International Journal of Scientific & Engineering Research, Volume 6, Issue 8, August-2015
ISSN 2229-5518

SELECTION OF SUITABLE STABILIZATION METHOD FOR PROBLEMATIC SOIL

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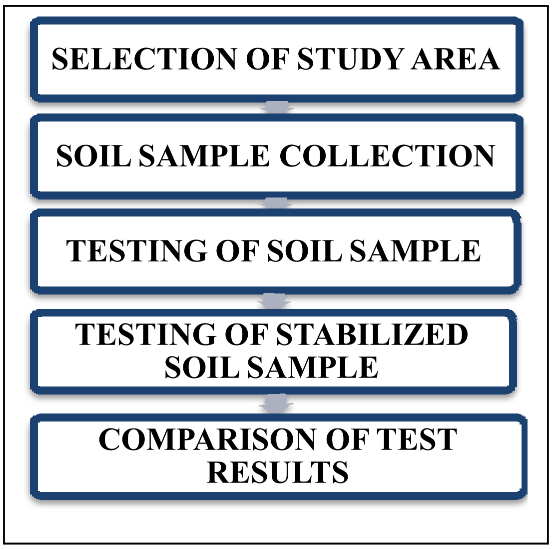
CHAPTER – 1

INTRODUCTION

1.1.GENERAL

Soil is an unconsolidated material that has resulted from disintegration of rocks. The type and characteristics of soil depend largely on its origin transportation causes the sizes and shapes of particles to alter and sort into sizes. The engineering properties is permeability, consolidation and shear strength of a soil deposit are governed by the mode of formation stress history, ground water condition and physic chemical characteristics of the parent material. Based on method of formation soil may be categorized as residual and transported. Residual soils soils are formed from weathering of rocks and practically remain at location of origin with little or no movement of individual soil particles. Transported soils are true that have formed at one location but transported and deposited at another location. Soils that are carried and deposited by rivers are called alluvial deposit. River delta are formed in this manner. These deposits are weak and compressible and produce problems for foundation. Wind transported soil: Fine grained soils such as silts and clay can be transported by wind in arid regions. Marine deposits: The marine deposits all along Indian coast are generally derived from terrestrial sources. The deposits are very soft to soft clays and the thickness varies from 5m to 20m. These deposit generally need a pre treatment before application of any external load. In order to prevent failures during construction controlled loading should be planned.

1.2. METHODOLOGY



1.3.1 Introduction

Soil stabilization refers to the process of changing soil properties to improve strength and durability. There are many techniques for soil stabilization, including compaction, dewatering and by adding material to the soil. This summary will focus on mechanical and chemical stabilization based adding IRC materials. Mechanical stabilization improves soil properties by mixing other soil materials with the target soil to change the gradation and therefore change the engineering properties. Chemical stabilization used the addition of cementitious or pozzolanic materials to improve the soil properties. Chemical stabilization has traditionally relied on Portland cement and lime for chemical stabilization. There a number of IRC materials that can be used individually, or mixed with other materials, to achieve soil stabilization.

1.3.2 Principles of Soil Stabilization

Natural soil is both a complex and variable material. Yet because of its universal availability and its low cost winning it offers great opportunities for skilful use as an engineering material. Not uncommonly, however the soil at any particular locality is unsuited, wholly or partially, to the requirements of the construction engineer. A basic decision must therefore be made whether to:

- Accept the site material as it is and design to standards sufficient to meet the restrictions imposed by its existing quality.
- Remove the site material and replace with a superior material.
- Alter the properties of existing soil so as to create a new site material capable of better meeting the requirements of the task in hand.

The latter choice, the alteration of soil properties to meet specific engineering requirements is known as "Soil stabilization". It must also be recognized that stabilization not necessarily a magic wand by which every soil property is changed for the better. Correct usage demands a clear recognition of which soil properties must be upgraded, and this specific engineering requirement is an important element in the decision whether or not to stabilize. Properties of soil may be altered in many ways, among which are included chemical, thermal, mechanical and other means.

The chief properties of a soil with which the construction engineer is concerned are: volume stability, strength,

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ISSN 2229-5518

permeability, and durability. Methods of stabilization may be grouped under two main types:

- Modification or improvement of a soil property of the existing soil without any admixture.
- Modification of the properties with the help of admixtures. Compaction and drainage are the examples of the first type, which improve the inherent shear strength of soil. Examples of the second type are: mechanical stabilization, stabilization with cement, lime, bitumen and chemicals etc,.

1.3.3 Methods of soil stabilization

There are many techniques for soil stabilization, including compaction, dewatering and by adding material to the soil. These soil stabilization methods are listed below.

- Mechanical stabilization
- Cement stabilization
- Lime stabilization
- Chemical stabilization
- Bituminous stabilization
- Grouting concrete stabilization
- Geotextile stabilization
- Reinforced earth stabilization.

1.4 COMPONENTS OF STABILIZATION.

Soil stabilization involves the use of stabilizing agents (binder materials) in weak soils to improve its geotechnical properties such as compressibility, strength, permeability and durability. The components of stabilization technology include soils and or soil minerals and stabilizing agent or binders (cementitious materials).

1.4.1.Soil

Most of stabilization has to be undertaken in soft soils (silty, clayey peat or organic soils) in order to achieve desirable engineering properties. A clay soil compared to others has a large surface area due to flat and elongated particle shapes. On the other hand, silty materials can be sensitive to small change in moisture and, therefore, may prove difficult during stabilization. Peat soils and organic soils are rich in water content of up to about 2000%, high porosity and high organic content. The consistency of peat soil can vary from muddy to fibrous, and in most cases, the deposit is shallow, but in worst cases, it can extend to several meters below the surface. Organic soils have high exchange capacity; it can hinder the hydration process by retaining the calcium ions liberated during the hydration of calcium silicate and calcium aluminate in the cement to satisfy the exchange capacity. In such soils, successful stabilization has to depend on the proper selection of binder and amount of binder added.

1.4.2Stabilizing Agents

These are hydraulic (primary binders) or non-hydraulic (secondary binders) materials that when in contact with water or in the presence of pozzolanic minerals reacts with water to form cementitious composite materials. The commonly used binders are:

- Cement
- Lime
- Fly ash
- Gypsum
- Blast furnace slag etc.

1.5 FACTORS AFFECTING THE STRENGTH OF STABILIZED SOIL

Presence of organic matters, sulphates, sulphides and carbon dioxide in the stabilized soils may contribute to undesirable strength of stabilized materials.

1.5.1 Organic Matter

In many cases, the top layers of most soil constitute large amount of organic matters. However, in well drained soils organic matter may extend to a depth of 1.5 m. Soil organic matters react with hydration product e.g. calcium hydroxide ($Ca(OH)2$) resulting into low pH value. The resulting low pH value may retard the hydration process and affect the hardening of stabilized soils making it difficult or impossible to compact.

1.5.2 Sulphates

The use of calcium-based stabilizer in sulphate-rich soils causes the stabilized sulphate rich soil in the presence of excess moisture to react and form calcium sulphoaluminate (ettringite) and or thamausite, the product which occupy a greater volume than the combined volume of reactants. However, excess water to one initially present during the time of mixing may be required to dissolve sulphate in order to allow the reaction to proceed.

1.5.3 Sulphides

In many of waste materials and industrial by-product, sulphides in form of iron pyrites ($FeS2$) may be present. Oxidation of $FeS2$ will produce sulphuric acid, which in the presence of calcium carbonate, may react to form gypsum (hydrated calcium sulphate) according to the reactions (i) and (ii) below

i. $2FeS2 + 2H2O + 7O2 = 2FeSO4 + 2H2SO4$
ii. $CaCO3 + H2SO4 + H2O = CaSO4.2\,H2O + CO2$

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ISSN 2229-5518

The hydrated sulphate so formed, and in the presence of excess water may attack the stabilized material in a similar way as sulphate. Even so, gypsum can also be found in natural soil.

1.5.4 Compaction

In practice, the effect of addition of binder to the density of soil is of significant importance. Stabilized mixture has lower maximum dry density than that of unstabilized soil for a given degree of compaction. The optimum moisture content increases with increasing binders. In cement stabilized soils, hydration process takes place immediately after cement comes into contact with water. This process involves hardening of soil mix which means that it is necessary to compact the soil mix as soon as possible. Any delay in compaction may result in hardening of stabilized soil mass and therefore extra compaction effort may be required to bring the same effect.That may lead to serious bond breakage and hence loss of strength. Stabilized clay soils are more likely to be affected than other soils due to alteration of plasticity properties of clays. In contrary to cement, delay in compaction for lime-stabilized soils may have some advantages. Lime stabilized soil require mellowing period to allow lime to diffuse through the soil thus producing maximum effects on plasticity. After this period, lime stabilized soil may be remixed and given its final compaction resulting into remarkable strength.

1.5.5 Moisture Content

In stabilized soils, enough moisture content is essential not only for hydration process to proceed but also for efficient compaction. Fully hydrated cement takes up about 20% of its own weight of water from the surrounding ; on other hand, Quicklime (CaO) takes up about 32% of its own weight of water from the surrounding. Insufficient moisture content will cause binders to compete with soils in order to gain these amounts of moisture. For soils with great soil-water affinity (such as clay, peat and organic soils), the hydration process may be retarded due to insufficient moisture content, which will ultimately affect the final strength.

1.5.6 Temperature

Pozzolanic reaction is sensitive to changes in temperature. In the field, temperature varies continuously throughout the day. Pozzolanic reactions between binders and soil particles will slow down at low temperature and result into lower strength of the stabilized mass. In cold regions, it may be advisable to stabilize the soil during the warm season.

1.7.7 Freeze-Thaw and Dry-Wet Effect

Stabilized soils cannot withstand freeze-thaw cycles. Therefore, in the field, it may be necessary to protect the stabilized soils against frost damage.
Shrinkage forces in stabilized soil will depend on the chemical reactions of the binder. Cement stabilized soil are susceptible to frequent dry-wet cycles due to diurnal changes in temperature which may give rise to stresses within a stabilized soil and, therefore, should be protected from such effects.

1.6 STABILIZATION METHODS

1.6.1 In–Situ Stabilization

The method involves on site soil improvement by applying stabilizing agent without removing the bulk soil. This technology offer benefit of improving soils for deep foundations, shallow foundations and contaminated sites. Planning of the design mix involves the selection and assessment of engineering properties of stabilized soil and improved ground. The purpose is to determine the dimensions of improved ground on the basis of appropriate stability and settlement analyses to satisfy the functional requirements of the supported structure. The technology can be accomplished by injection into soils a cementitious material such cement and lime in dry or wet forms. The choice to either use dry or wet deep mixing methods depend among other things; the in-situ soil conditions, in situ moisture contents, effectiveness of binders to be used, and the nature of construction to be founded. Depending on the depth of treatment, the in situ stabilization may be regarded as either deep mixing method or mass stabilization.

1.6.2 Deep Mixing Method

The deep mixing method involves the stabilization of soils at large depth. It is an in situ ground modification technology in which a wet or dry binder is injected into the ground and blended with in situ soft soils (clay, peat or organic soils) by mechanical or rotary mixing tool. Depending on applications, the following patterns may be produced; single patterns, block patterns, panel pattern or stabilized grid pattern. Note that, the aim is to produce the stabilized soil mass which may interact with natural soil and not, to produce too stiffly stabilized soil mass like a rigid pile which may independently carry out the design load. The increased strength and stiffness of stabilized soil should not, therefore, prevent an effective interaction and load distribution between the stabilized soil and natural soil. Thus the design load should be distributed and carried out partly by natural soil and partly by stabilized soil mass.

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1.6.3 Wet Mixing

Applications of wet deep mixing involve binder turned into slurry form, which is then injected into the soil through the nozzles located at the end of the soil auger. The mixing tool comprise of drilling rod, transverse beams and a drill end with head. There are some modifications to suit the need and applications. For instance, the Trench cutting Re-mixing deep method (TRD) developed by circa Japan, in 1993 provides an effective tool for construction of continuous cutoff wall without the need for open trench. The method uses a crawler-mounted, chainsaw-like mixing tool to blend in-situ soil with cementitious binder to create the soil-cement wall. It further consists of a fixed post on which cutting, scratching teeth ride on a rotating chain and injection ports deliver grout into treatment zone. Wall depths up to 45 m having width between 0.5 m and 0.9 m are achievable. The wall quality for groundwater barrier is high with permeability between 1 x 10-6 and 1 x 10-8 cm/s (www.HaywardBaker.com). Similar to TRD, in 1994, Germany developed the FMI (Misch-Injektionsverfahren) machine. The FMI machine has a special cutting arm (trencher), along which cutting blades are rotated by two chain system. The cutting arm can be inclined up to 80 degrees and is dragged through the soil behind the power unit (Stocker and Seidel, 2005). Like TRD, the soil is not excavated, but mixed with binder which is supplied in slurry form through injection pipes and outlets mounted along the cutting arm.

