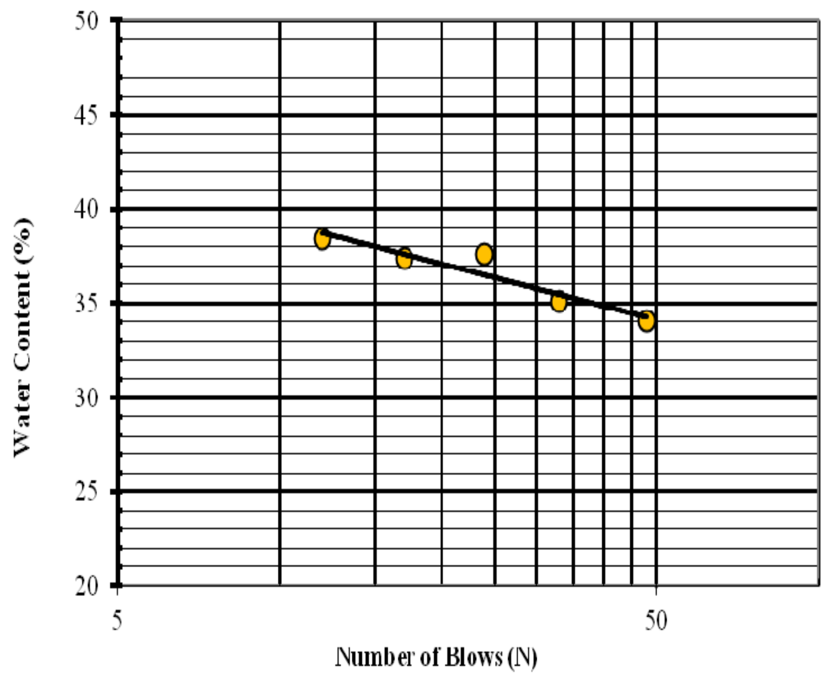

Fig. A.4 Flow curve of Lean Clay-Sand mixture (Clay=90% & Sand=10%)

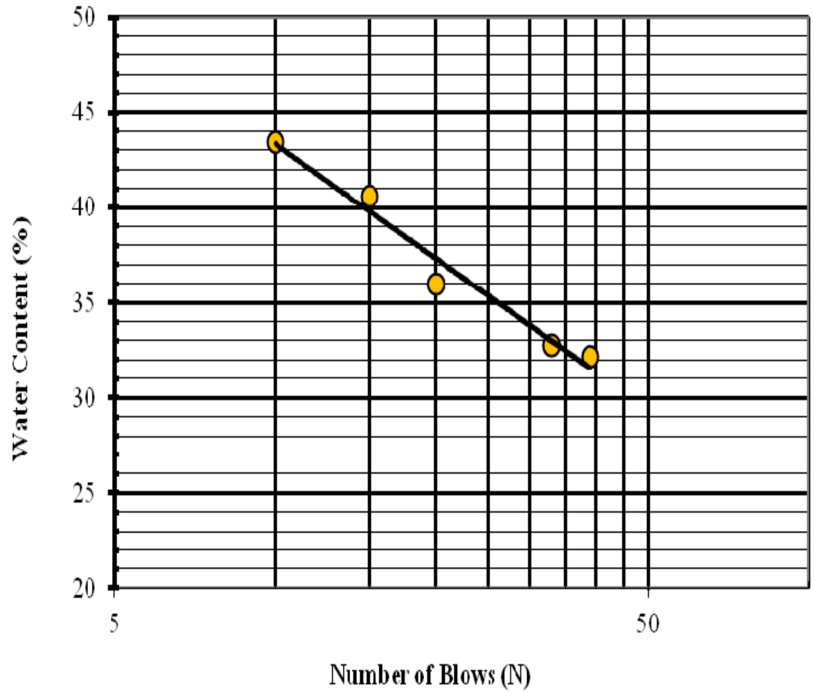

Fig. A.5 Flow curve of Lean Clay-Sand mixture (Clay=80% & Sand=20%)

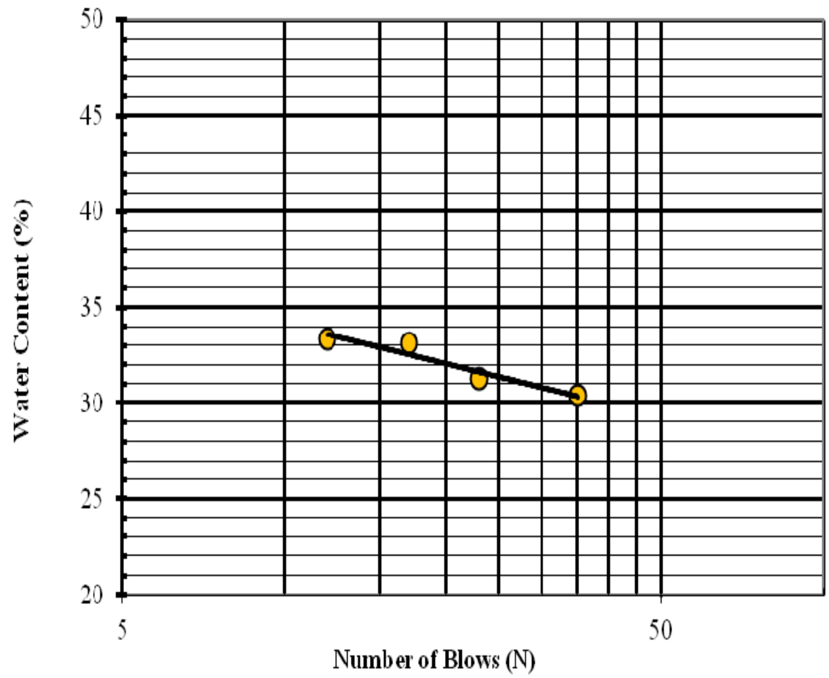

Fig. A.6 Flow curve of Lean Clay-Sand mixture (Clay=70% & Sand=30%)

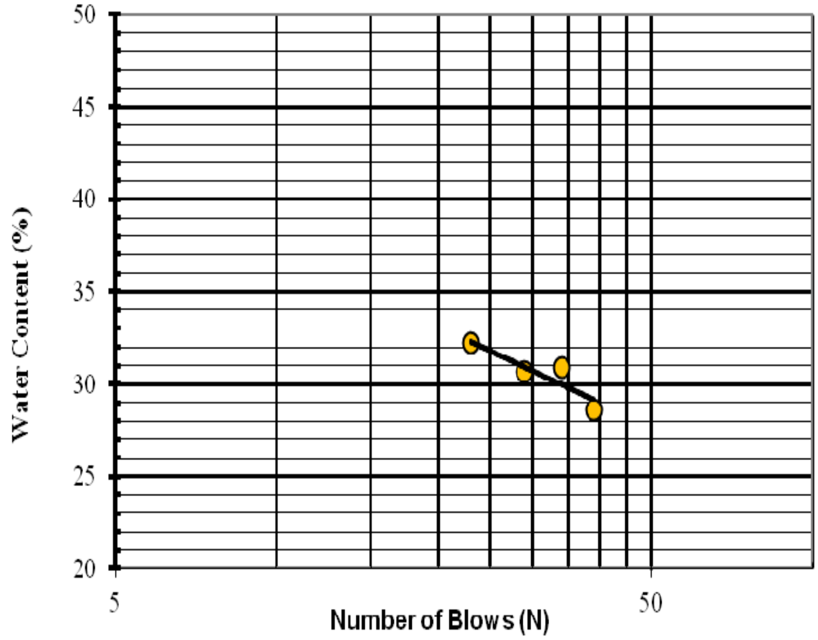

Fig. A.7 Flow curve of Lean Clay-Sand mixture (Clay=60% & Sand=40%)

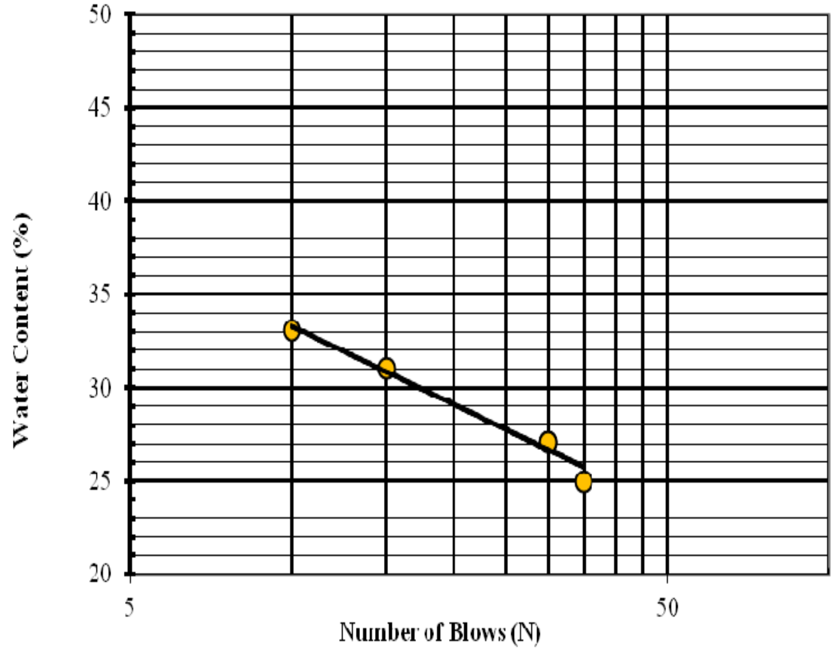

Fig. A.8 Flow curve of Lean Clay-Sand mixture (Clay=50% & Sand=50%)

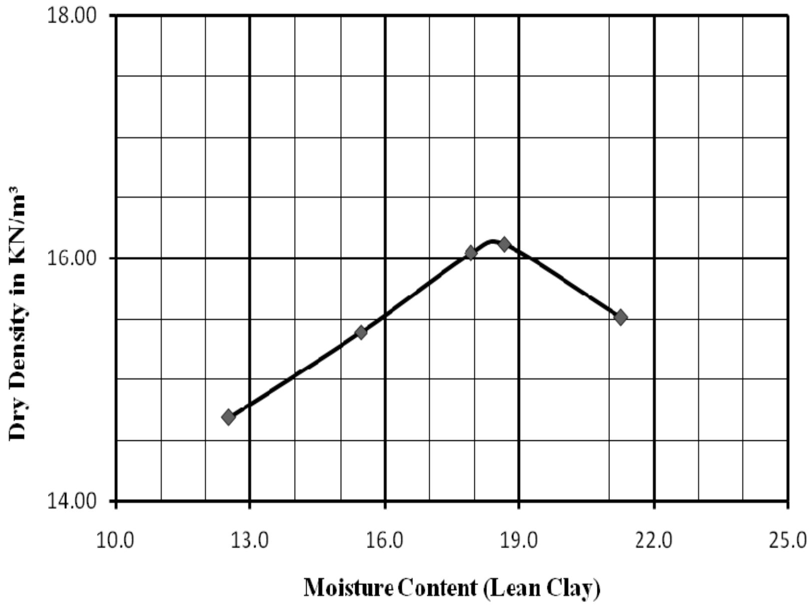

Fig. A.9 Compaction Curve of Lean Clay

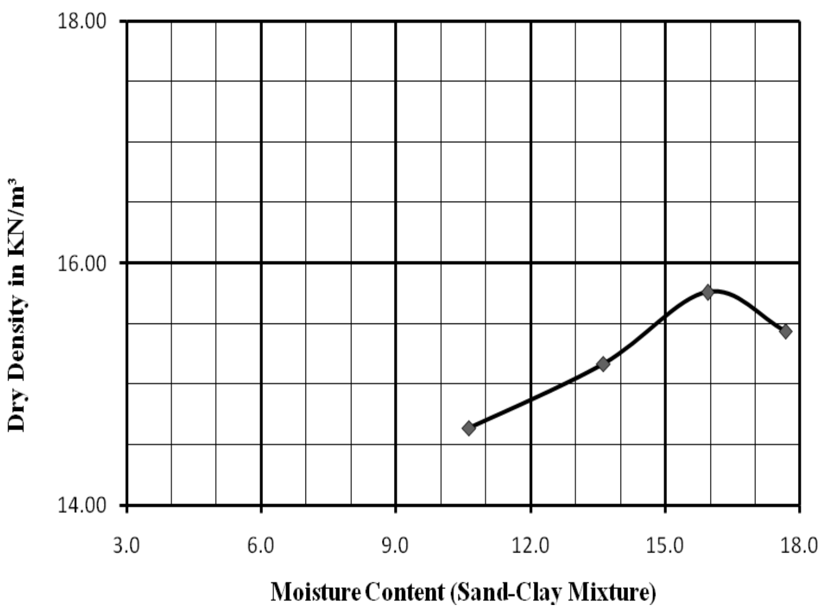

Fig. A.10 Compaction Curve of Lean Clay-Sand Mixture (Clay=90% & Sand=10%)