1.6.4 Dry Mixing

Dry mixing (DM) method is clean, quiet with very low vibration and produces no spoil for disposal. It has for many years extensively used in Northern Europe and Japan. The method involves the use of dry binders injected into the soil and thoroughly mixed with moist soil. The soil is premixed using specialized tool during downward penetration, until it reaches the desired depth. During withdrawal of the mixing tool, dry binder are then injected and mixed with premixed soil leaving behind a moist soil mix column. In Scandinavians countries and Sweden in particular, this method is referred to as Lime Cement Column (LCC), whereas, in Italy, the method is termed as Trevimix and in Japan, the same technology is called dry jet mixing (DJM).

1.7 FLYASH

1.7.1 Introduction

Fly ash, also known as flue-ash, is one of the residues generated in combustion, and comprises the fine particles that rise with the flue gases. Ash which does not rise is termed bottom ash. In an industrial context, fly ash usually refers to ash produced during combustion of coal. Fly ash is generally captured by electrostatic precipitators or other particle filtration equipment before the flue gases reach the chimneys of coal-fired power plants, and together with bottom ash removed from the bottom of the furnace is in this case jointly known as coal ash. Depending upon the source and makeup of the coal being burned, the components of fly ash vary considerably, but all fly ash includes substantial amounts of silicon dioxide (SiO_2) (both amorphous and crystalline) and calcium oxide(CaO), both being endemic ingredients in many coal-bearing rock strata.

1.7.2 Chemical composition

Fly ash material solidifies while suspended in the exhaust gases and is collected by electrostatic precipitators or filter bags. Since the particles solidify rapidly while suspended in the exhaust gases, fly ash particles are generally spherical in shape and range in size from 0.5 µm to 300 µm. The major consequence of the rapid cooling is that only few minerals will have time to crystallize and that mainly amorphous, quenched glass remains. Nevertheless, some refractory phases in the pulverized coal will not melt (entirely) and remain crystalline. In consequence, fly ash is a heterogeneous material. SiO_2, Al_2O_3, Fe_2O_3 and occasionally CaO are the main chemical components present in fly ashes.

1.7.3 Classification

Class F flyash

The burning of harder, older anthracite and bituminous coal typically produces Class F fly ash. This flyash is pozzolanic in nature, and contains less than 20% lime (CaO). Possessing pozzolanic properties, the glassy silica and alumina of Class F fly ash requires a cementing agent, such as Portland cement, quicklime, or hydrated lime, with the presence of water in order to react and produce cementitious compounds. Alternatively, the addition of a chemical activator such as sodium silicate (water glass) to a Class F ash can lead to the formation of a geopolymer.

Class C flyash

Fly ash produced from the burning of younger lignite or subbituminous coal, in addition to having pozzolanic properties, also has some self-cementing properties. In the presence of water, Class C fly ash will harden and gain strength over time. Class C fly ash generally contains more than 20% lime (CaO). Unlike Class F, self-cementing Class C fly ash does not require an activator. Alkali and sulfate (SO_4) contents are generally higher in Class C fly ashes.

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1.8 LIME

1.8.1 Introduction

Lime can be used to treat soils in order to improve their workability and load-bearing characteristics in a number of situations. Quicklime is frequently used to dry wet soils at construction sites and elsewhere, reducing downtime and providing an improved working surface. An even more significant use of lime is in the modification and stabilization of soil beneath road and similar construction projects. Lime can substantially increase the stability, impermeability, and load-bearing capacity of the subgrade. Both quicklime and hydrated lime may be used for this purpose. Application of lime to subgrades can provide significantly improved engineering properties.

1.8.2 Soil Modification

Lime is an excellent choice for short-term modification of soil properties. Lime can modify almost all fine-grained soils, but the most dramatic improvement occurs in clay soils of moderate to high plasticity. Modification occurs because calcium cations supplied by hydrated lime replace the cations normally present on the surface of the clay mineral, promoted by the high pH environment of the lime-water system. Thus, the clay surface mineralogy is altered, producing the following benefits: Plasticity reduction, reduction in moisture-holding capacity (drying), swell reduction, improved stability and ability to construct a solid working platform.

1.8.3 Soil Stabilization

Soil stabilization occurs when lime is added to a reactive soil to generate long-term strength gain through a pozzolanic reaction. This reaction produces stable calcium silicate hydrates and calcium aluminate hydrates as the calcium from the lime reacts with the aluminates and silicates solubilized from the clay. The full-term pozzolanic reaction can continue for a very long period of time, even decades -- as long as enough lime is present and the pH remains high (above 10). As a result, lime treatment can produce high and long-lasting strength gains. The key to pozzolanic reactivity and stabilization is a reactive soil, a good mix design protocol, and reliable construction practices.

1.8.4 Benefits of Soil Stabilization

Lime substantially increases soil resilient modulus values (by a factor of 10 or more in many cases). In addition, when lime is added to soil, users see substantial improvements in shear strength (by a factor of 20 or more in some cases), continued strength over time, even after periods of environmental or load damage (autogenous healing), and long-term durability over decades of service even under severe environmental conditions.

1.9 GYPSUM

1.9.1 Introduction

Gypsum is a by-product available from Phosphoric Acid Plant and used in production of Ammonium Sulphate. GSFC has been first in the country to develop and implement successfully Phospho-Gypsum process for manufacture of Ammonium Sulphate.This product is available in a fine mesh powder form Phosphoric Acid Plant mainly used in Agriculture as soil amendment.

1.9.2 Specifications

$CaSO_4, 2H_20$	94.21 % (Dry basis)
$T - P_2O_5$	0.70 %
Water Soluble P_2O_5	0.28 %
Acid Insoluble	3.58 %

1.9.3 Applications For Agricultural use:

It works as an agent to remove Saline/Alkaline ingredients in the soil. It acts more or less like manure. For industrial use: It is used to manufacture Gypsum boards. It is used in manufacturing lime and in cement industry. It is also used in manufacturing Plaster of Paris. Packing: Available in 50 kg Bag Packing and in loose powder form on "As is where is" basis.

Figure 1.2 Stabilizers of lime, flyash and gypsum

1.10 SOIL CLASSIFICATION

1.10.1 Introduction

Soil classification is the arrangement of soils into different groups such that the soils in a particular group have similar behaviour. As there are a wide variety of soils covering earth, it is desirable to systematize or classify the soils into broad groups of similar behaviour. Soils, in general, may be classified as cohesionless and cohesive or as coarse-grained and fine-grained. These terms, however, are too general and include a wide range of engineering properties. Hence, additional means of

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ISSN 2229-5518

categorization are necessary to make the terms more meaningful in engineering practice. These terms are compiled to form soil classification systems.

1.10.2 Soil Classification Systems

Several classification systems were evolved by different organizations having a specific purpose as the object. A Casagrande (1948) describes the systems developed and used in highway engineering, airfield construction etc. The two classification systems, which are adopted by the US engineering agencies and the State Departments, are the Unified Soil Classification (UCCS) and the American Association of State Highway and Transportation Officials (AASHTO) system. Other countries, including India, have mostly the USCS with minor modifications.

For general engineering purposes, soils may be classified by the following systems

- Particle size classification.
- Textural classification.
- Highway Research Board (HRB) classification.
- Unified Soil Classification.
- Indian Soil Classification.

1.10.3 Atterberg limits

The Atterberg limits are a basic measure of the critical water contents of a fine-grained soil, such as its shrinkage limit, plastic limit, and liquid limit. As a dry, clayey soil takes on increasing amounts of water, it undergoes dramatic and distinct changes in behavior and consistency. Depending on the water content of the soil, it may appear in four states: solid, semi-solid, plastic and liquid. In each state, the consistency and behavior of a soil is different and consequently so are its engineering properties. Thus, the boundary between each state can be defined based on a change in the soil's behavior. The Atterberg limits can be used to distinguish between silt and clay, and it can distinguish between different types of silts and clays. These limits were created by Albert Atterberg, a Swedish chemist. They were later refined by Arthur Casagrande. These distinctions in soil are used in assessing the soils that are to have structures built on. Soils when wet retain water and some expand in volume. The amount of expansion is related to the ability of the soil to take in water and its structural make-up (the type of atoms present). These tests are mainly used on clayey or silty soils since these are the soils that expand and shrink due to moisture content. Clays and silts react with the water and thus change sizes and have varying shear strengths. Thus these tests are used widely in the preliminary stages of designing 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.

1.11 LABORATORY TESTS

1.11.1 Shrinkage limit

The shrinkage limit (SL) is the water content where further loss of moisture will not result in any more volume reduction. The test to determine the shrinkage limit is ASTM International D4943. The shrinkage limit is much less commonly used than the liquid and plastic limits. It is the minimum water content at which a soil is still in saturated condition.

1.11.2 Plastic limit

The plastic limit is determined by rolling out a thread of the fine portion of a soil on a flat, non- porous surface. The procedure is defined in ASTM standard D 4318. If the soil is plastic, this thread will retain its shape down to a very narrow diameter. The sample can then be remoulded and the test repested. As the moisture content falls due to evaporation, the thread will begin to break apart at larger diameters. The plastic limit is defined as the moisture content where the thread breaks apart at a diameter of 3.2 mm (about 1/8 inch). A soil is considered non-plastic if a thread cannot be rolled out down to 3.2 mm at any moisture.

1.11.3 Liquid limit

Figure 1.3 Casagrande Apparatus

The liquid limit (LL) is often conceptually defined as the water content at which the behavior of a clayey soil changes from plastic to liquid. Actually, clayey soil does have a very small shear strength at the liquid limit and the strength decreases as water content increases; the transition from plastic to liquid behavior occurs over a range of water contents. The precise definition of the liquid limit is based on standard test procedures described below.

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ISSN 2229-5518

The original liquid limit test of Atterberg's involved mixing a pat of clay in a round-bottomed porcelain bowl of 10–12 cm diameter. A groove was cut through the pat of clay with a spatula, and the bowl was then struck many times against the palm of one hand.

Casagrande subsequently standardized the apparatus and the procedures to make the measurement more repeatable. Soil is placed into the metal cup portion of the device and a groove is made down its center with a standardized tool of 13.5 millimetres (0.53 in) width. The cup is repeatedly dropped 10 mm onto a hard rubber base at a rate of 120 blows per minute, during which the groove closes up gradually as a result of the impact. The number of blows for the groove to close is recorded. The moisture content at which it takes 25 drops of the cup to cause the groove to close over a distance of 13.5 millimetres (0.53 in) is defined as the liquid limit. The test is normally run at several moisture contents, and the moisture content which requires 25 blows to close the groove is interpolated from the test results. The Liquid Limit test is defined by ASTM standard test method D 4318. The test method also allows running the test at one moisture content where 20 to 30 blows are required to close the groove; then a correction factor is applied to obtain the liquid limit from the moisture content. The following is when one should record the N in number of blows needed to close this 1/2-inch gap: The materials needed to do a liquid limit test are as follows

- Casagrande cup (liquid limit device)
- Grooving tool
- Soil pat before test
- Soil pat after test

Another method for measuring the liquid limit is the fall cone test. It is based on the measurement of penetration into the soil of a standardized cone of specific mass. Although the Casagrande test is widely used across North America, the fall cone test is much more prevalent in Europe due to being less dependent on the operator in determining the Liquid Limit.

Importance of liquid limit test: 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.

Derived limits: The values of these limits are used in a number of ways. There is also a close relationship between the limits and properties of a soil such as compressibility, permeability, and strength. This is thought to be very useful because as limit determination is relatively simple, it is more difficult to determine these other properties. Thus the Atterberg limits are not only used to identify the soil's classification, but it allows for the use of empirical correlations for some other engineering properties.

1.11.4 Plasticity index

The plasticity index (PI) is a measure of the plasticity of a soil. The plasticity index is the size of the range of water contents where the soil exhibits plastic properties. The PI is the difference between the liquid limit and the plastic limit (PI = LL-PL). Soils with a high PI tend to be clay, those with a lower PI tend to be silt, and those with a PI of 0 (non-plastic) tend to have little or no silt or clay.

PI and their meanings

- (0-3)- Nonplastic
- (3-15) - Slightly plastic
- (15-30) - Medium plastic
- >30 - Highly plastic

1.11.5 Liquidity index

The liquidity index (LI) is used for scaling the natural water content of a soil sample to the limits. It can be calculated as a ratio of difference between natural water content, plastic limit, and liquid limit: LI=(W-PL)/(LL-PL) where W is the natural water content. The effects of the water content on the strength of saturated remolded soils can be quantified by the use of the liquidity index, LI: When the LI is 1, remolded soil is at the liquid limit and it has an undrained shear strength of about 2 kPa. When the soil is at the plastic limit, the LI is 0 and the undrained shear strength is about 200 kPa.

1.12 STRENGTH TESTS

1.12.1 Standard Proctor Test

The Proctor compaction test is a laboratory method of experimentally determining the optimal moisture content at which a given soil type will become most dense and achieve its maximum drydensity. The term Proctor is in honor of R. R. Proctor, who in 1933 showed that the dry density of a soil for a given compactive effort depends on the amount of water the soil contains during soil compaction. His original test is most commonly referred to as the standard Proctor compaction test; later on, his test was updated to create the modified Proctor compaction test.

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1.12.2 Proctor Device

These laboratory tests generally consist of compacting soil at known moisture content into a cylindrical mold of standard dimensions using a compactive effort of controlled magnitude. The soil is usually compacted into the mold to a certain amount of equal layers, each receiving a number blows from a standard weighted hammer at a specified height. This process is then repeated for various moisture contents and the dry densities are determined for each. The graphical relationship of the dry density to moisture content is then plotted to establish the compaction curve. The maximum dry density is finally obtained from the peak point of the compaction curve and its corresponding moisture content, also known as the optimal moisture content.

Figure.1.4 Proctor mould

1.12.3 California bearing ratio test

The California bearing ratio (CBR) is a penetration test for evaluation of the mechanical strength of road subgrades and base courses. It was developed by the California Department of Transportation before World War II.

The test is performed by measuring the pressure required to penetrate a soil sample with a plunger of standard area. The measured pressure is then divided by the pressure required to achieve an equal penetration on a standard crushed rock material. The CBR test is described in ASTM Standards D1883-05 (for laboratory-prepared samples) and D4429 (for soils in place in field), and AASHTO T193. The CBR test is fully described in BS 1377 : Soils for civil engineering purposes : Part 4, Compaction related tests.

The CBR rating was developed for measuring the load-bearing capacity of soils used for building roads. The CBR can also be used for measuring the load-bearing capacity of unimproved airstrips or for soils under paved airstrips. The harder the surface, the higher the CBR rating. A CBR of 3 equates to tilled farmland, a CBR of 4.75 equates to turf or moist clay, while moist sand may have a CBR of 10. High quality crushed rock has a CBR over 80. The standard material for this test is crushed California limestone which has a value of 100.

$$\boxed{CBR = \frac{p}{p_s} \cdot 100}$$

CBR = CBR [%]

p = measured pressure for site soils [N/mm²]

p_s = pressure to achieve equal penetration on standard soil [N/mm²]

1.13. OBJECTIVES

Our main objectives of this study is as follows:

- To study the collected soil sample
- To study the performance of stabilizers including flyash, lime & gypsum.
- To study the performance of stabilized soil.

1.14.SCOPE

From this project our main scope is the various stabilizers are to be added with the collected samples in suitable proportions. The test is to be conducted to find out the properties. Then to select the suitable stabilizers.

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ISSN 2229-5518

CHAPTER – 2

REVIEW OF LITERATURES

2.1 GENERAL

In order to achieve the objective of this project, the previous work done in this topic soil stabilization method used for soil stabilization, merits and demerits of the soil stabilization methods and its application in the various fields are get reviewed. And the project is also carried out in that manner are get reviewed.

2.2 JOURNALS REVIEWED

Anitha K.R.(2013)et al studied *The effect of stabilizer RBI Grade 81 in the stabilization of kaolinite, red soil and lateritic soil.* The application of RBI Grade 81 stabilizer was studied by comparing the strength parameter of subgrade soil in terms of CBR value before and after the application of different percentages of RBI Grade 81 varying from 2% to 8%. From the test results it is observed that substantial reduction in plasticity index for soil with RBI Grade 81 viz 42 percent for kaolinite, 4 percent for red soil and 116 percent for laterite. Soaked CBR value increased for all three soils with RBI. OMC increased and MDD decreased with addition of RBI Grade 81 for red soil and kaolinite.

B.M.Patil, K.A.Patil (2013) reported about *Effect of Pond Ash and RBI Grade 81 on Properties of Subgrade Soil and Base Course of Flexible Pavement*, in the International Journal of Civil, Architectural Science and Engineering Vol:7 No:12, 2013; It deals with use of pond ash and RBI Grade 81 for improvement in CBR values of clayey soil and grade-III materials used for base course of flexible pavement. The pond ash is a thermal power plant waste and RBI Grade 81 is chemical soil stabilizer. The geotechnical properties like Maximum Dry Density (MDD), Optimum Moisture Content (OMC), Unconfined Compressive Strength (UCS), CBR value and Differential Free Swell (DFS) index of soil are tested in the laboratory for different mixes of soil, pond ash and RBI Grade 81 for different proportions. The mixes of grade-III material, pond ash and RBI Grade 81 tested for CBR test. From the study it is found that the geotechnical properties of clayey soil are improved significantly, if pond ash added with RBI Grade 81. The optimum mix recommended for subgrade is soil: pond ash: RBI Grade 81 in proportions of 76:20:4. The CBR value of grade-III base course treated with 20% pond ash and 4% RBI Grade 81 is increased by 125.93% as compared to untreated grade-III base course.

Monica Malhotra, Sanjeev Naval (2013) reported about *Stabilization of Expansive Soils Using Low Cost Materials*, in the International Journal of Engineering and Innovative Technology (IJEIT) Volume 2, the experimental results obtained in the laboratory on expansive soils treated with low cost materials (lime and fly ash) are presented. A study is carried out to check the improvements in the properties of expansive soil with fly ash and lime in varying percentages. The test results such as liquid limit, standard proctor compaction, and differential free swelling test obtained on expansive clays mixed at different proportions of lime and fly ash admixture are presented and discussed in this paper. The results show that the stabilized clay has lesser swelling potential whereas increase in optimum moisture content has been observed.

Gyanen. Takhelmayum, Savitha.A.L, Krishna Gudi (2013) reported about *Laboratory Study on Soil Stabilization Using Fly ash Mixtures* in the International Journal of Engineering Science and Innovative Technology (IJESIT) Volume 2, Issue 1, evaluate the compaction and unconfined compressive strength of stabilized black cotton soil using fine and coarse fly ash mixtures. The percentage of fine and coarse fly ash mixtures which is used in black cotton soil varied from 5 to 30. In the study concludes that with percentage addition of fine, coarse fly ash improves the strength of stabilized black cotton soil and exhibit relatively well-defined moisture-density relationship. It was found that the peak strength attained by fine fly ash mixture was 25% more when compared to coarse fly ash.

Vinay Agrawal (2011) reported about *Expansive Soil Stabilization Using Marble Dust* in the International Journal of Earth Sciences and Engineering; stabilization characteristics of Makrana marble dust are mainly due to its high lime content. Marble dust finds bulk utilization in roads, embankment and soil treatment for foundation. Particle size distribution, consistency limits, specific gravity, swelling percentage, and rate of swell were determined for the samples. Addition of marble dust decreases liquid limit, plasticity index and shrinkage index, increase plastic limit and shrinkage limit. Also experimental results shows that the swelling percentage decreases and rate of swell increases with increasing percentage of marble dust in expansive soils. Specimens have been cured for 7 and 28 days. The rate of swelling and swelling percentage of the stabilized specimens was affected by curing in a positive direction such that effectiveness of the stabilizer increases.

Dr. Robert M. Brooks (2009) reported about *soil stabilization with flyash and rice husk ash* in the International Journal of Research and Reviews in Applied Sciences Volume 1, Issue 3, Stress strain behavior of unconfined compressive strength showed that failure stress and strains increased by 106% and 50% respectively when the flyash content was increased from

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0 to 25%. When the RHA content was increased from 0 to 12%, Unconfined Compressive Stress increased by 97% while CBR improved by 47%. Therefore, an RHA content of 12% and a flyash content of 25% are recommended for strengthening the expansive subgrade soil. A flyash content of 15% is recommended for blending into RHA for forming a swell reduction layer because of its satisfactory performance in the laboratory tests.

Brooks (2009) investigated the *soil stabilization with flayash and rice husk ash*. This study reports; stress strain behavior of unconfined compressive strength showed that failure stress and strains increased by 106% and 50% respectively when the flyash content was increased from 0 to 25%. When the rice husk ash (RHA) content was increased from 0 to 12%, Unconfined Compressive Stress increased by 97% while California Bearing Ratio (CBR) improved by 47%. Therefore, an RHA content of 12% and a flyash content of 25% are recommended for strengthening the expansive subgrade soil. A flyash content of 15% is recommended for blending into RHA for forming a swell reduction layer because of its satisfactory performance in the laboratory tests.

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CHAPTER–3

EXPERIMENTAL METHODS

3.1 SELECTION OF STUDY AREA

For this study the area selected railway broad gauge line near by Thirukkuvalai in Thiruvarur District and soil settlement for Sripuranthan in Ariyalur District. This study area is selected in order to change the poor conditions of soil in the area.
Key map of our study area is given below

Figure 3.1 Thirukkuvalai location

Figure 3.2 Sripuranthan location

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3.2 SAMPLING

3.2.1 Soil sampling

Soil sampling is done in that study area. The soil samples are collected at the location of Thirukkuvalai and Sripuranthan. Totally two problematic soil samples are collected at two places by open bore well sampling method. The soil samples are named as below.

In Thirukkuvalai soil sample - Sample A
In Sripuranthan soil sample - Sample B

Sample A

Sample B

Figure 3.3 Soil Samples in the laboratory

3.2.2 Stabilizer sampling

Flyash is collected from Neyveli Lignite Corporation. In thermal power plants it is mostly available. Gypsum and lime is collected from corresponding market.

Figure 3.4 Lime, Fly ash & Gypsum in the laboratory

3.4 PROPERTIES OF SOIL SAMPLE A

3.4.1 Moisture content for soil sample A

Table 3.1 Moisture content for soil sample A

S.no	Observation	Trial-1	Trial-2
1	Weight of the container ,W1 (g)	130	135
2	Weight of the container and wet soil, W2 (g)	630	635
3	Weight of the container and dry soil, W3 (g)	536	545
4	Weight of the dry soil ,W3-W1(g)	500	500
5	Weight of water, W2-W3 (g)	94	90
6	Water content in % W =(w_2-w_3)/ (w_3-w_1) x 100%	18.8	18
Average water content in % = 18.4			

3.4.2 Differential free swell index for soil sample A

Table 3.2 Differential free swell index for soil sample A

S.no	Observation	Trail
1	Volume of the soil in kerosene after swelling,V1 ml	21
2	Volume of soil in water after swelling,V2 ml	32
3	The free swell index of the soil %)	52.38 %
Degree of expansiveness of soil is very high , since the degree of free swell index is >50		

3.4.3 Plastic limit for soil sample A

Table 3.3 Plastic limit for soil sample A

S.no	Description	Sample A
1	Weight of can ,W1	36
2	Weight of can + wet soil ,W2	46
3	Weight of can + dry soil , W3	43
4	Weight of water, (W2-W3)	3
5	Weight of dry soil, (W3-W1)	7
6	Moisture content, (W2-W3)/ (W3-W1) %	42.857 %

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3.4.4 Liquid limit for soil sample A

Table 3.4 Liquid limit for soil sample A

Liquid limit – sample A		
Wt of soil	% of water	No of blows
100	50	180
100	52	154
100	54	121
100	56	98
100	58	61
100	60	45
100	62	27
100	64	18

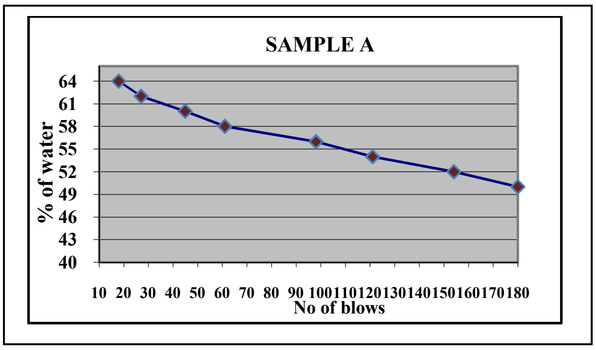

Figure 3.5 Liquid Limit For Soil Sample A

3.4.5 Plasticity index for soil sample A

Table 3.5 Plasticity index for soil sample A

Sample	Liquid limit (w_l) %	Plastic limit (w_P) %	Plasticity index (i_P)(w_l- w_P) %
Sample A	63	42.857	20.143

3.4.6 Shrinkage limit for soil sample A

Table 3.6 Shrinkage Limit for soil sample A

S.no	Determination no.	Sample A
1	Wt. of container in gm, W_1	183
2	Wt. of container + wet soil pat in gm, W_2	229
3	Wt. of container + dry soil pat in gm, W_3	213
4	Wt. of wet soil, $W4= W_2 - W_1$	46
5	Wt. of dry soil, $W5= W_3 – W_1$	30
6	Wt. of container + mercury filling dish, W_6 in gm	588
7	Wt of mercury filling dish $W7= W_6 – W_1$	405
8	Wt. of dish + mercury after displayed by dry pat W8 gms	250
9	Wt. Of mercury displayed by dry pat, W9=W6-W8	338
10	Volume of wet soil pat (V1=W7/13.6), in cm^3	29.8
11	Volume of dry soil pat (V2=W7/13.6), in cm^3	24.9
12	Shrinkage limit (W_S) = [((W4 -W5) - (V_1-V_o)) / W5] x 100 %	37
13	Shrinkage ratio (R) = W5 / V2	1.205
14	Volumetric shrinkage VS= [((W4 -W5)/ W5)-SL] X SR	0.1968