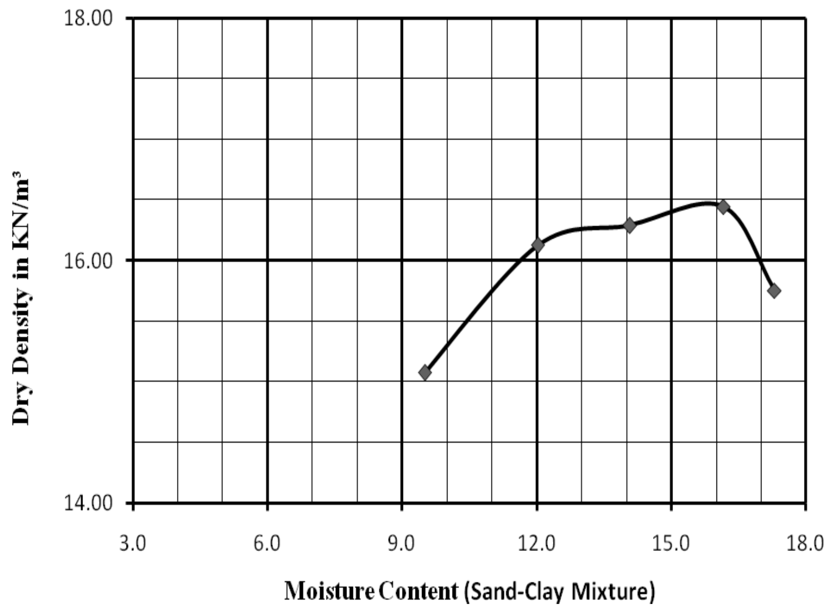

Fig. A.11 Compaction Curve of Lean Clay-Sand Mixture (Clay=80% & Sand=20%)

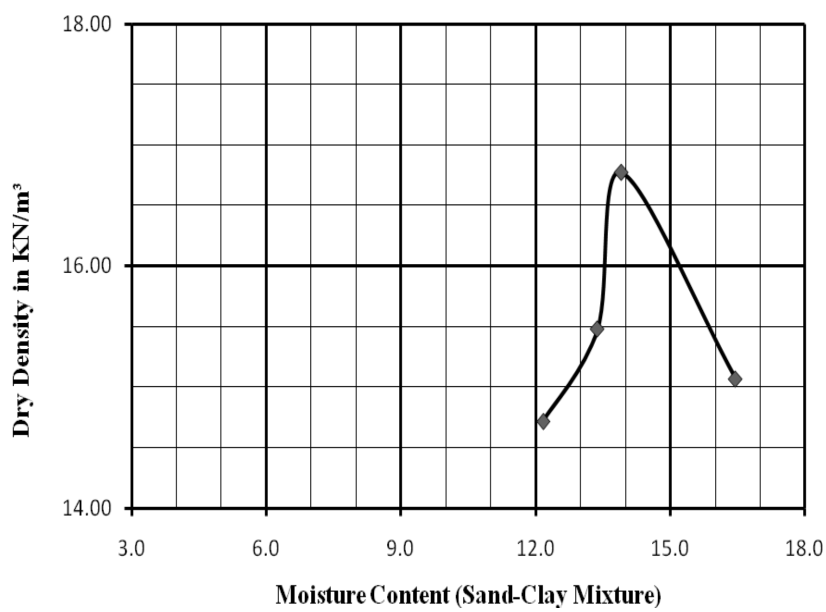

Fig. A.12 Compaction Curve of Lean Clay-Sand Mixture (Clay=70% & Sand=30%)

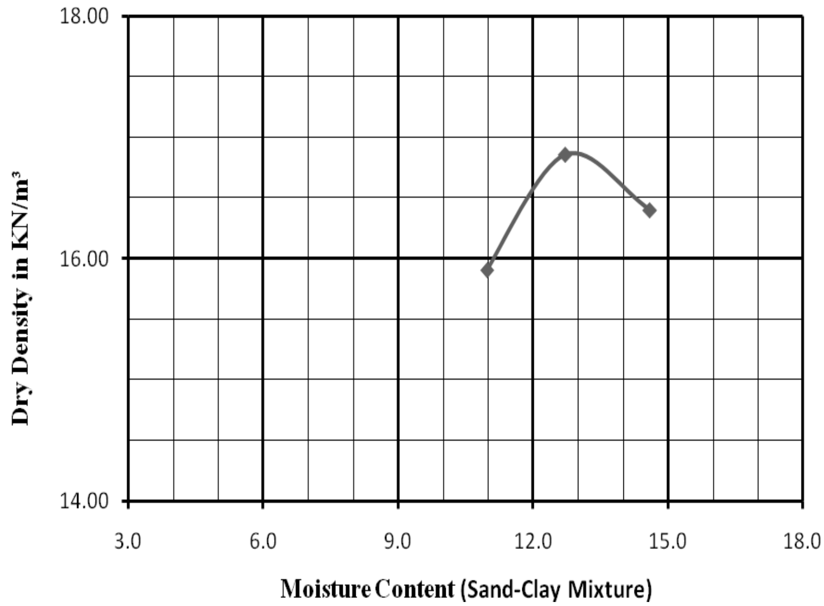

Fig. A.13 Compaction Curve of Lean Clay-Sand Mixture (Clay=60% & Sand=40%)

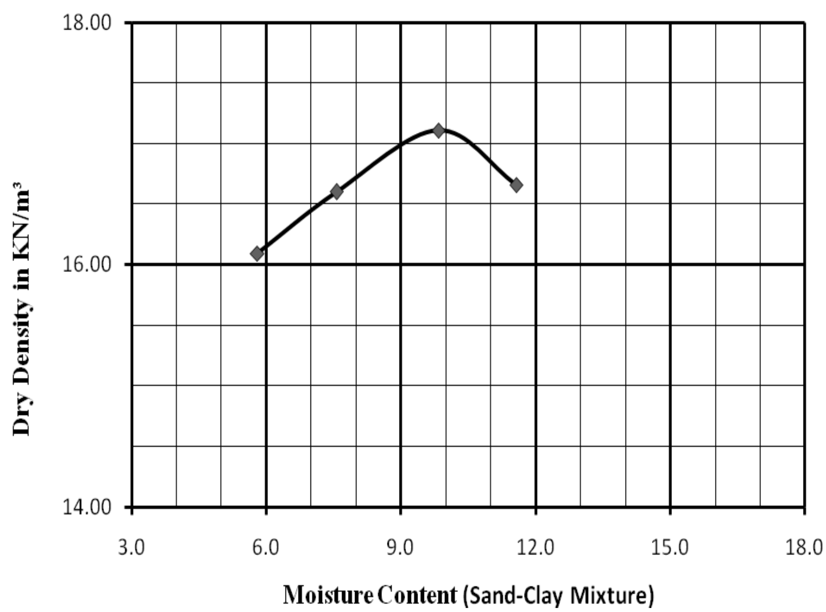

Fig. A.14 Compaction Curve of Lean Clay-Sand Mixture (Clay=50% & Sand=50%)

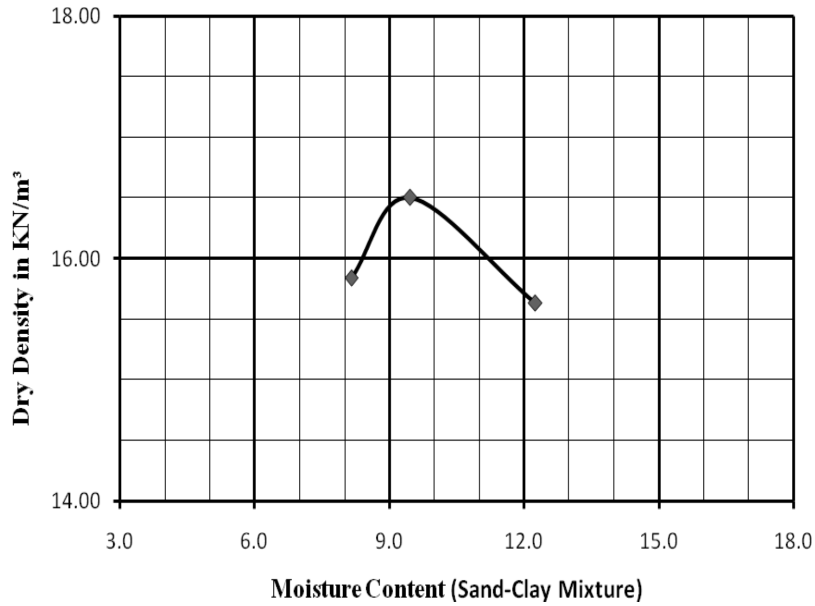

Fig. A.15 Compaction Curve of Lean Clay-Sand Mixture (Clay=40% & Sand=60%)

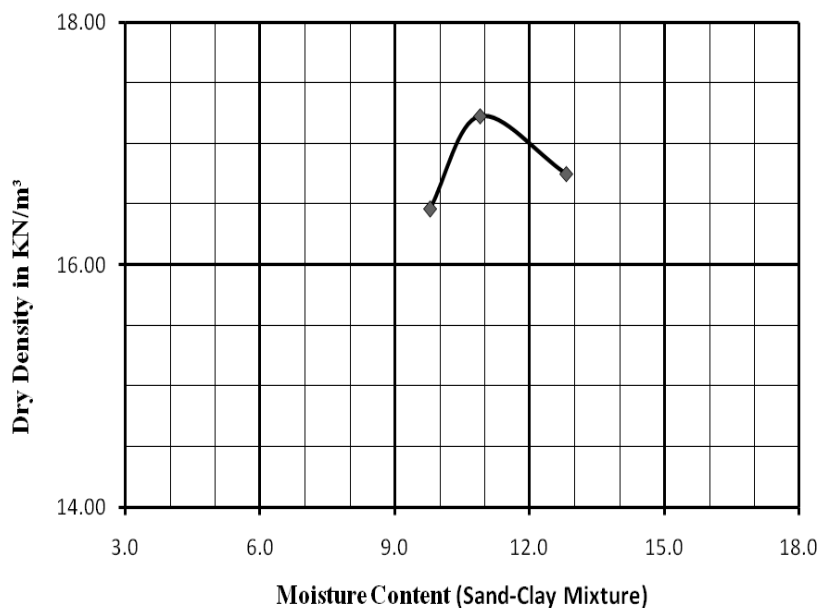

Fig. A.16 Compaction Curve of Lean Clay-Sand Mixture (Clay=30% & Sand=70%)

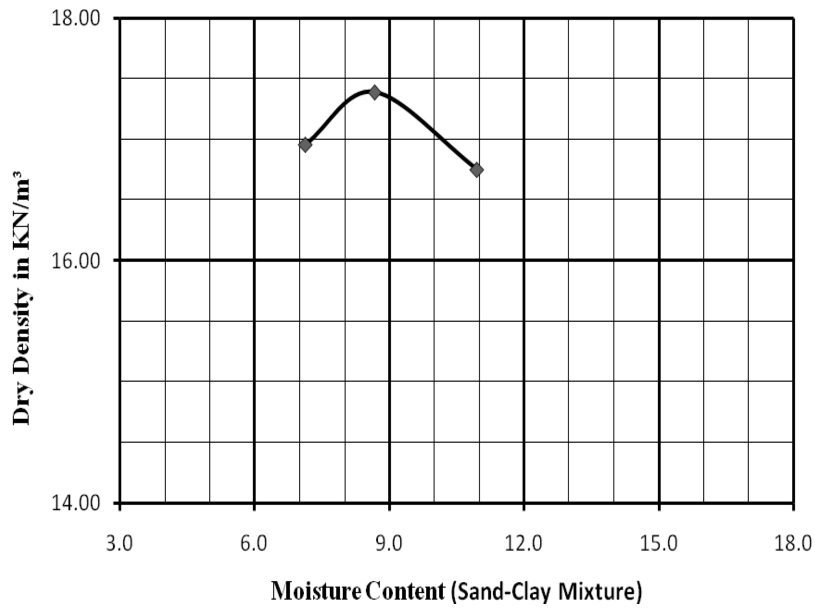

Fig. A.17 Compaction Curve of Lean Clay-Sand mixture (Clay=20% & Sand=80%)

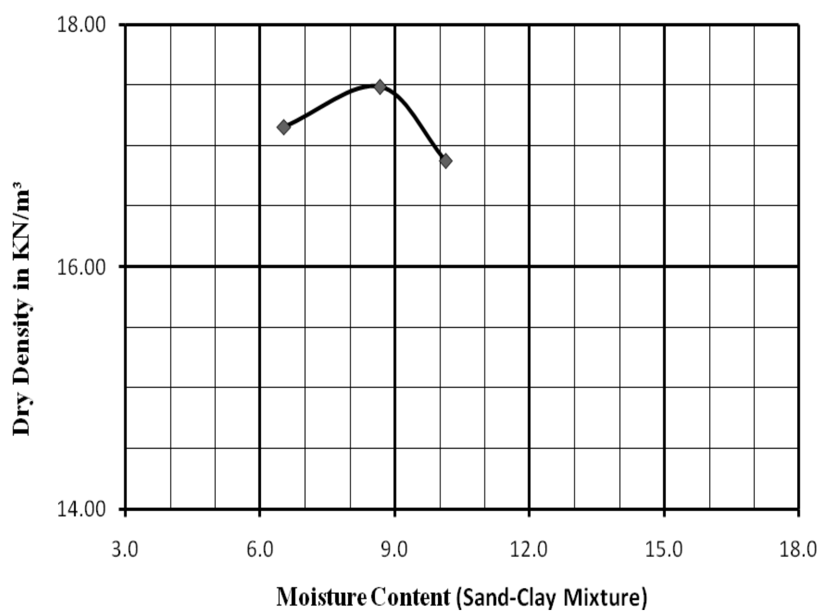

Fig. A.18 Compaction Curve of Lean Clay-Sand mixture (Clay=10% & Sand=90%)

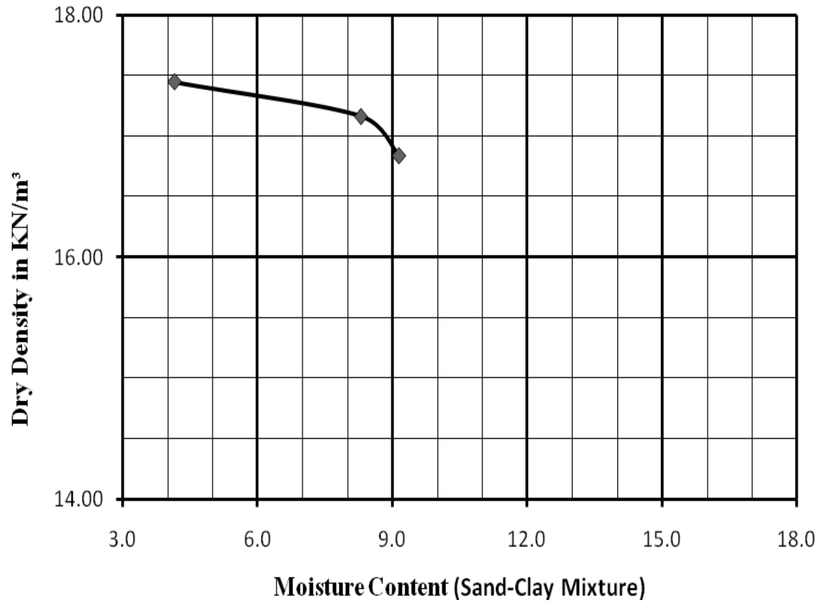

Fig. A.19 Compaction Curve of Lean Clay-Sand mixture (Clay=0% & Sand=100%)

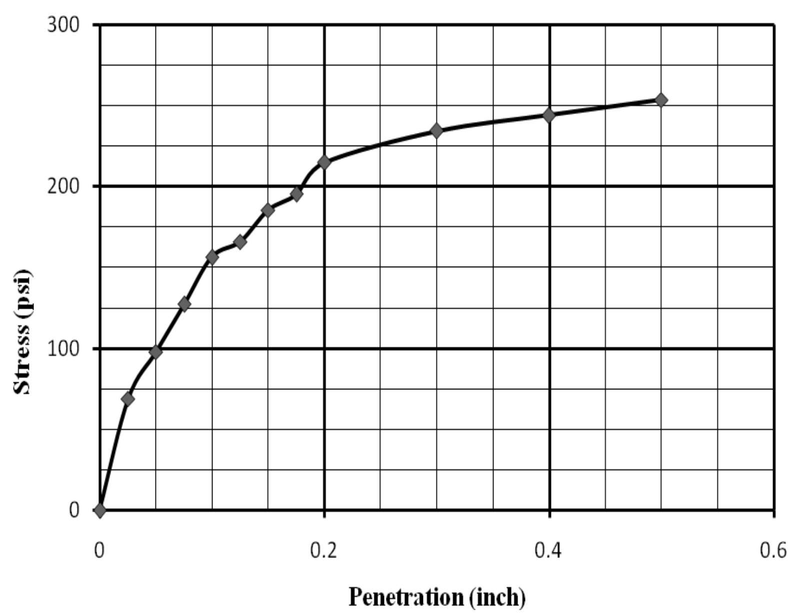

Fig. A.20 Stress vs. Penetration Curve for Sand (Sand=100%)

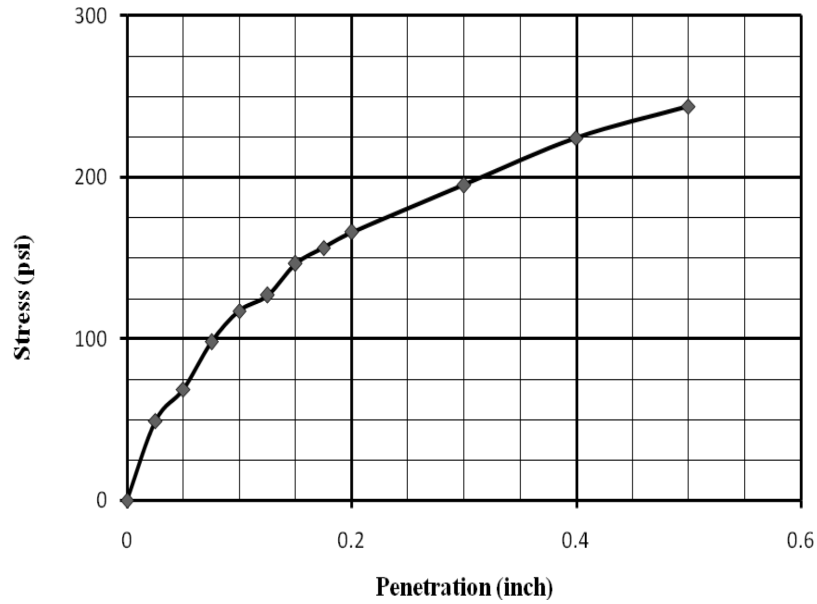

Fig. A.21 Stress vs. Penetration Curve for Lean Clay-Sand Mixture (Clay=10% & Sand=90%)

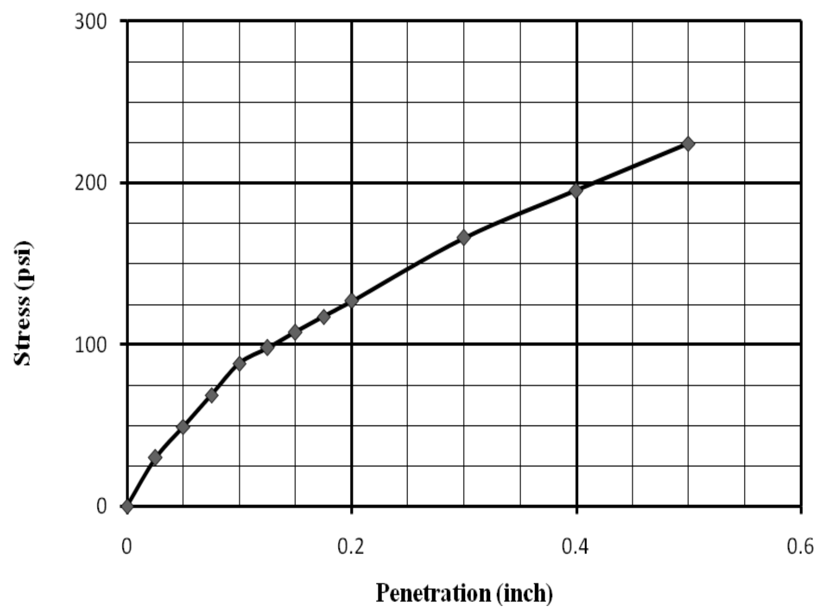

Fig. A.22 Stress vs. Penetration Curve for Lean Clay-Sand Mixture (Clay=30% & Sand=70%)

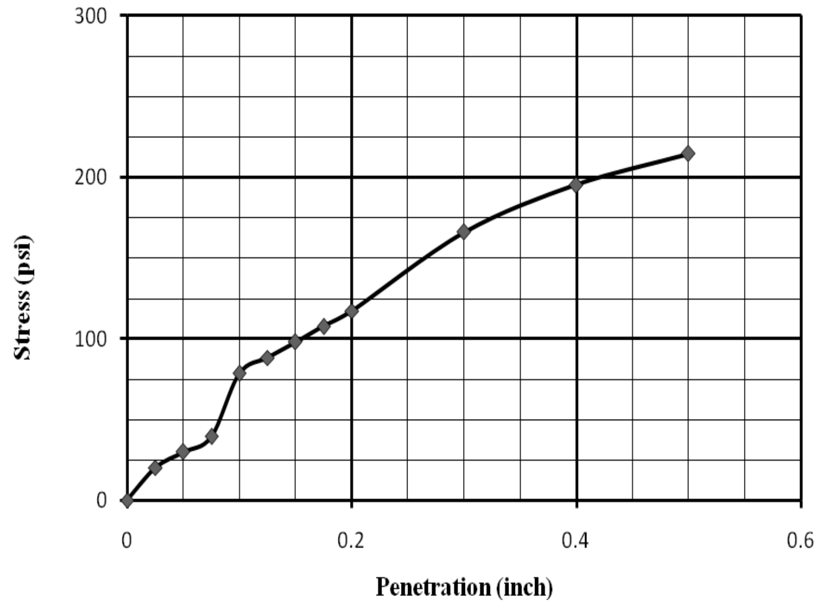

Fig. A.23 Stress vs. Penetration Curve for Lean Clay-Sand Mixture

(Clay=50% & Sand=50%)