3.4.7 Standard proctor compaction test for soil sample A

Table 3.7 Standard proctor compaction test for soil sample A

Standard proctor compaction test - sample A					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ϱ) g/cc	Dry Density($ϱ_d$) g/cc
6	5907	4505	1402	1.383694	1.305371
8	5986	4505	1481	1.461662	1.353391
10	6036	4505	1531	1.511009	1.373645
12	6114	4505	1609	1.587991	1.417849
14	6202	4505	1697	1.674842	1.46916
16	6234	4505	1729	1.706424	1.471055
18	6241	4505	1736	1.713333	1.451977
20	6252	4505	1747	1.724189	1.436824
22	6243	4505	1738	1.715306	1.405989
Maximum dry density					1.47023
Optimum moisture content					15%

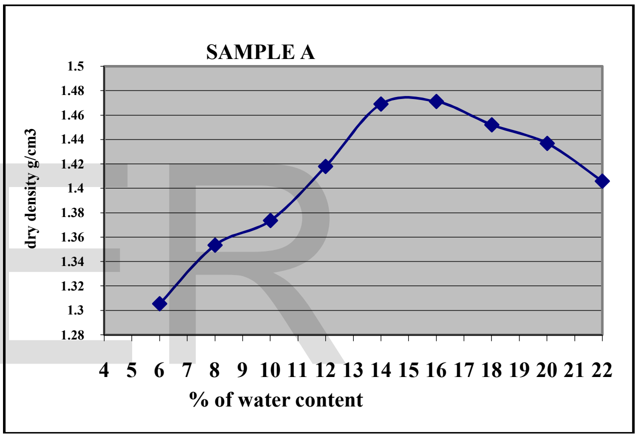

Figure 3.6 Standard Proctor Compaction Test For Soil Sample A

3.4.8 Unconfined compressive strength test for soil sample A

Table 3.8 Unconfined compressive strength test for sample A first specimen

Unconfined compressive strength test- sample A- First specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	1.4	11.074	11.3354	12.13367	0.912667
100	1	0.1315789	3.6	28.476	11.3354	13.05288	2.181587
150	1.5	0.1973684	6.2	49.042	11.3354	14.12279	3.472542
200	2	0.2631579	8.4	66.444	11.3354	15.38376	4.319101
250	2.5	0.3289474	10	79.1	11.3354	16.89197	4.682699
300	3	0.3947368	10.8	85.428	11.3354	18.72805	4.561499
350	3.5	0.4605263	10.6	83.846	11.3354	21.01196	3.990394

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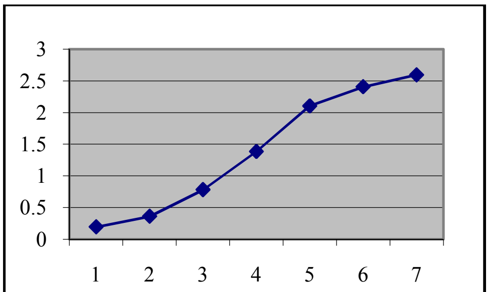

Figure 3.7 Unconfined Compressive Strength Test For Soil Sample A- First specimen

Table 3.9 Unconfined compressive strength test for sample A second specimen

Unconfined compressive strength test- Sample A - Second specimen							
Reading	Dl	E(strain)	Proving gauge reading	Load	Ao	Corrected area	Stress
50	0.5	0.0657895	0.2	1.582	11.3354	12.13367	0.130381
100	1	0.1315789	1.7	13.447	11.3354	13.05288	1.030194
150	1.5	0.1973684	4	31.64	11.3354	14.12279	2.24035
200	2	0.2631579	7.4	58.534	11.3354	15.38376	3.804922
250	2.5	0.3289474	10.2	80.682	11.3354	16.89197	4.776353
300	3	0.3947368	12	94.92	11.3354	18.72805	5.068333
350	3.5	0.4605263	13	102.83	11.3354	21.01196	4.893879
400	4	0.5263158	12.8	101.248	11.3354	23.93029	4.230956
450	4.5	0.5921053	12.4	98.084	11.3354	27.79001	3.529469

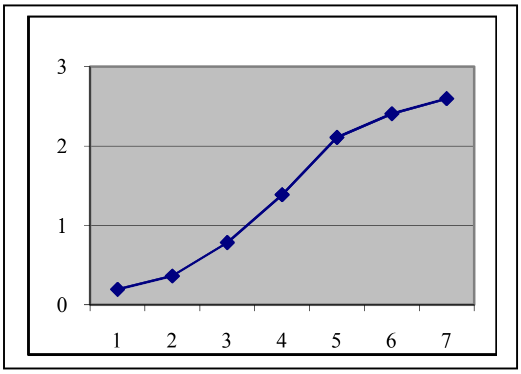

Figure 3.8 Unconfined Compressive Strength Test For Soil Sample A- second specimen

3.5 PROPERTIES OF SOIL SAMPLE B

3.5.1 Moisture Content for soil sample B

Table 3.10 Moisture Content for soil sample B

S.no	Observation	Trial-1	Trial-2
1	Weight of the container ,W1 (g)	141	135
2	Weight of the container and wet soil, W2 (g)	641	635
3	Weight of the container and dry soil, W3 (g)	543	532
4	Weight of the dry soil ,W3-W1 (g)	402	397
5	Weight of water, W2-W3 (g)	98	103
6	Water content in % W =(w_2-w_3)/ (w_3-w_1) x 100%	24.3	25.9
	Average water content in % = 25.1		

3.5.2 Differential free swell index for soil sample B

Table 3.11 Differential free swell index for soil sampleB

S.no	Observation	Trail
1	Volume of the soil in kerosene after swelling, V1 ml	20
2	Volume of soil in water after swelling,V2 ml	24
3	The free swell index of the soil (%) [(V1-V2)/V1] X 100 %	20 %
	Degree of expansiveness of soil is Moderate , since the degree of free swell index is 20	

3.5.3 Plastic limit for soil sample B

Table 3.12 plastic limit for soil sample B

S.no	Description	Sample B
1	Weight of can ,W1	36
2	Weight of can + wet soil ,W2	46
3	Weight of can + dry soil , W3	44
4	Weight of water, (W2-W3)	2
5	Weight of dry soil, (W3-W1)	8
6	Moisture content, (W2-W3)/ (W3-W1) %	25 %

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3.5.4 Liquid limit for soil sample B

Table 3.13 Liquid limit for soil sample B

Liquid limit – Sample B		
Wt of soil	% of water	No of blows
100	36	80
100	38	65
100	40	49
100	42	36
100	44	22

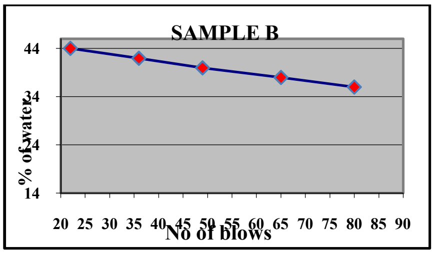

Figure 3.9 Liquid limit for soil sample A

3.5.5 Plasticity index for soil sample B

Table 3.14 Plasticity index for soil sample B

Sample	Liquid limit (w_l) %	Plastic limit (w_p) %	Plasticity index (i_p) %
Sample B	42	25	17

3.5.6 Shrinkage limit for soil sample B

Table 3.15 Shrinkage limit for soil sample B

S.no	Determination no.	Sample B
1	Wt. of container in gm, W_1	154
2	Wt. of container + wet soil pat in gm, W_2	208
3	Wt. of container + dry soil pat in gm, W_3	192
4	Wt. of wet soil, $W4 = W_2 - W_1$	54
5	Wt. of dry soil, $W5 = W_3 - W_1$	38
6	Wt. of container + mercury filling dish, W_6 in gm	578.5
7	Wt of mercury filling dish $W7 = W_6 - W_1$	424.5
8	Wt. of dish + mercury after displayed by dry pat W8 gms	298.5
9	Wt. Of mercury displayed by dry pat, W9=W6-W8	280
10	Volume of wet soil pat (V1=W7/13.6), in cm^3	31.21
11	Volume of dry soil pat (V2=W9/13.6), in cm^3	20.59
12	Shrinkage limit (W_s) = $[((W4 - W5) - (V_1 - V_o)) / W5] \times 100$ %	14.2
13	Shrinkage ratio (R) = W5 / V2	1.85
14	Volumetric shrinkage VS= [((W4 -W5)/ W5)-SL] X SR	0.52

3.5.7 Standard proctor compaction test for sample B

Table 3.16 Standard proctor compaction test for soil sample B

Standard proctor compaction test - Sample B					
Water content %	Wt of mou	Empty wt. of	Wt of compacted soil	Wet Density(ρ) g/cc	Dry Density(ρ_d) g/cc
6	6178	4446	1732	1.709385	1.612627
8	6253	4446	1807	1.783406	1.651301
10	6402	4446	1956	1.93046	1.754964
12	6534	4446	2088	2.060736	1.839943
14	6560	4446	2114	2.086397	1.830173
16	6504	4446	2058	2.031128	1.750973

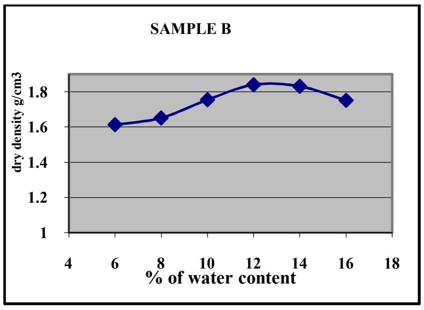

Figure 3.10 Standard proctor compaction for soil sample B

3.5.8 CBR Test for soil sample B

Table 3.17 CBR Test for soil sample B

Sl No.	Un soaked					Soaked				
	Penetration		Load			Penetration		Load		
	Div	mm	Div	N	kg	Div	mm	Div	N	kg
1	50	0.5	26	735.94	73.59	50	0.5	7	198.14	19.81
2	100	1	34	962.39	96.23	100	1	16	452.89	45.28
3	150	1.5	40	1132.22	113.22	150	1.5	20	566.11	56.61
4	200	2	44	1245.44	124.54	200	2	22	622.72	62.27
5	250	2.5	50	1415.28	141.52	250	2.5	23	651.03	65.10
6	300	3	64	1811.55	181.15	300	3	24	679.33	67.93
7	350	3.5	70	1981.39	198.13	350	3.5	25	707.64	70.76
8	400	4	82	2321.05	232.10	400	4	26	735.94	73.59
9	450	4.5	90	2547.50	254.74	450	4.5	28	792.55	79.25
10	500	5	102	2887.16	288.71	500	5	30	849.17	84.91
11	550	5.5	104	2943.77	294.37	550	5.5	31	877.47	87.74
12	600	6	108	3056.99	305.69	600	6	33	934.08	93.40
13	650	6.5	116	3283.44	328.34	650	6.5	34	962.39	96.23
14	700	7	118	3340.05	334.00	700	7	34	962.39	96.23
15	750	7.5	120	3396.66	339.66	750	7.5	35	990.69	99.06
16	800	8	122	3453.27	345.32	800	8	35	990.69	99.06
17	850	8.5	124	3509.88	350.98	850	8.5	36	1019.00	101.89
18	900	9	126	3566.49	356.64	900	9	36	1019.00	101.89
19	950	9.5	128	3623.10	362.31	950	9.5	37	1047.30	104.73
20	1000	10	130	3679.72	367.97	1000	10	37	1047.30	104.73