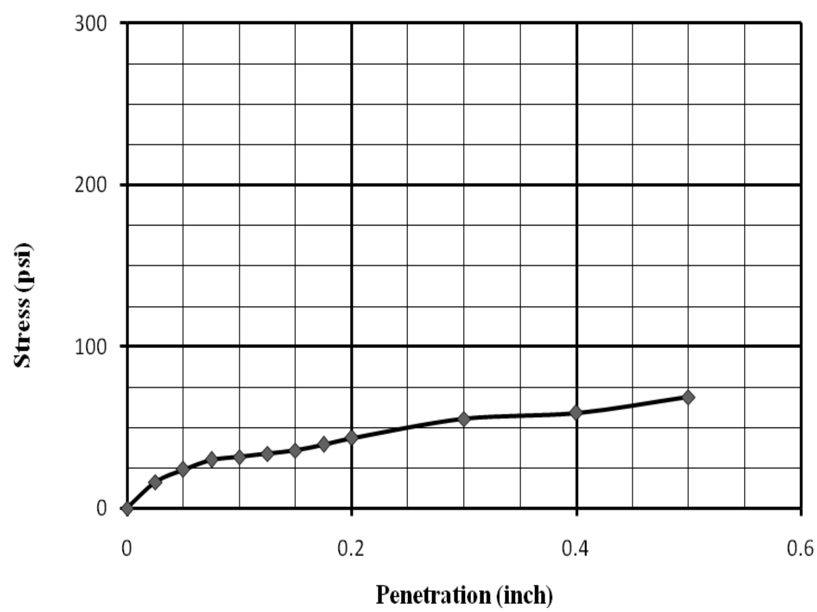

Fig. A.24 Stress vs. Penetration Curve for Lean Clay-Sand Mixture

(Clay=70% & Sand=30%)

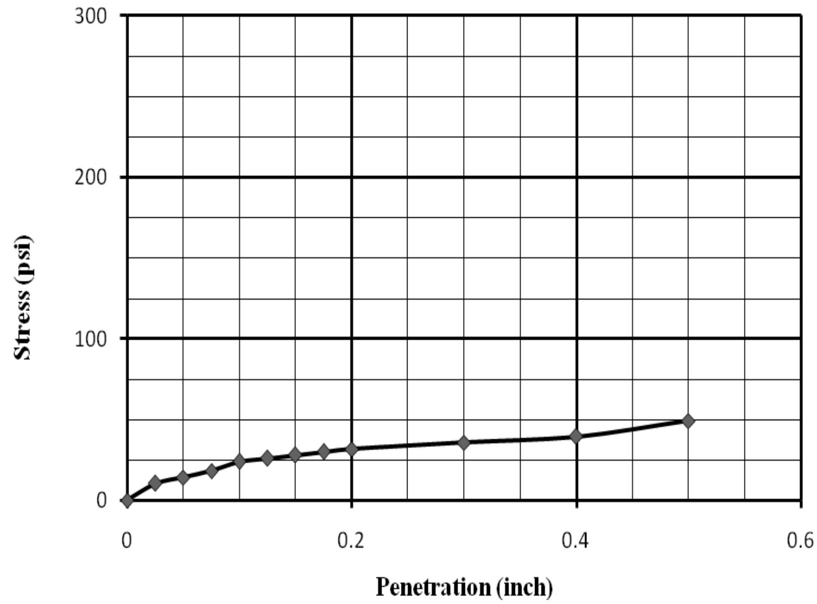

Fig. A.25 Stress vs. Penetration Curve for Lean Clay-Sand Mixture

(Clay=90% & Sand=10%)

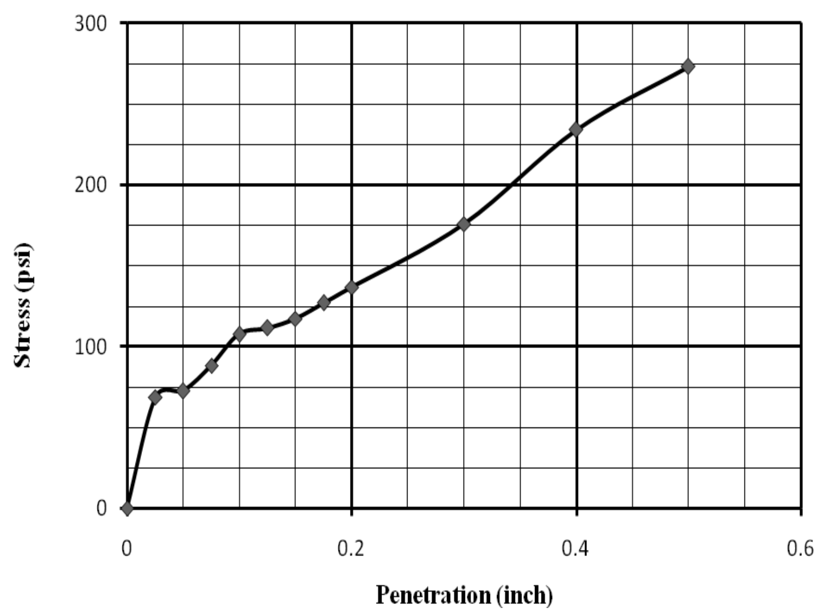

Fig. A.26 Stress vs. Penetration Curve for Lean Clay-Sand Mixture Compacted by Standard Effort (Clay=70% & Sand=30%)

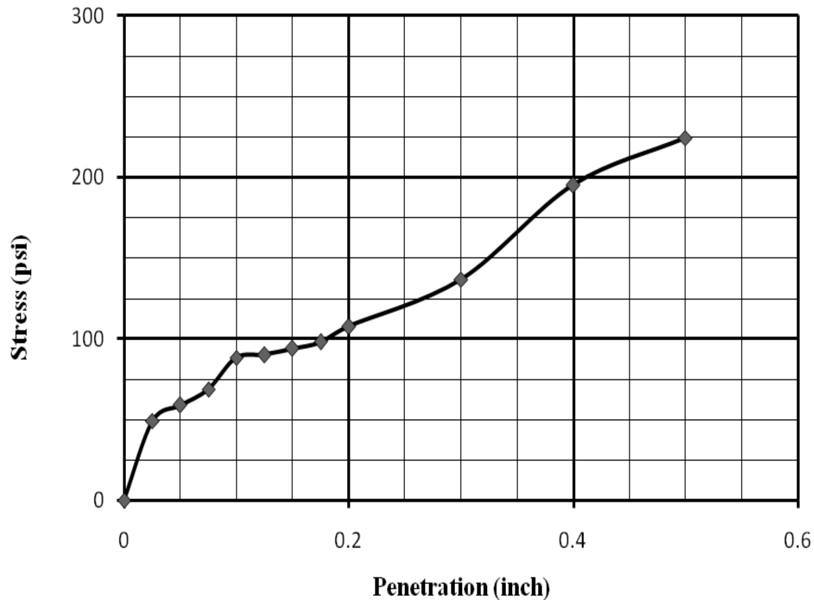

Fig. A.27 Stress vs. Penetration Curve for Lean Clay-Sand Mixture Compacted by Standard Effort (Clay=80% & Sand=20%)

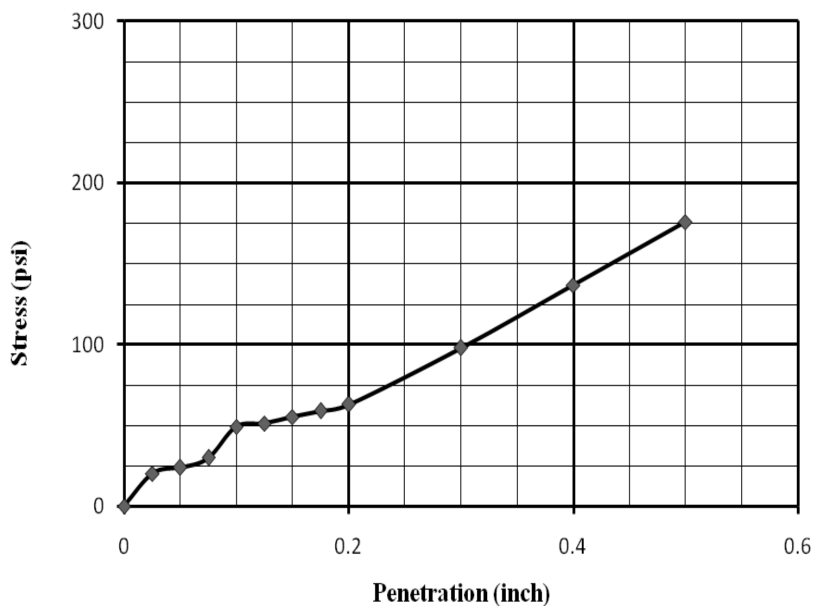

Fig. A.28 Stress vs. Penetration Curve for Lean Clay-Sand Mixture Compacted by Standard Effort (Clay=90% & Sand=10%)

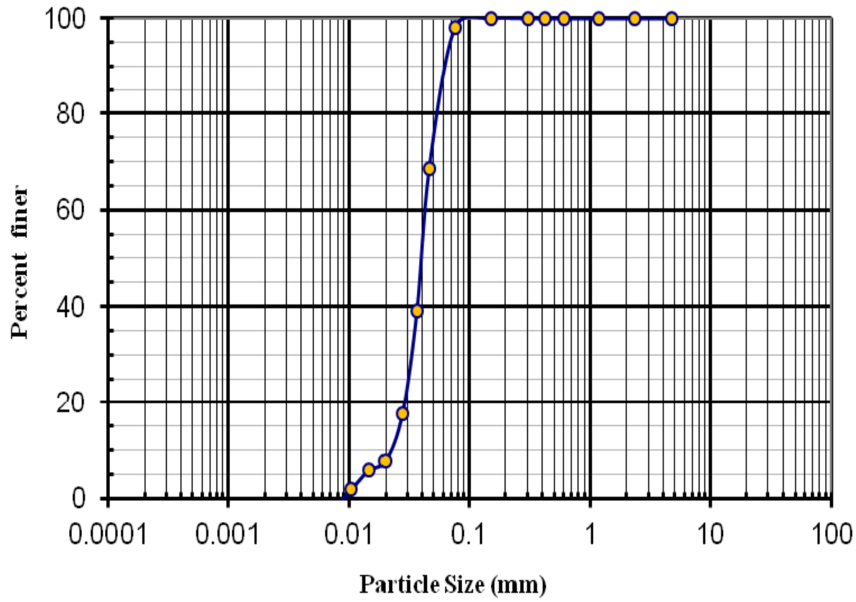

Fig. A.29 Typical Grain size Distribution Curve of experimented silt sample

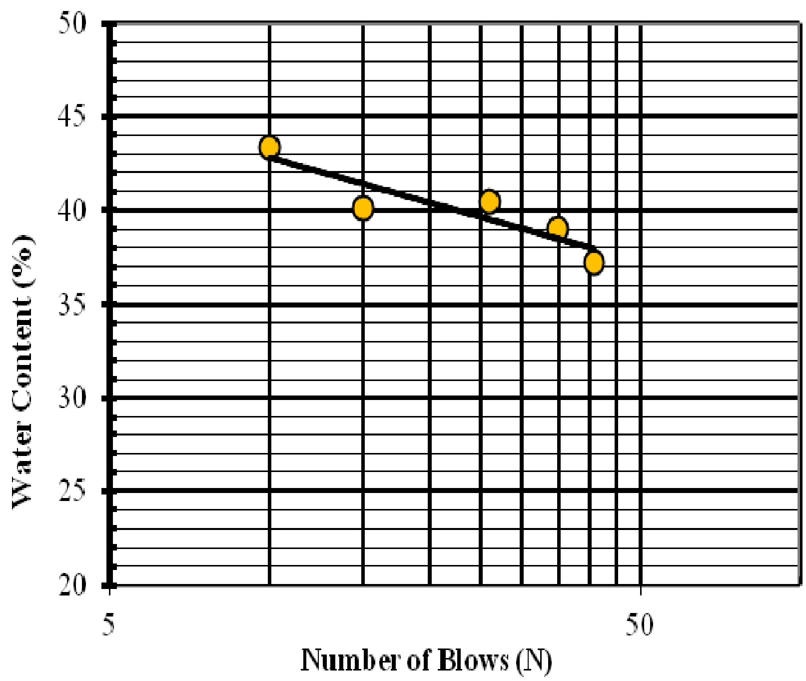

Fig. A.30 Flow curve of Lean Clay-Silt mixture (Clay=90% & Silt=10%)

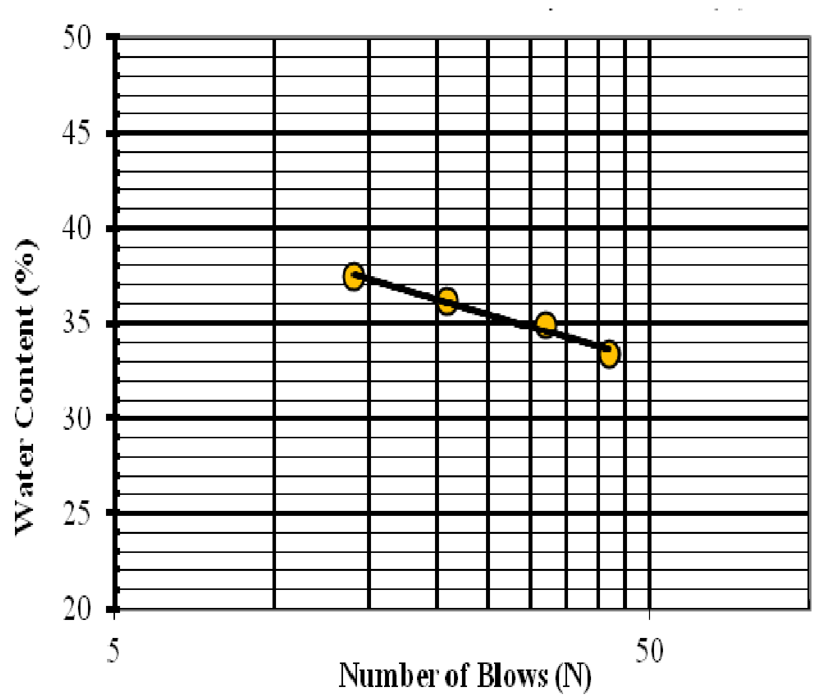

Fig. A.31 Flow curve of Lean Clay-Silt mixture (Clay=80% & Silt=20%)

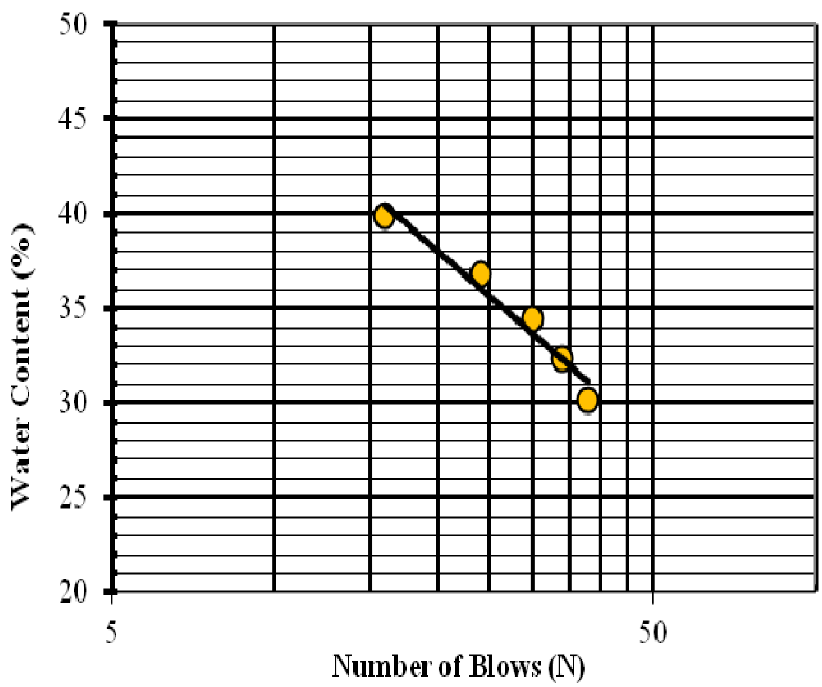

Fig. A.32 Flow curve of Lean Clay-Silt mixture (Clay=70% & Silt=30%)

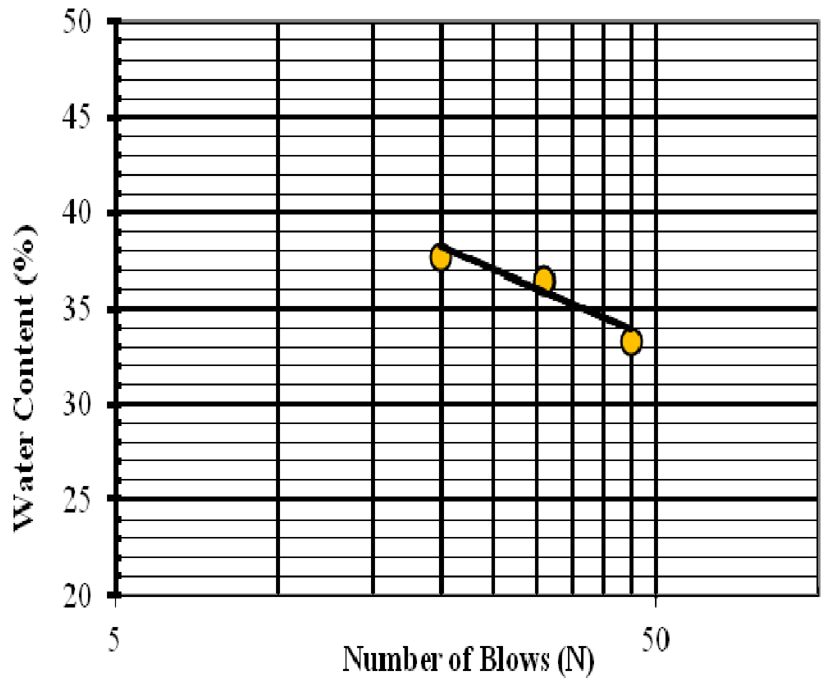

Fig. A.33 Flow curve of Lean Clay-Silt mixture (Clay=60% & Silt=40%)

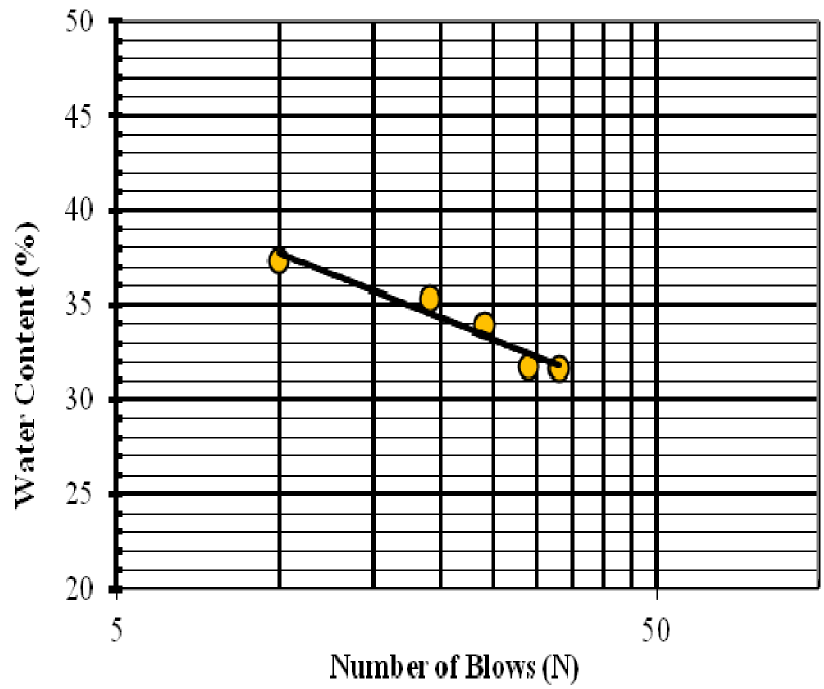

Fig. A.34 Flow curve of Lean Clay-Silt mixture (Clay=50% & Silt=50%)

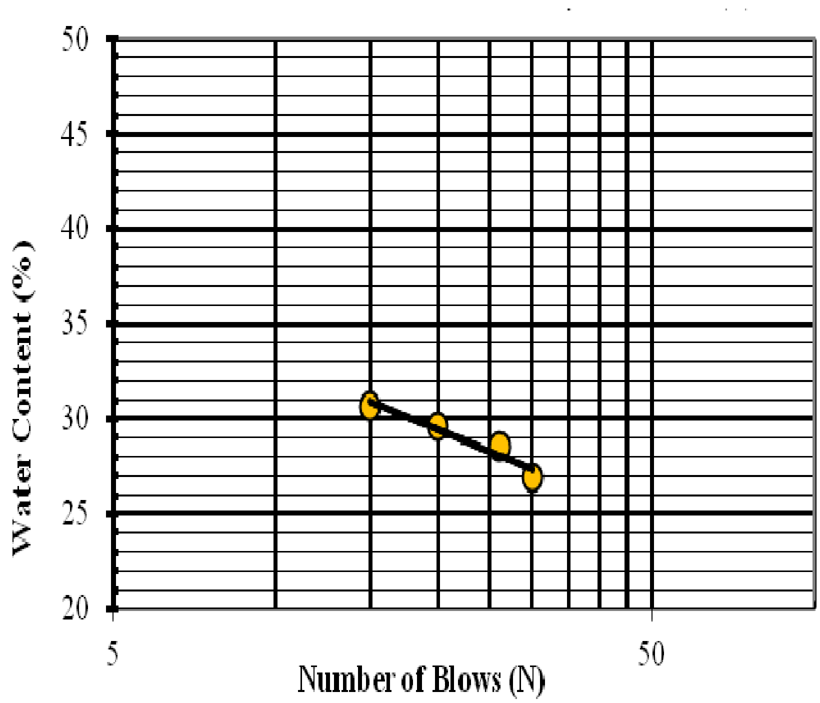

Fig. A.35 Flow curve of Lean Clay-Silt mixture (Clay=40% & Silt=60%)