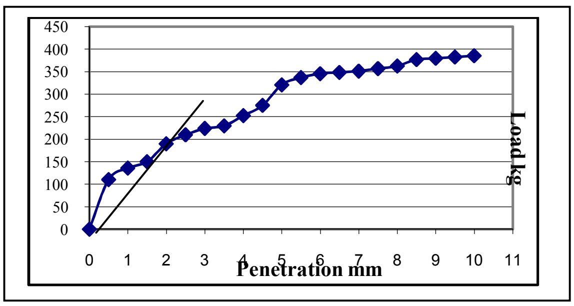

Figure 3.11 CBR test graph for soil sample B with Un soaked

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CBR

2.5 mm	141	10.33724
5 mm	289	14.15973

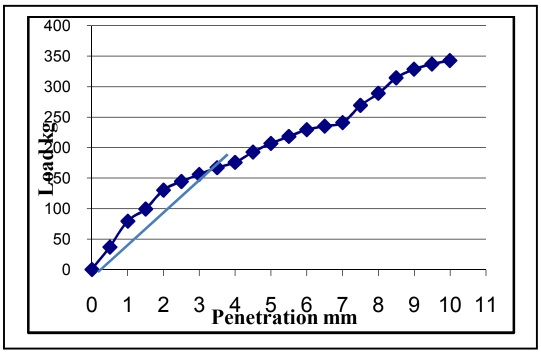


Figure 3.12 CBR Test graph for soil sample B with soaked

CBR

2.5 mm	57	4.160584
5 mm	84	4.087591

3.6 SOIL SAMPLE A WITH FLYASH IN VARIOUS PROPORTIONS

3.6.1 Differential free swell index for soil sample A with flyash in various proportions

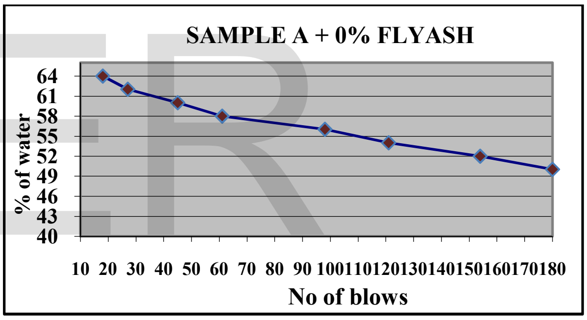
Table 3.18 Differential free swell index for sample A with flyash in various proportions

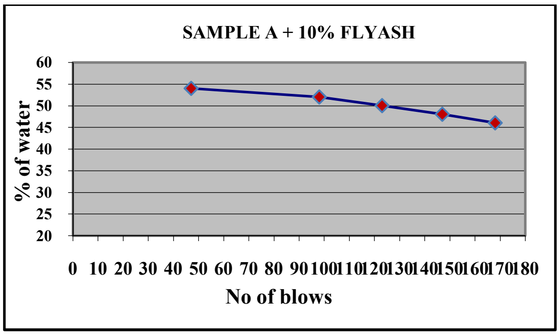
S. no	Observation	0% Flyash	10% Flyash	20% Flyash	30% Flyash
1	Volume of the soil in kerosene after swelling, V1ml	21	21	21	21
2	Volume of soil in water after swelling,V2 ml	32	32	33	34
3	The free swell index of the soil (%) [(V1-V2)/V1] X 100 %	52.38 %	52.38 %	57.14 %	61.9 %
Degree of expansiveness of soil: Since, The Free swell index is greater than 50.		very high	very high	very high	very high

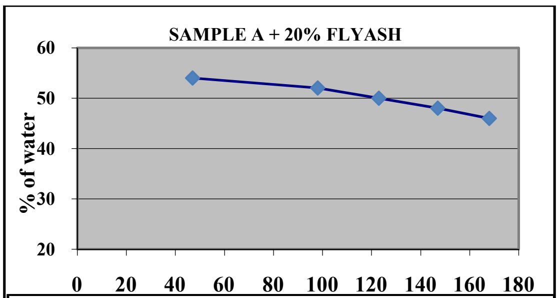
3.6.2 Liquid limit for sample A with flyash in various proportions

Table 3.19 Liquid limit for sample A with flyash in various proportions

Samples	Observations								Liquid limit (from graph)
0% Flyash	No of Blows	180	154	121	98	61	45	27	62.4 %
	Water Content	50	52	54	56	58	60	62	
	Wt. Of soil	100	100	100	100	100	100	100	
10% Flyash	No of Blows	141	102	74	43	21	-	-	48 %
	Water Content	40	42	44	46	48	-	-	
	Wt. Of soil	100	100	100	100	100	-	-	
20% Flyash	No of Blows	183	132	77	46	20	-	-	47 %
	Water Content	40	42	44	46	48	-	-	
	Wt. Of soil	100	100	100	100	100	-	-	
30% Flyash	No of Blows	168	147	123	98	49	38	17	57 %
	Water Content	46	48	50	52	54	56	58	
	Wt. Of soil	100	100	100	100	100	100	100	







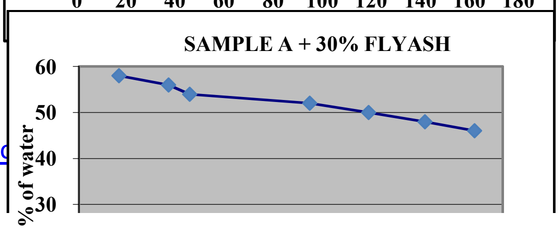


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Figure 3.13 Liquid limit for soil sample A with flyash in various proportions

3.6.3 Plastic limit for soil sample A with flyash in various proportions

Table 3.20 Plastic limit for soil sample A with flyash in various proportions

S.No	Description	0% flyash	10% flyash	20% flyash	30% flyash
1	Weight of can ,W1	36	36	23	36
2	Weight of can + wet soil ,W2	46	46	35	47
3	Weight of can + dry soil , W3	43	45	33	45
4	Weight of water, (W2-W3)	3	3	2	2
5	Weight of dry soil, (W3-W1)	7	9	10	9
6	Moisture content, (W2-W3)/ (W3-W1) %	42.857 %	33.33	20	22.22

3.6.4 Plasticity index for soil sample A with flyash in various proportions

Table 3.21 Plasticity index for soil sample A with flyash in various proportions

sample	Liquid limit (w_l) %	Plastic limit (w_P) %	Plasticity index (i_P) %
0% flyash	62.4	42.857	19.543
10% flyash	48	33.33	14.67
20% flyash	47	20	27
30% flyash	57	22.22	34.78

3.6.5 Shrinkage limit for sample A with flyash in various proportions

Table 3.22 Shrinkage limit for sample A with flyash in various proportions

S.no	Determination no.	0% flyash	10% flyash	20% flyash	30% flyash
1	Wt. of container in gm,W_1	183	183	157	154
2	Wt. of container + wet soil pat in gm,W_2	229	228	205	215
3	Wt. of container + dry soil pat in gm,W_3	213	216	192	199
4	Wt. of wet soil, W4= W_2 - W_1	46	45	48	61
5	Wt. of dry soil, W5= W_3 – W_1	30	33	35	45
6	Wt. of container + mercury filling dish, W_6 in gm	588	587	525	584
7	Wt of mercury filling dish W7= W_6 – W_1	405	404	368	430
8	Wt. of dish + mercury after displayed by dry pat W8 gms	250	307	253	252
9	Wt. Of mercury displayed by dry pat, W9=W6-W8	338	280	272	332
10	Volume of wet soil pat (V1=W7/13.6), in cm^3	29.8	29.7	27.06	31.6
11	Volume of dry soil pat (V2=W9/13.6), in cm^3	24.9	20.58	20	24.4
12	Shrinkage limit (W_S) = [((W4 - W5) - (V_1-V_o)) / W5] x 100 %	37	8.72	16.9	19.56
13	Shrinkage ratio (R) = W5 / V2	1.205	1.331	1.75	1.844
14	Volumetric shrinkage VS= [((W4 -W5)/ W5)-SL] X SR	0.1968	0.4217	0.354	0.1599

3.6.6 Standard proctor compaction test for sample A with flyash in various proportions

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Table 3.23 Standard proctor compaction test for sample A with 0% flyash

Standard proctor compaction test - Sample A + 0% flysh					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ϱ) g/cc	Dry Density (ϱd) g/cc
6	5907	4505	1402	1.383694	1.305371
8	5986	4505	1481	1.461662	1.353391
10	6036	4505	1531	1.511009	1.373645
12	6114	4505	1609	1.587991	1.417849
14	6202	4505	1697	1.674842	1.46916
16	6234	4505	1729	1.706424	1.471055
18	6241	4505	1736	1.713333	1.451977
20	6252	4505	1747	1.724189	1.436824
22	6243	4505	1738	1.715306	1.405989

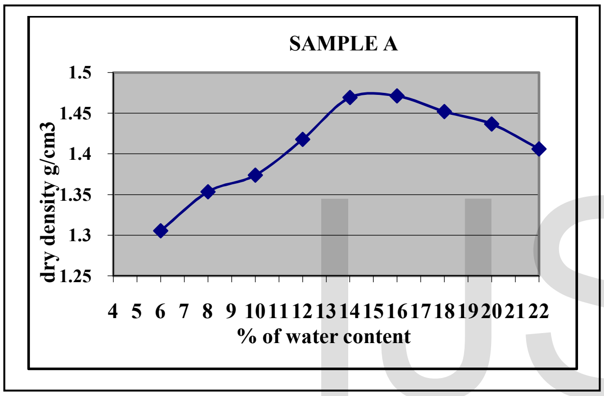

Figure 3.13 Standard proctor compaction test for soil sample A with 0% flyash

Table 3.24 Standard proctor compaction test for sample A with 10% flyash

Standard Proctor Compaction Test - Sample A (10 % Flyash)					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ϱ) g/cc	Dry Density(ϱd) g/cc
6	6024	4505	1519	1.499166	1.414308
8	6067	4505	1562	1.541605	1.427412
10	6112	4505	1607	1.586017	1.441834
12	6148	4505	1643	1.621547	1.44781
14	6184	4505	1679	1.657077	1.453576
16	6210	4505	1705	1.682737	1.450636
18	6192	4505	1687	1.664972	1.410994

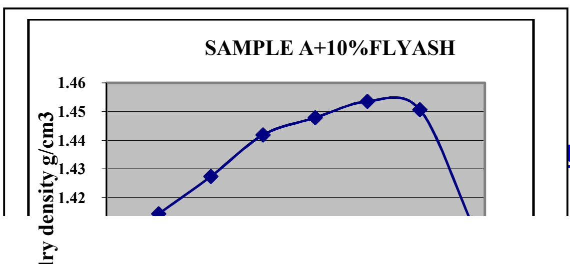

Figure 3.14 Standard proctor compaction test for soil sample A with 10% flyash

Table 3.25 Standard proctor compaction test for sample A with 20% flyash

Standard Proctor Compaction Test - Sample A (20 % Flyash)					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ϱ) g/cc	Dry Density(ϱd) g/cc
6	6105	4505	1600	1.579108	1.489725
8	6143	4505	1638	1.616612	1.496863
10	6187	4505	1682	1.660038	1.509125
12	6227	4505	1722	1.699515	1.517424
14	6267	4505	1762	1.738993	1.525433
16	6319	4505	1814	1.790314	1.543374
18	6342	4505	1837	1.813014	1.536452
20	6320	4505	1815	1.791301	1.492751

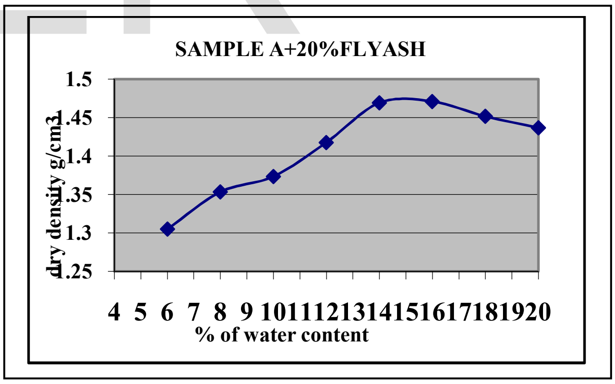

Figure 3.15 Standard proctor compaction test for soil sample A with 20% flyash

Table 3.26 Standard proctor compaction test for sample A with 30% flyash

Standard Proctor Compaction Test - Sample A (30

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% Flyash)					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ρ) g/cc	Dry Density (ρ_d) g/cc
6	6034	4505	1529	1.509035	1.423618
8	6134	4505	1629	1.60773	1.488639
10	6176	4505	1671	1.649181	1.499256
12	6243	4505	1738	1.715306	1.531524
14	6278	4505	1773	1.749849	1.534956
16	6313	4505	1808	1.784392	1.538269
18	6361	4505	1856	1.831766	1.552344
20	6363	4505	1858	1.83374	1.528116
22	6349	4505	1844	1.819922	1.49174

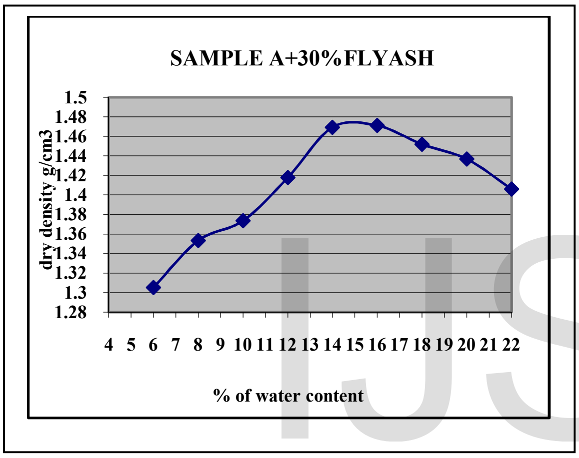

Figure 3.16 Standard proctor compaction test for soil sample A with 30% flyash

Table 3.27 Maximum dry density and OMC value for sample A

Sample A and flyash Proportions	MDD (g/cm³)	OMC %
Sample A+ 0% flyash	1.464	16
Sample A+ 10% flyash	1.455	15
Sample A+ 20% flyash	1.68	16
Sample A+ 30% flyash	1.47	16