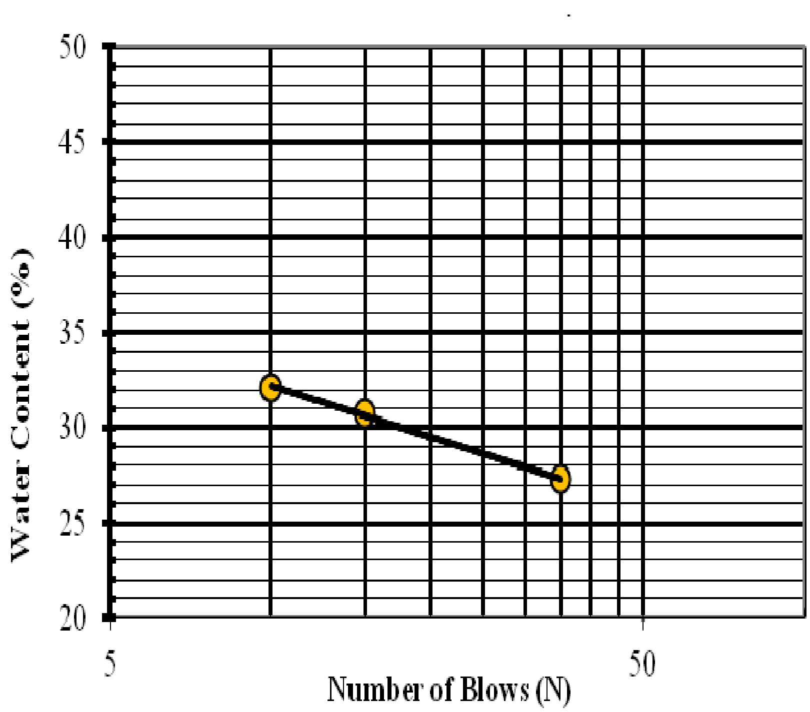

Fig. A.36 Flow curve of Lean Clay-Silt mixture (Clay=30% & Silt=70%)

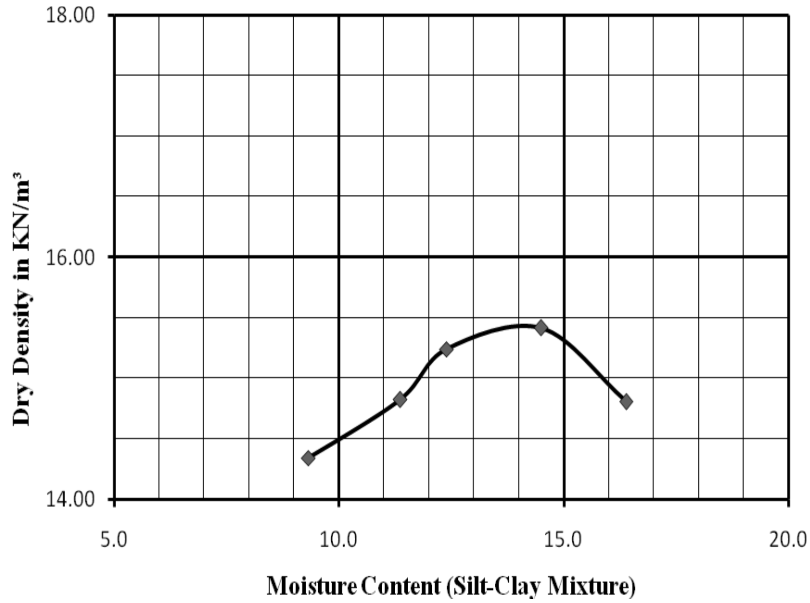

Fig. A.37 Compaction Curve of Lean Clay-Silt Mixture (Clay=90% & Silt=10%)

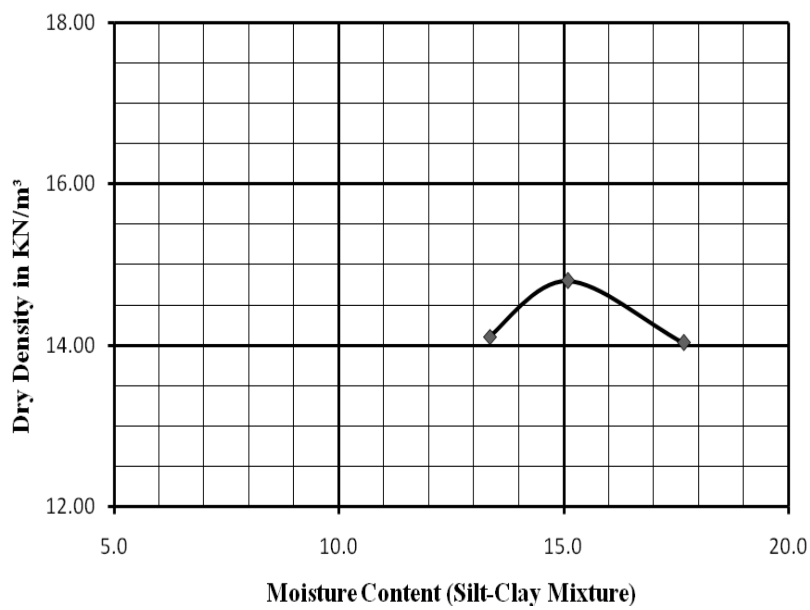

Fig. A.38 Compaction Curve of Lean Clay-Silt Mixture (Clay=80% & Silt=20%)

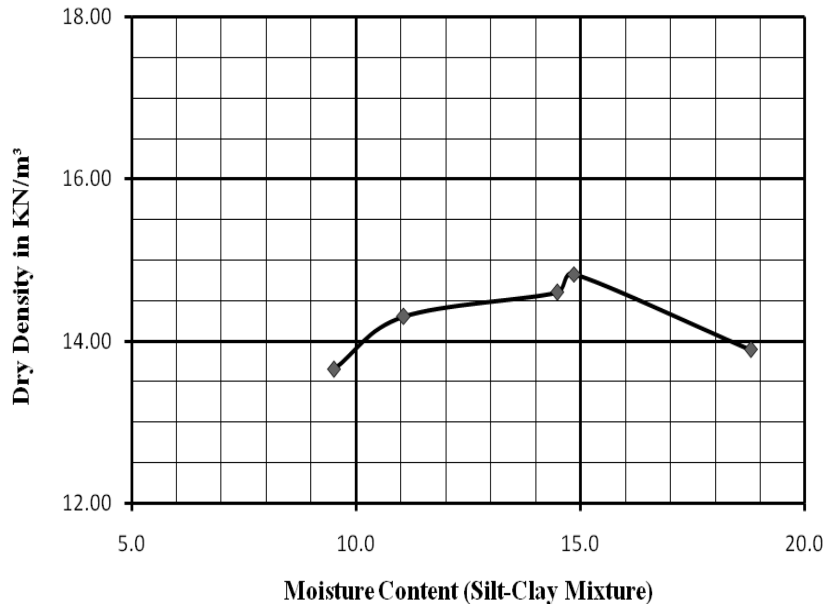

Fig. A.39 Compaction Curve of Lean Clay-Silt Mixture (Clay=70% & Silt=30%)

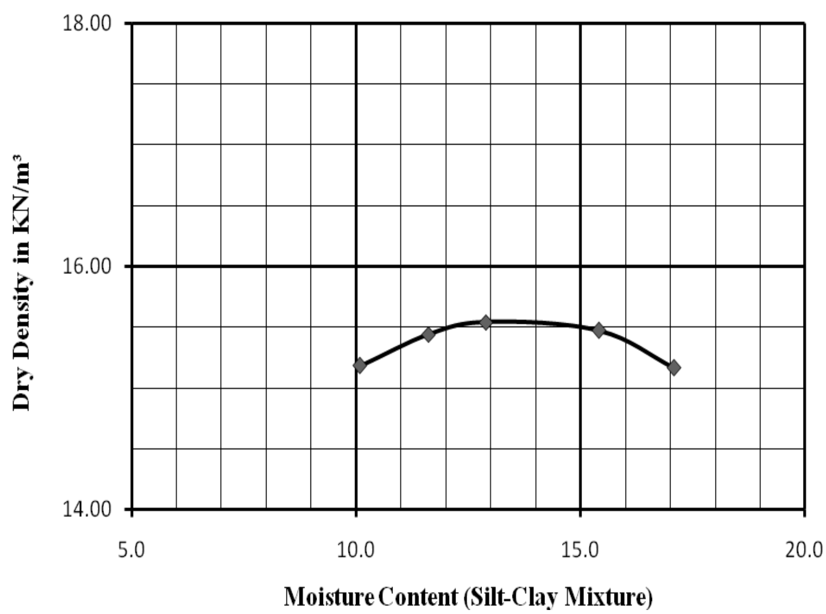

Fig. A.40 Compaction Curve of Lean Clay-Silt Mixture (Clay=60% & Silt=40%)

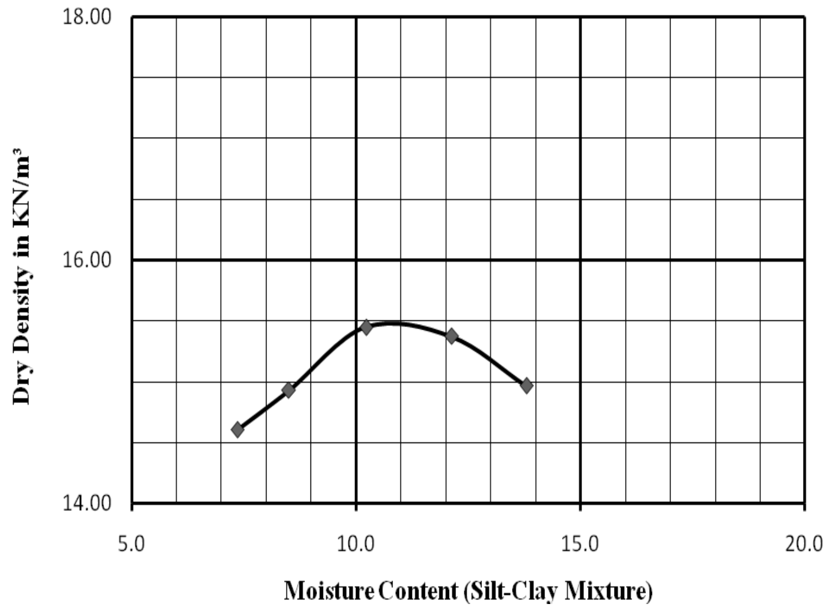

Fig. A.41 Compaction Curve of Lean Clay-Silt Mixture (Clay=50% & Silt=50%)

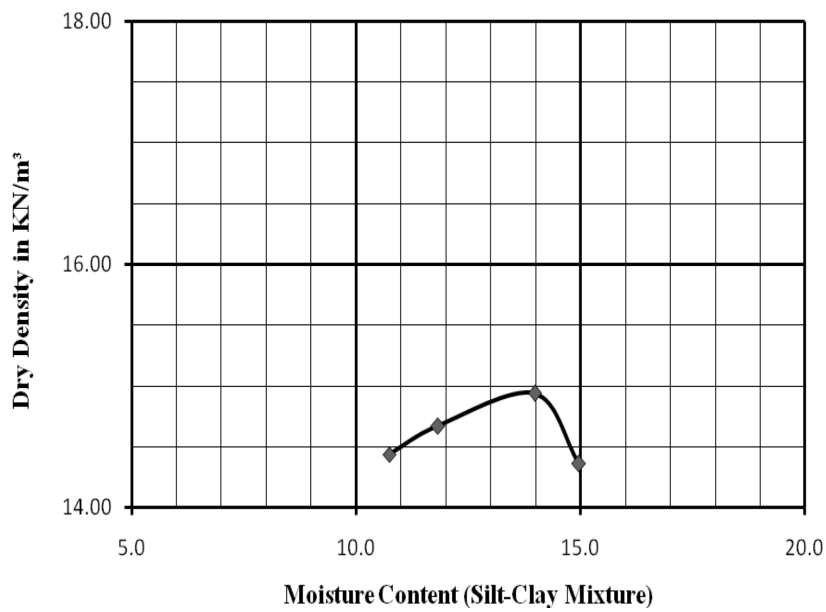

Fig. A.42 Compaction Curve of Lean Clay-Silt Mixture (Clay=40% & Silt=60%)

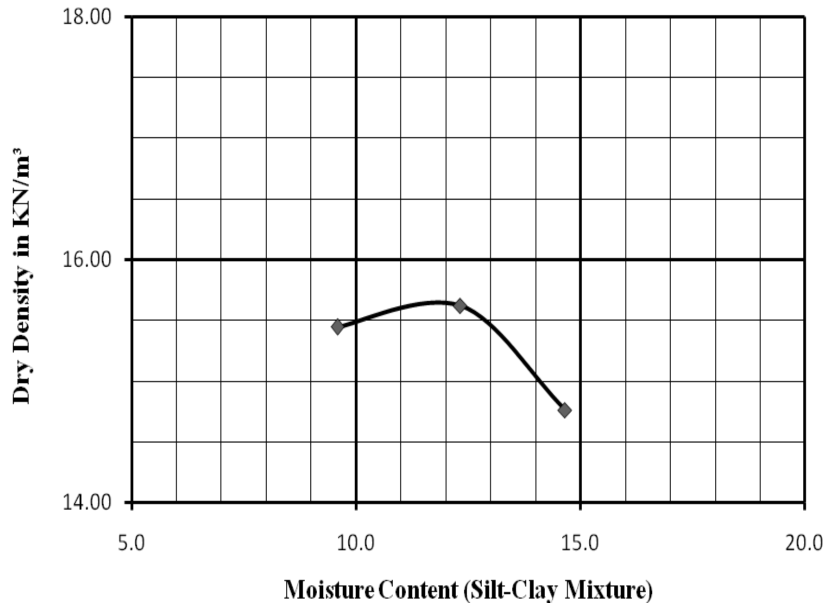

Fig. A.43 Compaction Curve of Lean Clay-Silt Mixture (Clay=30% & Silt=70%)

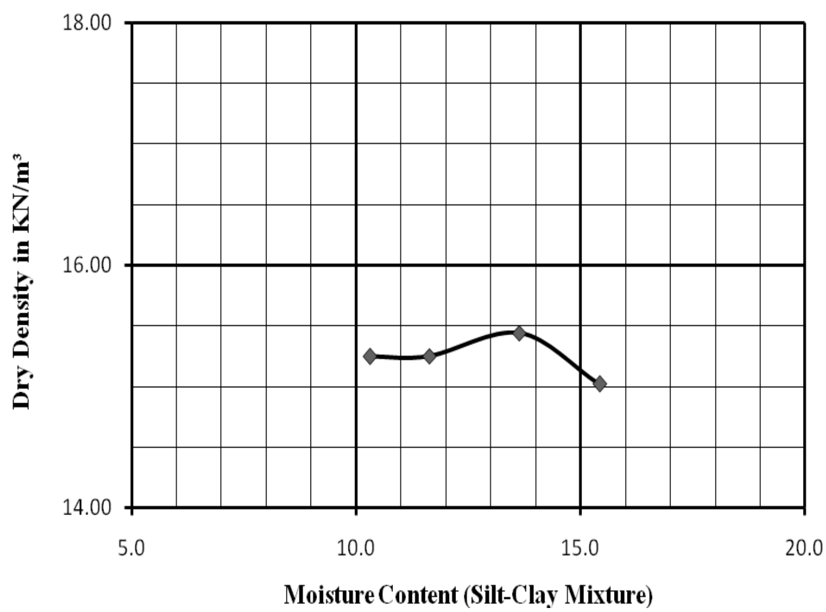

Fig. A.44 Compaction Curve of Lean Clay-Silt Mixture (Clay=20% & Silt=80%)

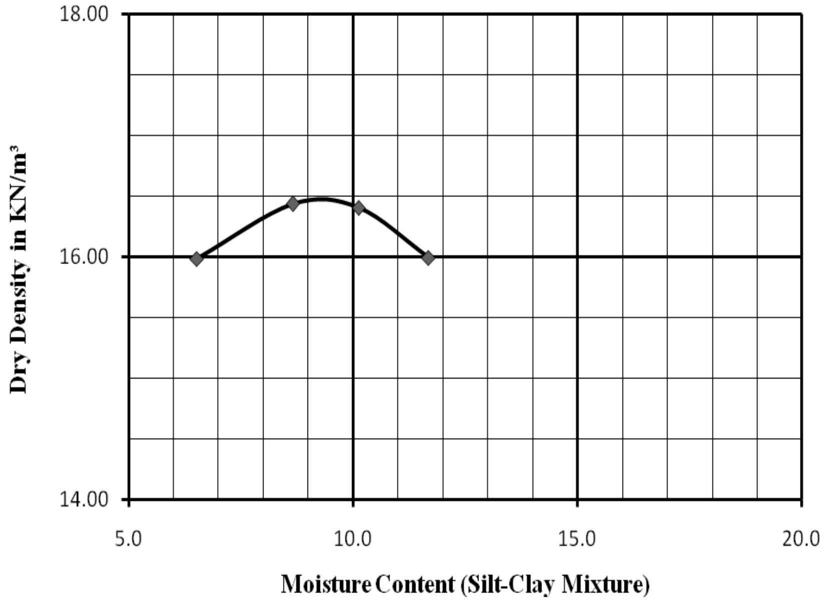

Fig. A.45 Compaction Curve of Lean Clay-Silt Mixture (Clay=10% & Silt=90%)

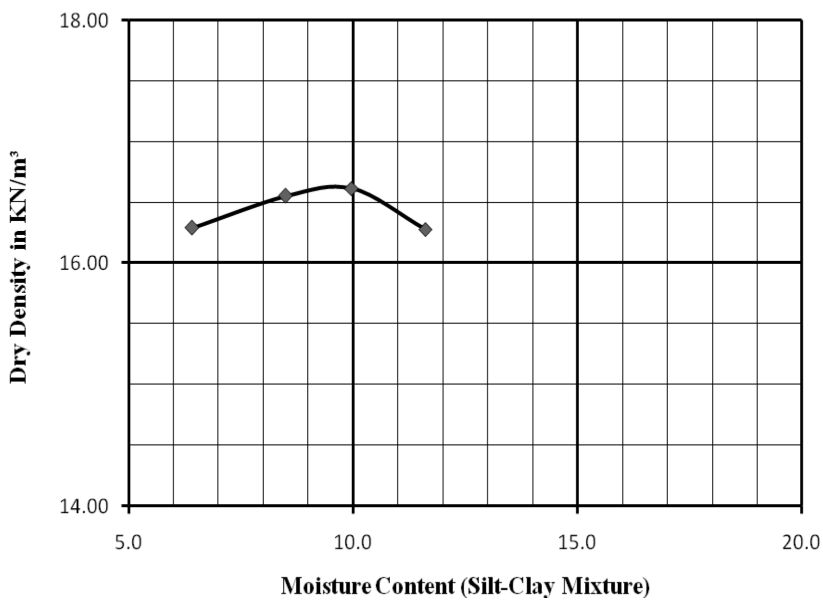

Fig. A.46 Stress vs. Penetration Curve for Lean Clay-Silt Mixture

(Clay=0% & Silt=100%)

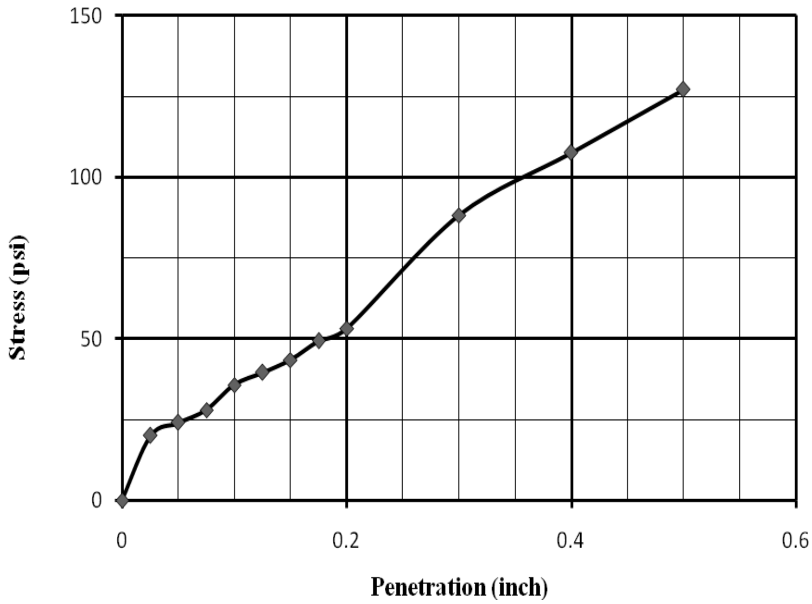

Fig. A.47 Stress vs. Penetration Curve for Lean Clay-Silt Mixture

(Clay=0% & Silt=100%)

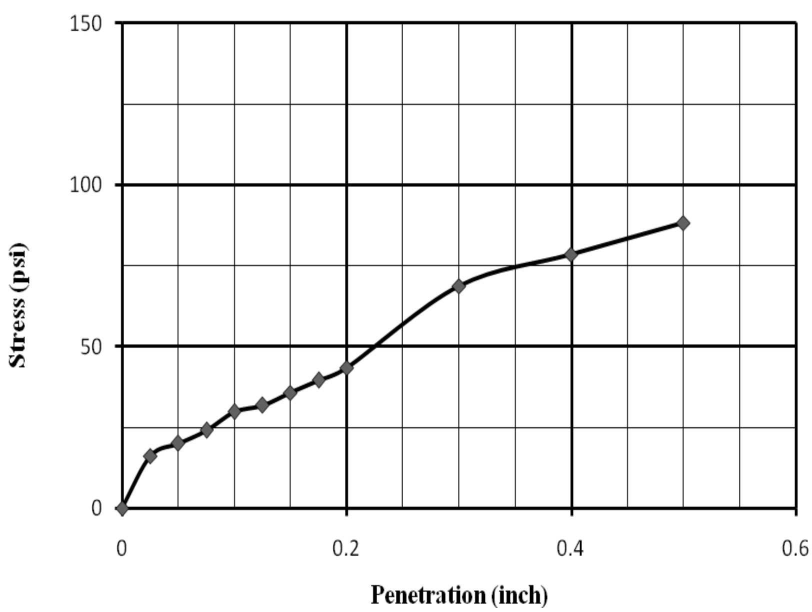

Fig. A.48 Stress vs. Penetration Curve for Lean Clay-Silt Mixture

(Clay=10% & Silt=90%)