3.6.7 Unconfined Compressive Strength test value for sample A with flyash in various proportion

Table 3.28 Unconfined Compressive Strength test value for sample A with 0%Flyash

Unconfined compressive strength test- Sample A + 0% Flyash-First specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	1.4	11.074	11.3354	12.13367	0.912667
100	1	0.1315789	3.6	28.476	11.3354	13.05288	2.181587
150	1.5	0.1973684	6.2	49.042	11.3354	14.12279	3.472542
200	2	0.2631579	8.4	66.444	11.3354	15.38376	4.319101
250	2.5	0.3289474	10	79.1	11.3354	16.89197	4.682699
300	3	0.3947368	10.8	85.428	11.3354	18.72805	4.561499
350	3.5	0.4605263	10.6	83.846	11.3354	21.01196	3.990394

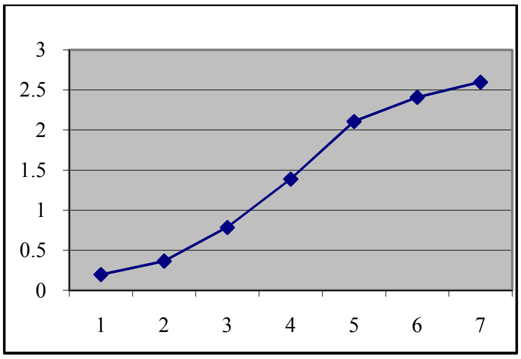

Figure 3.17 Unconfined Compressive Strength test value for sample A with 0%Flyash-1

Table 3.29 Unconfined Compressive Strength test value for sample A with 0%Flyash

Unconfined compressive strength test- Sample A + 0% Flyash-Second specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	0.2	1.582	11.3354	12.13367	0.130381
100	1	0.1315789	1.7	13.447	11.3354	13.05288	1.030194
150	1.5	0.1973684	4	31.64	11.3354	14.12279	2.24035
200	2	0.2631579	7.4	58.534	11.3354	15.38376	3.804922
250	2.5	0.3289474	10.2	80.682	11.3354	16.89197	4.776353
300	3	0.3947368	12	94.92	11.3354	18.72805	5.068333
350	3.5	0.4605263	13	102.83	11.3354	21.01196	4.893879
400	4	0.5263158	12.8	101.248	11.3354	23.93029	4.230956
450	4.5	0.5921053	12.4	98.084	11.3354	27.79001	3.529469

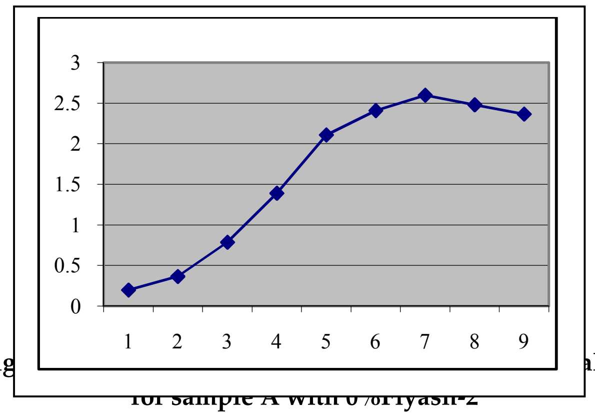

Fig ... alue for sample A with 0%Flyash-2

Table 3.30 Unconfined Compressive Strength test value for sample A with 10%Flyash

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Unconfined compressive strength test- Sample A (10% Flyash) – First specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	1.4	11.074	11.3354	12.13367	0.912667
100	1	0.1315789	4.2	33.222	11.3354	13.05288	2.545184
150	1.5	0.1973684	6.6	52.206	11.3354	14.12279	3.696577
200	2	0.2631579	8.4	66.444	11.3354	15.38376	4.319101
250	2.5	0.3289474	9.8	77.518	11.3354	16.89197	4.589045
300	3	0.3947368	10.6	83.846	11.3354	18.72805	4.477027
350	3.5	0.4605263	10.8	85.428	11.3354	21.01196	4.065684
400	4	0.5263158	11	87.01	11.3354	23.93029	3.635978
450	4.5	0.5921053	10.8	85.428	11.3354	27.79001	3.074054

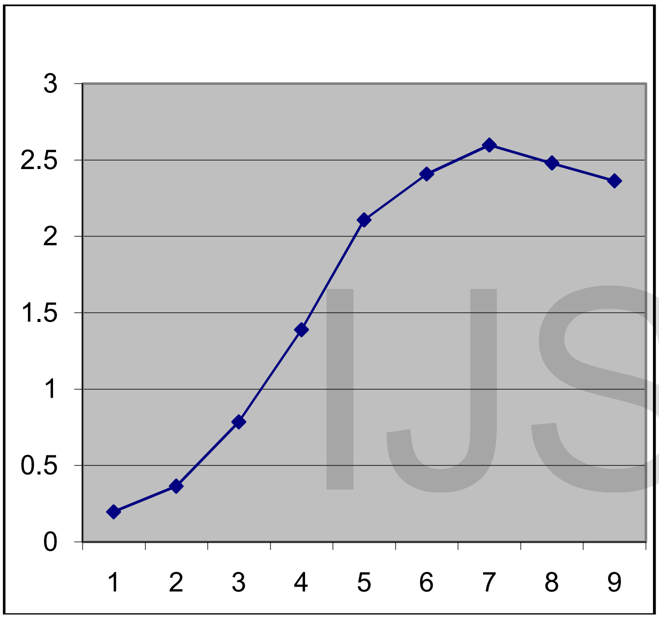

Figure 3.19 Unconfined Compressive Strength test value for sample A with 10%Flyash-2

Table 3.31 Unconfined Compressive Strength test value for sample A with 10%Flyash

Unconfined compressive strength test- Sample A (10% Flyash) – Second specimen							
reading	dl	E(strain)	proving gauge	Load	Ao	Corrected Area	Stress

			reading				
50	0.5	0.0657895	1.2	9.492	11.3354	12.13367	0.782286
100	1	0.1315789	4	31.64	11.3354	13.05288	2.423985
150	1.5	0.1973684	6.2	49.042	11.3354	14.12279	3.472542
200	2	0.2631579	8.1	64.071	11.3354	15.38376	4.164847
250	2.5	0.3289474	9.8	77.518	11.3354	16.89197	4.589045
300	3	0.3947368	10.4	82.264	11.3354	18.72805	4.392555
350	3.5	0.4605263	10.6	83.846	11.3354	21.01196	3.990394
400	4	0.5263158	10.9	86.219	11.3354	23.93029	3.602923
450	4.5	0.5921053	10.6	83.846	11.3354	27.79001	3.017127

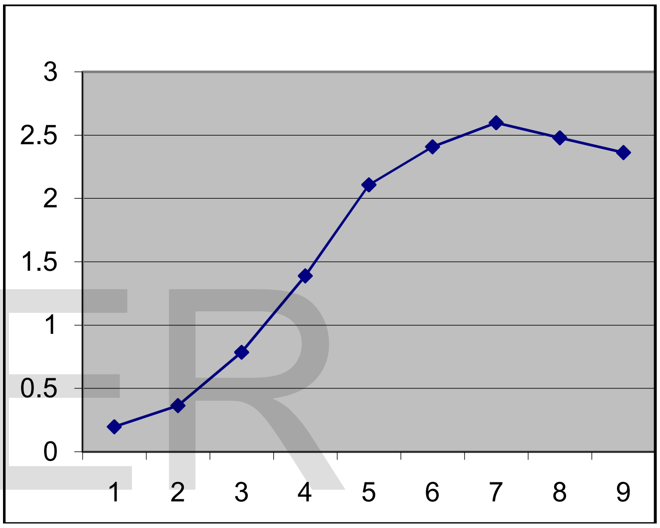

Figure 3.20 Unconfined Compressive Strength test value for sample A with 10%Flyash-2

Table 3.32 Unconfined Compressive Strength test value for sample A with 20%Flyash

Unconfined compressive strength test- Sample A (20% flyash) – First specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress

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50	0.5	0.0657895	2.2	17.402	11.3354	12.13367	1.434191
100	1	0.1315789	4.4	34.804	11.3354	13.05288	2.666384
150	1.5	0.1973684	6.4	50.624	11.3354	14.12279	3.58456
200	2	0.2631579	7.6	60.116	11.3354	15.38376	3.907758
250	2.5	0.3289474	8	63.28	11.3354	16.89197	3.746159
300	3	0.3947368	7.8	61.698	11.3354	18.72805	3.294416
350	3.5	0.4605263	7.6	60.116	11.3354	21.01196	2.861037

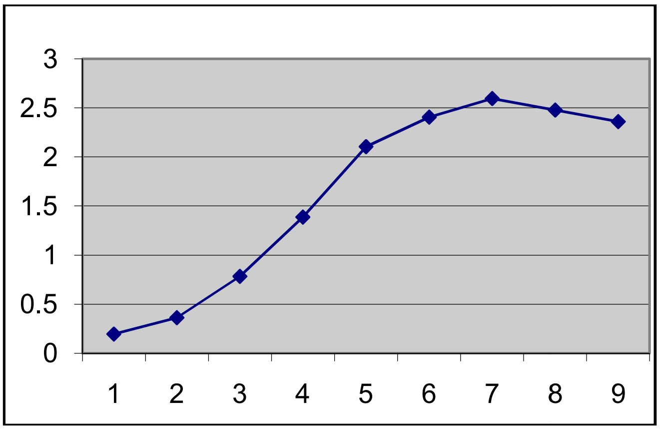

Figure 3.21 Unconfined Compressive Strength test value for sample A with 0%Flyash-1

Table 3.33 Unconfined Compressive Strength test value for sample A with 20%Flyash

Unconfined compressive strength test- Sample A (20% flyash) – Second specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	2.6	20.566	11.3354	12.13367	1.694953
100	1	0.1315789	5.2	41.132	11.3354	13.05288	3.151181
150	1.5	0.1973684	6.8	53.788	11.3354	14.12279	3.808595
200	2	0.2631579	7.2	56.952	11.3354	15.38376	3.702087
250	2.5	0.3289474	7.8	61.698	11.3354	16.89197	3.652505
300	3	0.3947368	7.2	56.952	11.3354	18.72805	3.041
350	3.5	0.4605263	6.8	53.788	11.3354	21.01196	2.559875

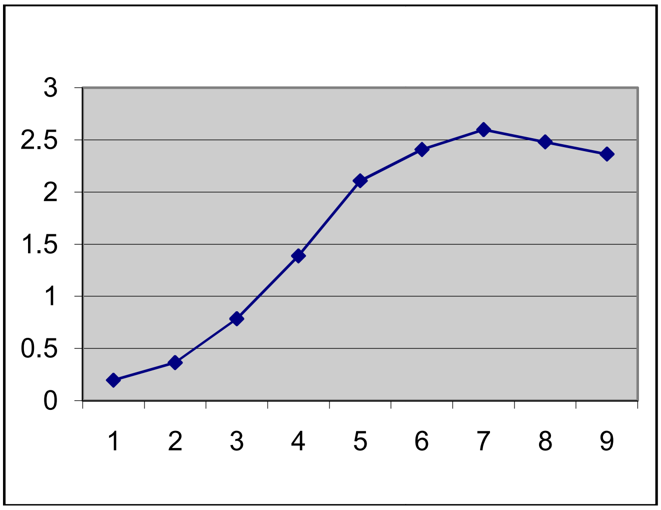

Figure 3.22 Unconfined Compressive Strength test value for sample A with 0%Flyash-1

Table 3.34 Unconfined Compressive Strength test value for sample A with 30%Flyash

Unconfined compressive strength test- Sample A (30% flyash) – First specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	0.2	1.582	11.3354	12.13367	0.130381
100	1	0.1315789	0.4	3.164	11.3354	13.05288	0.242399
150	1.5	0.1973684	1.2	9.492	11.3354	14.12279	0.672105
200	2	0.2631579	2.6	20.566	11.3354	15.38376	1.336865
250	2.5	0.3289474	4.2	33.222	11.3354	16.89197	1.966733
300	3	0.3947368	5.6	44.296	11.3354	18.72805	2.365222
350	3.5	0.4605263	6.6	52.206	11.3354	21.01196	2.484585
400	4	0.5263158	7.6	60.116	11.3354	23.93029	2.51213
450	4.5	0.5921053	8.4	66.444	11.3354	27.79001	2.390931
500	5	0.6578947	9	71.19	11.3354	33.13425	2.148532
550	5.5	0.7236842	9.2	72.772	11.3354	41.02335	1.773916
600	6	0.7894737	9.4	74.354	11.3354	53.84315	1.380937
650	6.5	0.8552632	9.6	75.936	11.3354	78.31731	0.969594
700	7	0.9210526	9.4	74.354	11.3354	143.5817	0.517851
750	7.5	0.9868421	9.4	74.354	11.3354	861.4904	0.086309