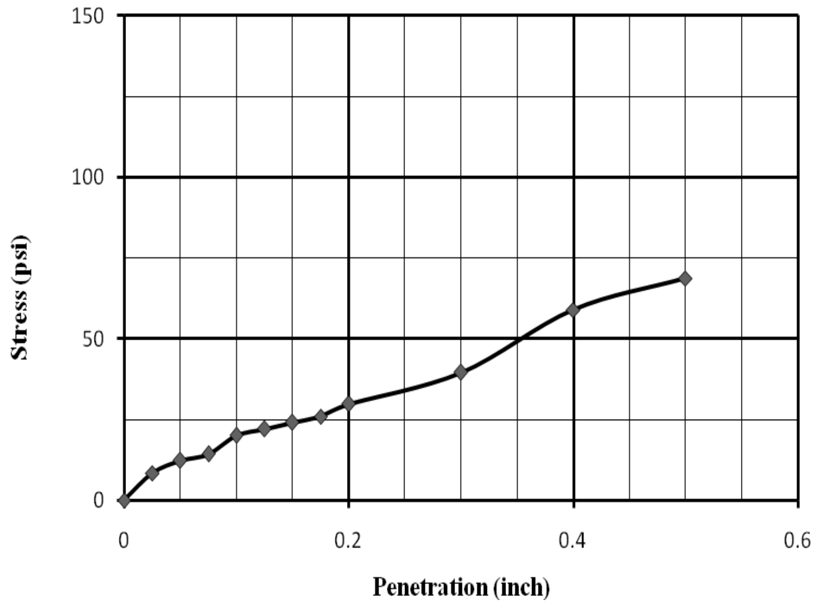

Fig. A.49 Stress vs. Penetration Curve for Lean Clay-Silt Mixture

(Clay=30% & Silt=70%)

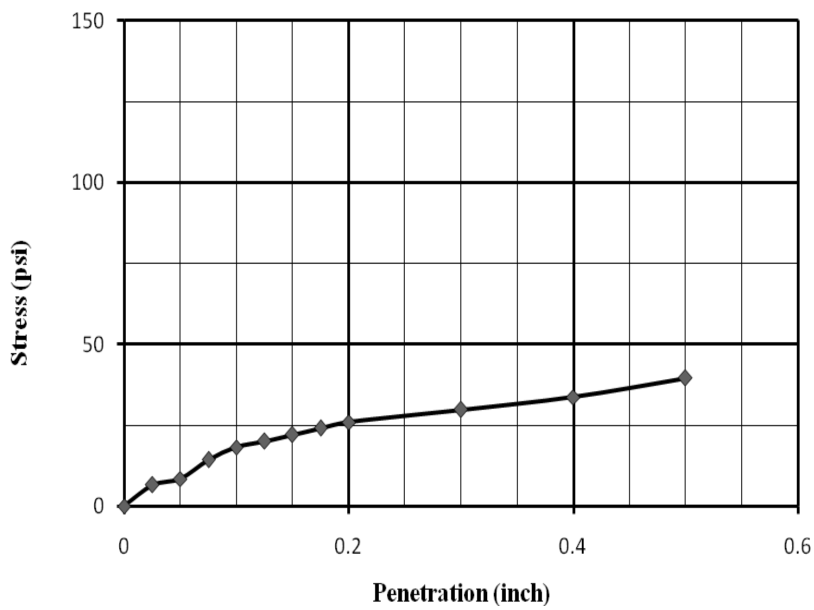

Fig. A.50 Stress vs. Penetration Curve for Lean Clay-Silt Mixture

(Clay=50% & Silt=50%)

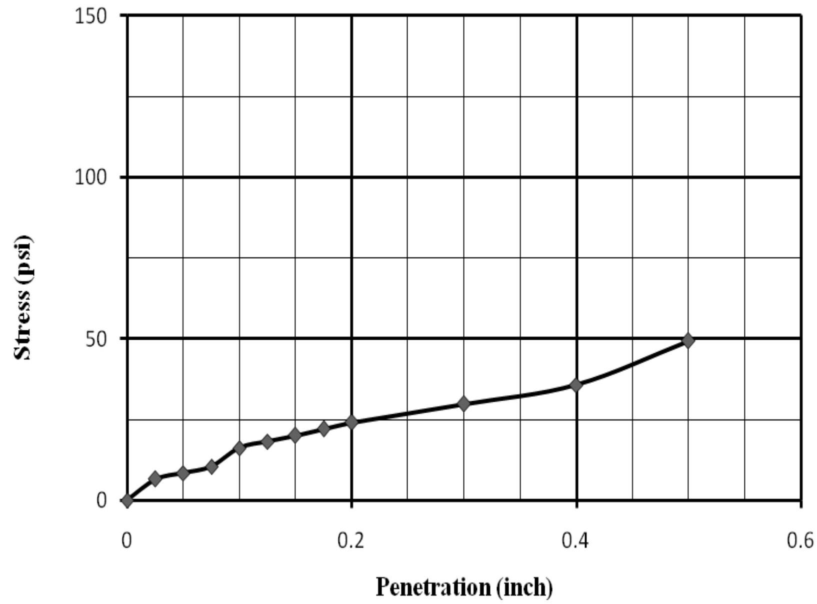

Fig. A.51 Stress vs. Penetration Curve for Lean Clay-Silt Mixture

(Clay=70% & Silt=30%)

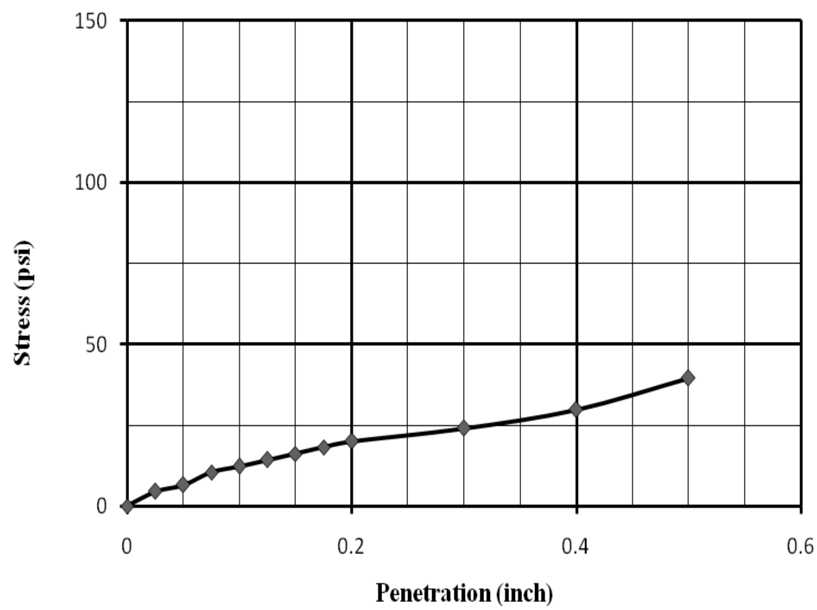

Fig. A.52 Stress vs. Penetration Curve for Lean Clay-Silt Mixture

(Clay=90% & Silt=10%)

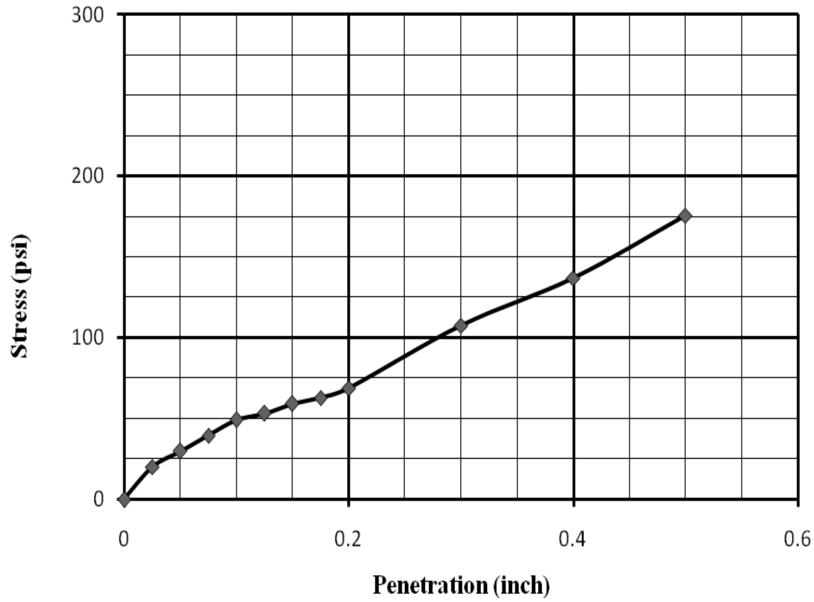

Fig. A.53 Stress vs. Penetration Curve for Lean Clay-Silt Mixture Compacted by Standard Effort (Silt=100%)

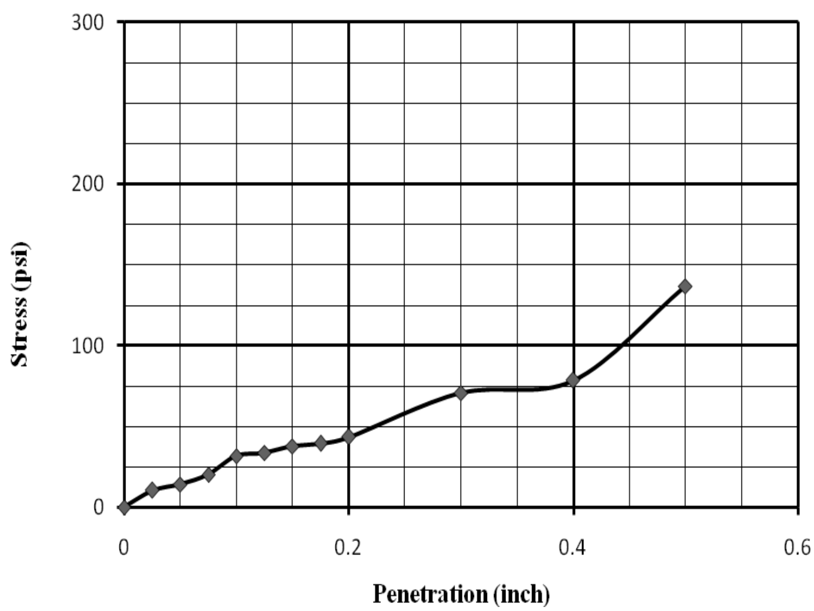

Fig. A.54 Stress vs. Penetration Curve for Lean Clay-Silt Mixture Compacted by Standard Effort (Clay=80% & Silt=20%)

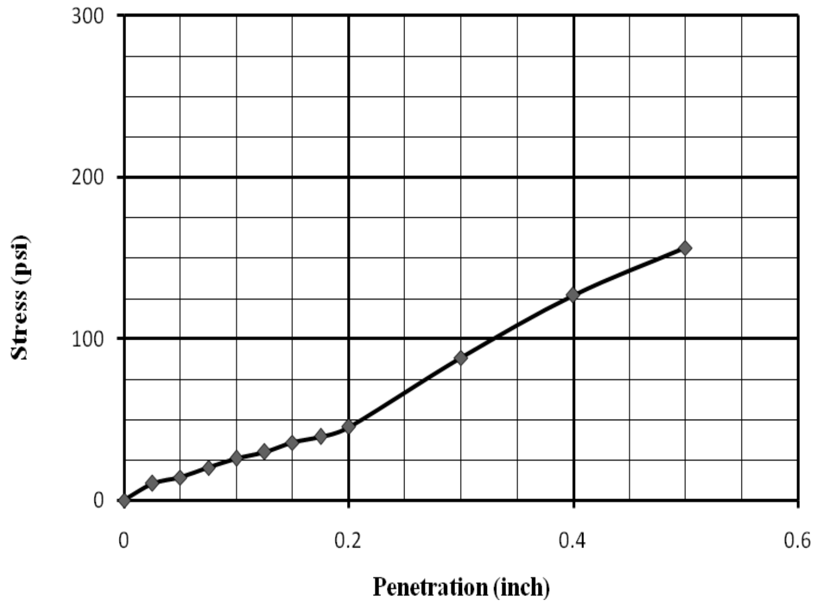

Fig. A.55 Stress vs. Penetration Curve for Lean Clay-Silt Mixture Compacted by Standard Effort (Clay=90% & Silt=10%)