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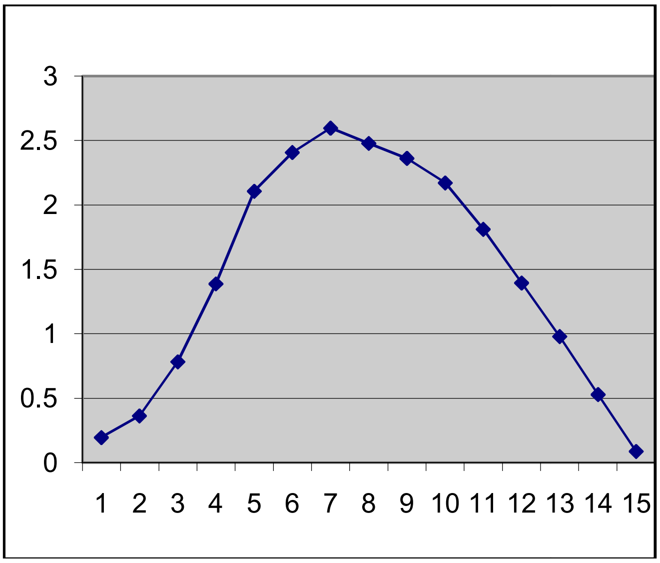

Figure 3.23 Unconfined Compressive Strength test value for sample A with 0%Flyash-1

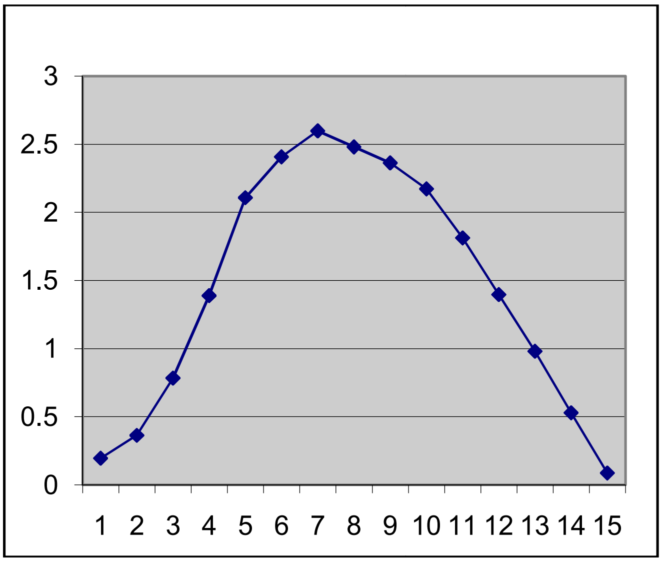

Figure 3.24 Unconfined Compressive Strength test value for sample A with 0%Flyash-2

Table 3.35 Unconfined Compressive Strength test value for sample A with 30%Flyash

Unconfined compressive strength test- Sample A (30% flyash) – Second specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	0.3	2.373	11.3354	12.13367	0.195572
100	1	0.1315789	0.6	4.746	11.3354	13.05288	0.363598
150	1.5	0.1973684	1.4	11.074	11.3354	14.12279	0.784122
200	2	0.2631579	2.7	21.357	11.3354	15.38376	1.388282
250	2.5	0.3289474	4.5	35.595	11.3354	16.89197	2.107214
300	3	0.3947368	5.7	45.087	11.3354	18.72805	2.407458
350	3.5	0.4605263	6.9	54.579	11.3354	21.01196	2.597521
400	4	0.5263158	7.5	59.325	11.3354	23.93029	2.479076
450	4.5	0.5921053	8.3	65.653	11.3354	27.79001	2.362467
500	5	0.6578947	9.1	71.981	11.3354	33.13425	2.172405
550	5.5	0.7236842	9.4	74.354	11.3354	41.02335	1.81248
600	6	0.7894737	9.5	75.145	11.3354	53.84315	1.395628
650	6.5	0.8552632	9.7	76.727	11.3354	78.31731	0.979694
700	7	0.9210526	9.6	75.936	11.3354	143.5817	0.52887
750	7.5	0.9868421	9.5	75.145	11.3354	861.4904	0.087227

3.7 SOIL SAMPLE B WITH FLYASH IN VARIOUS PROPORTIONS

3.7.1 Liquid limit for sample B with flyash in various proportions

Table 3.36 Liquid limit for sample B with flyash in various proportions

Samples	Observations								Liquid limit (from graph)
0% flyash	No of Blows	80	65	49	36	22	-	-	42%
	Water Content	36	38	40	42	44	-	-	
	Wt. of soil	100	100	100	100	100	-	-	
10 % flyash	No of Blows	174	164	159	121	80	29	13	41.45 %
	Water Content	30	32	34	36	38	40	42	
	Wt. of soil	100	100	100	100	100	100	100	
20 % flyash	No of Blows	152	124	101	59	19	-	-	35.73 %
	Water Content	28	30	32	34	36	-	-	
	Wt. of soil	100	100	100	100	100	-	-	
30 % flyash	No of Blows	135	127	89	55	18	-	-	21.79 %
	Water Content	14	16	18	20	22	-	-	
	Wt. of soil	100	100	100	100	100	-	-	

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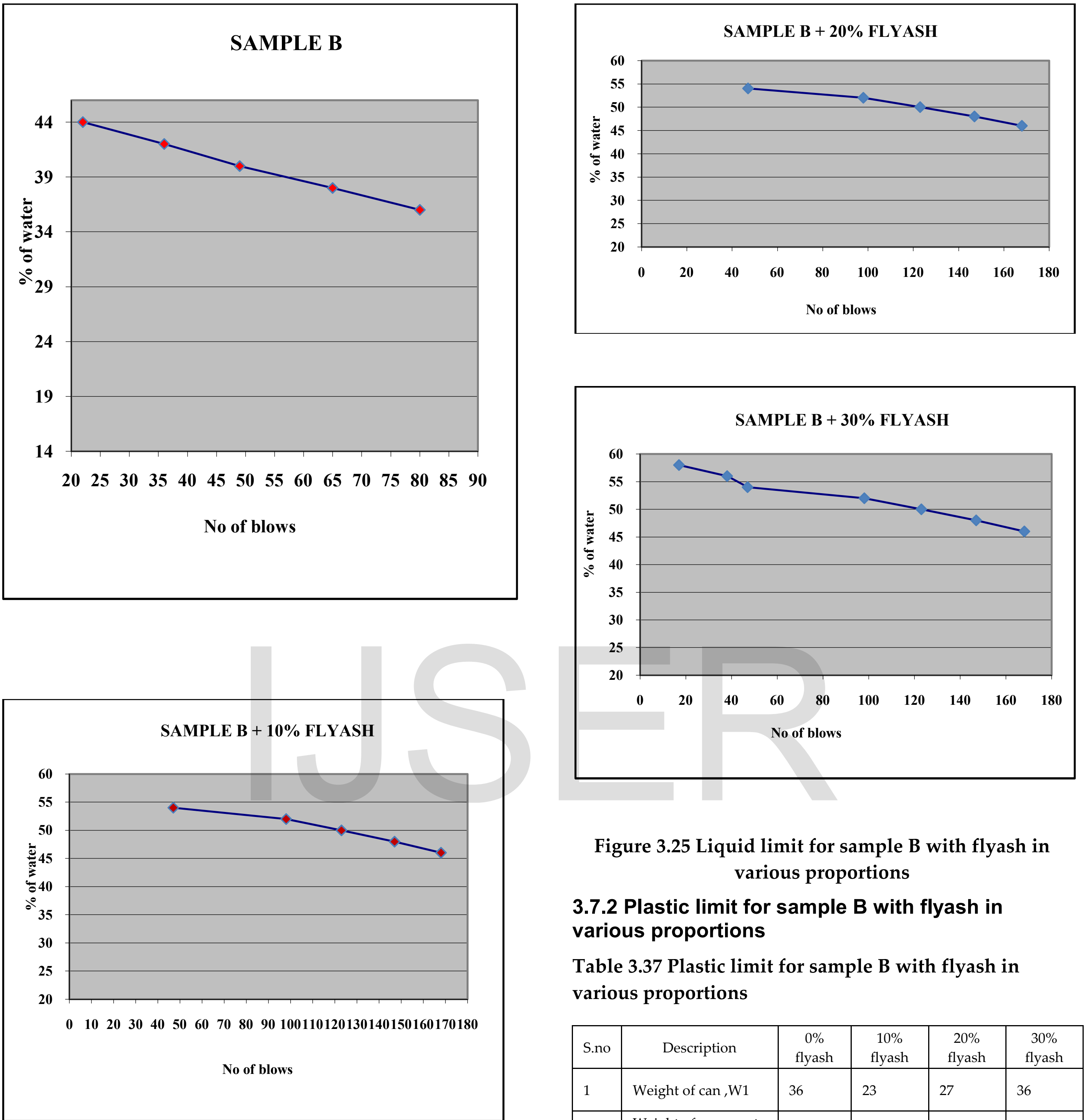

Figure 3.25 Liquid limit for sample B with flyash in various proportions

3.7.2 Plastic limit for sample B with flyash in various proportions

Table 3.37 Plastic limit for sample B with flyash in various proportions

S.no	Description	0% flyash	10% flyash	20% flyash	30% flyash
1	Weight of can ,W1	36	23	27	36
2	Weight of can + wet soil ,W2	46	35	38	47
3	Weight of can + dry soil , W3	44	33	36	46
4	Weight of water, (W2-W3)	2	2	2	1
5	Weight of dry soil, (W3-W1)	8	10	9	10
6	Moisture content, (W2-W3)/ (W3-W1) %	25	20	11.11	10

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Table 3.38 Plasticity index for sample B with flyash in various proportions

Samples	Liquid limit (w_l) %	Plastic limit (w_p) %	Plasticity index (i_p) (w_l - w_p) %
0% flyash	42	25	17
10% flyash	41.45	20	21.45
20% flyash	35.73	11.11	24.26
30% flyash	21.79	10	11.79

3.7.4 Shrinkage limit for sample B with flyash in various proportions

Table 3.39 Shrinkage limit for sample B with flyash in various proportions

S.No	Determination No.	0% flyash	10% flyash	20% flyash	30% flyash
1	Wt. of container in gm,W_1	183	154	157	183
2	Wt. of container + wet soil pat in gm,W_2	229	215	205	228
3	Wt. of container + dry soil pat in gm,W_3	213	199	192	216
4	Wt. of wet soil, W4= W_2 - W_1	46	61	48	45
5	Wt. of dry soil, W5= W_3 – W_1	30	45	35	33
6	Wt. of container + mercury filling dish, W_6 in gm	588	584	525	587
7	Wt of mercury filling dish W7= W_6 – W_1	405	430	368	404
8	Wt. of dish + mercury after displayed by dry pat W8 gms	250	252	253	307
9	Wt. Of mercury displayed by dry pat, W9	338	332	272	280
10	Volume of wet soil pat V1, in cm^3	29.8	31.6	27.06	29.7
11	Volume of dry soil pat V2, in cm^3	24.9	24.4	20	20.58
12	Shrinkage limit (W_s)	37	19.56	16.9	8.72
13	Shrinkage ratio (R) = W5 / V2	1.205	1.844	1.75	1.331
14	Volumetric shrinkage	0.1968	0.1599	0.354	0.4217

3.7.5 Standard proctor compaction test for sample B with flyash in various proportions

Table 3.40 Standard proctor compaction test for sample B with 0%flyash

Standard proctor compaction test - Sample B + 0% flyash					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ρ) g/cc	Dry Density(ρ_d) g/cc
6	6178	4446	1732	1.709385	1.612627
8	6253	4446	1807	1.783406	1.651301
10	6402	4446	1956	1.93046	1.754964
12	6534	4446	2088	2.060736	1.839943
14	6560	4446	2114	2.086397	1.830173
16	6504	4446	2058	2.031128	1.750973

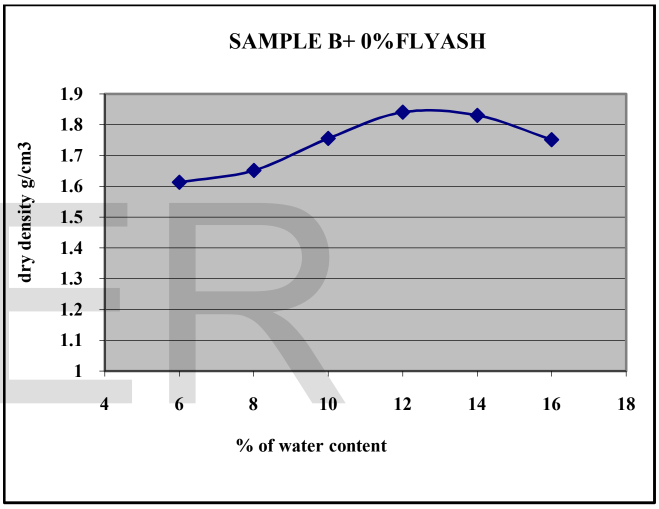

Figure 3.26 Standard proctor compaction test for sample B with 0%flyash

Table 3.41 Standard proctor compaction test for sample B with 10%flyash

Standard Proctor Compaction Test - Sample B (10 % Flyash)					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ρ) g/cc	Dry Density(ρ_d) g/cc
6	6023	4446	1577	1.556409	1.46831
8	6118	4446	1672	1.650168	1.527934
10	6207	4446	1761	1.738006	1.580006
12	6274	4446	1828	1.804131	1.610832
14	6308	4446	1862	1.837687	1.612006
16	6384	4446	1938	1.912695	1.648875
18	6298	4446	1852	1.827818	1.548998

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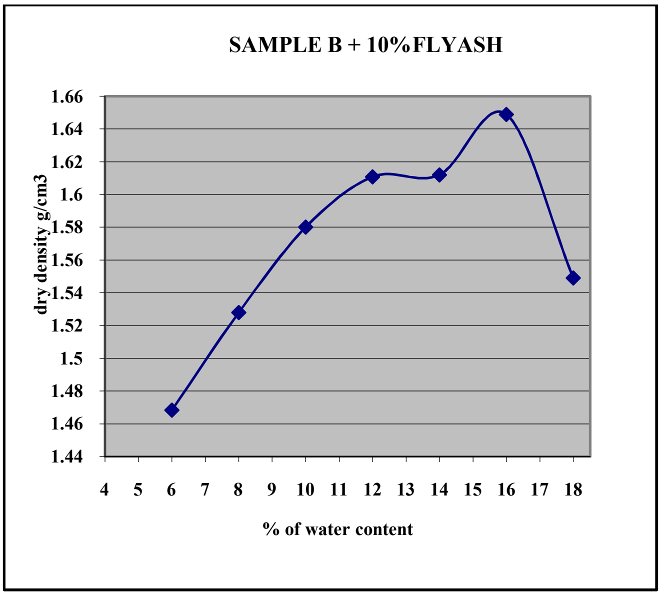

Figure 3.27 Standard proctor compaction test for sample B with 10%flyash

Table 3.42 Standard proctor compaction test for sample B with 20%flyash

Standard Proctor Compaction Test - Sample B (20 % Flyash)					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of comp. soil (g)	Wet density(ϱ) g/cc	Dry Density(ϱd) g/cc
6	6034	4446	1588	1.567265	1.478552
8	6137	4446	1691	1.66892	1.545296
10	6289	4446	1843	1.818935	1.653578
12	6388	4446	1942	1.916643	1.711288
14	6438	4446	1992	1.96599	1.724553
16	6487	4446	2041	2.01435	1.736509
18	6449	4446	2003	1.976846	1.675293

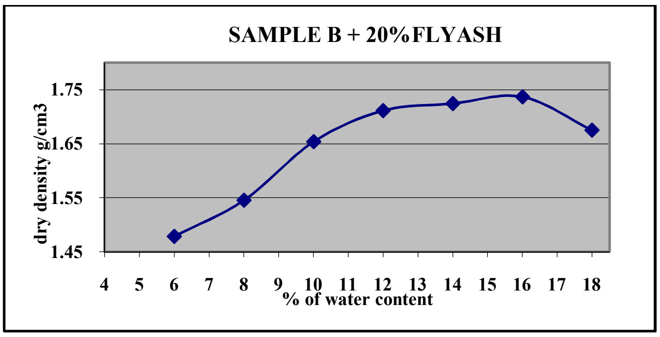

Figure 3.28 Standard proctor compaction test for sample B with 20%flyash

Table 3.43 Standard proctor compaction test for sample B with 30%flyash

Standard Proctor Compaction Test - Sample B (30 % Flyash)					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of com. soil (g)	Wet Den. (ϱ) g/cc	Dry Den. (ϱd) g/cc
6	6478	4552	1926	1.900852	1.793256
8	6647	4552	2095	2.067645	1.914486
10	6715	4552	2163	2.134757	1.940688
12	6669	4552	2117	2.089358	1.865498

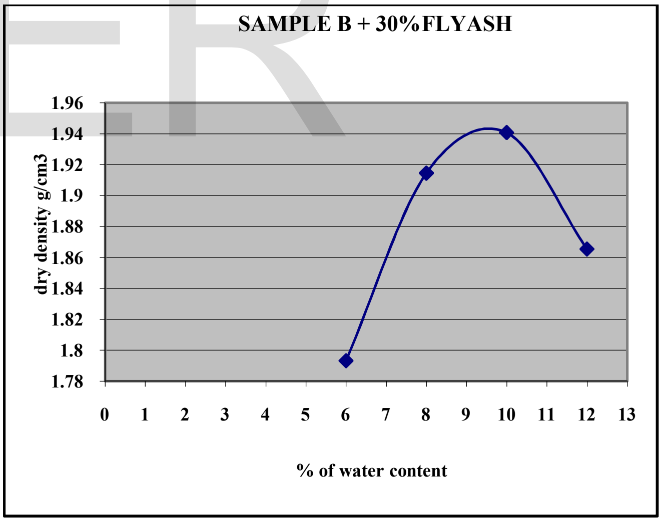

Figure 3.29 Standard proctor compaction test for sample B with 30%flyash

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Table 3.33 Maximum dry density and OMC value for sample B

Sample B with flyash for various Proportions	MDD (g/cm³)	OMC %
Sample B+ 0% flyash	1.839943	12
Sample B+ 10% flyash	1.736509	16
Sample B+ 20% flyash	1.736509	16
Sample B+ 30% flyash	1.940688	10

3.7.6 CBR test for soil sample B with flyash in various proportions

Table 3.44 CBR test for soil sample B with 0%flyash

Sample B + 0% flyash										
Sl. No	Un soaked					Soaked				
	Penetration		Load			Penetration		Load		
	Div	mm	Div	N	kg	Div	mm	Div	N	kg
1	50	0.5	26	735.94	73.59	50	0.5	7	198.14	19.81
2	100	1	34	962.39	96.23	100	1	16	452.89	45.28
3	150	1.5	40	1132.22	113.22	150	1.5	20	566.11	56.61
4	200	2	44	1245.44	124.54	200	2	22	622.72	62.27
5	250	2.5	50	1415.28	141.52	250	2.5	23	651.03	65.10
6	300	3	64	1811.55	181.15	300	3	24	679.33	67.93
7	350	3.5	70	1981.39	198.13	350	3.5	25	707.64	70.76
8	400	4	82	2321.05	232.10	400	4	26	735.94	73.59
9	450	4.5	90	2547.50	254.74	450	4.5	28	792.55	79.25
10	500	5	102	2887.16	288.71	500	5	30	849.17	84.91
11	550	5.5	104	2943.77	294.37	550	5.5	31	877.47	87.74
12	600	6	108	3056.99	305.69	600	6	33	934.08	93.40
13	650	6.5	116	3283.44	328.34	650	6.5	34	962.39	96.23
14	700	7	118	3340.05	334.00	700	7	34	962.39	96.23
15	750	7.5	120	3396.66	339.66	750	7.5	35	990.69	99.06
16	800	8	122	3453.27	345.32	800	8	35	990.69	99.06
17	850	8.5	124	3509.88	350.98	850	8.5	36	1019.00	101.89
18	900	9	126	3566.49	356.64	900	9	36	1019.00	101.89
19	950	9.5	128	3623.10	362.31	950	9.5	37	1047.30	104.73
20	1000	10	130	3679.72	367.97	1000	10	37	1047.30	104.73

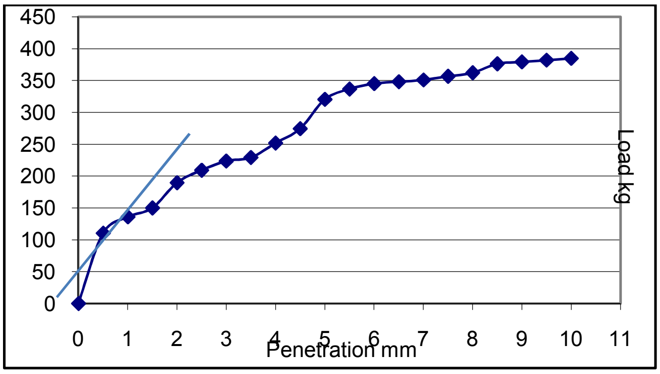

Figure 3.30 CBR test for soil sample B with 0%flyash- Un soaked

CBR		
2.5 mm	141	10.33724
5 mm	289	14.15973

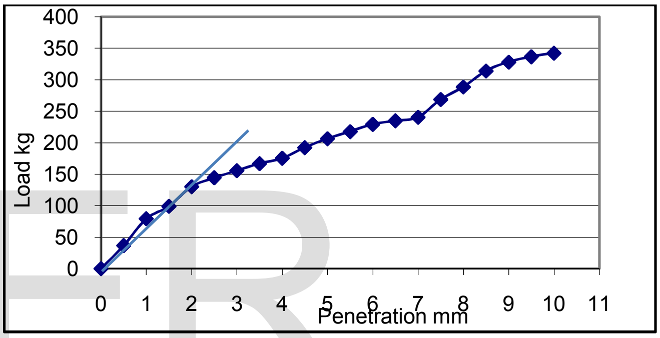

Figure 3.31 CBR test for soil sample B with 0%flyash- Soaked

CBR		
2.5 mm	57	4.160584
5 mm	84	4.087591

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Table 3.44 CBR test for soil sample B with 10%flyash

Sample B + 10% Flyash										
Sl No.	Un soaked					Soaked				
	Penetration		Load			Penetration		Load		
	Div	mm	Div	N	kg	Div	mm	Div	N	kg
1	50	0.5	32	905.78	90.5776	50	0.5	9	254.75	25.47495
2	100	1	41	1160.53	116.0526	100	1	18	509.50	50.9499
3	150	1.5	48	1358.66	135.8664	150	1.5	22	622.72	62.2721
4	200	2	56	1585.11	158.5108	200	2	23	651.03	65.10265
5	250	2.5	62	1754.94	175.4941	250	2.5	25	707.64	70.76375
6	300	3	68	1924.77	192.4774	300	3	27	764.25	76.42485
7	350	3.5	77	2179.52	217.9524	350	3.5	29	820.86	82.08595
8	400	4	89	2519.19	251.919	400	4	33	934.08	93.40815
9	450	4.5	101	2858.86	285.8856	450	4.5	35	990.69	99.06925
10	500	5	115	3255.13	325.5133	500	5	37	1047.30	104.7304
11	550	5.5	116	3283.44	328.3438	550	5.5	39	1103.91	110.3915
12	600	6	117	3311.74	331.1744	600	6	41	1160.53	116.0526
13	650	6.5	118	3340.05	334.0049	650	6.5	43	1217.14	121.7137
14	700	7	119	3368.35	336.8355	700	7	45	1273.75	127.3748
15	750	7.5	120	3396.66	339.666	750	7.5	47	1330.36	133.0359
16	800	8	124	3509.88	350.9882	800	8	49	1386.97	138.697
17	850	8.5	127	3594.80	359.4799	850	8.5	51	1443.58	144.3581
18	900	9	128	3623.10	362.3104	900	9	53	1500.19	150.0192
19	950	9.5	130	3679.72	367.9715	950	9.5	54	1528.50	152.8497
20	1000	10	132	3736.33	373.6326	1000	10	55	1556.80	155.6803

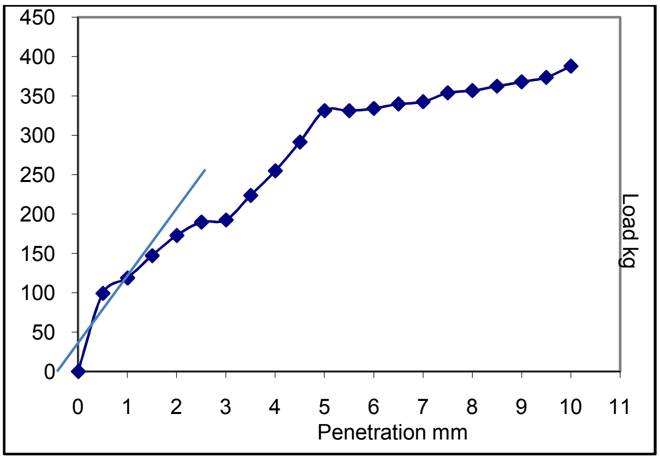

Figure 3.32 CBR test for soil sample B with 10%flyash-Un soaked

CBR		
2.5 mm	175	12.82991
5 mm	325	15.92357

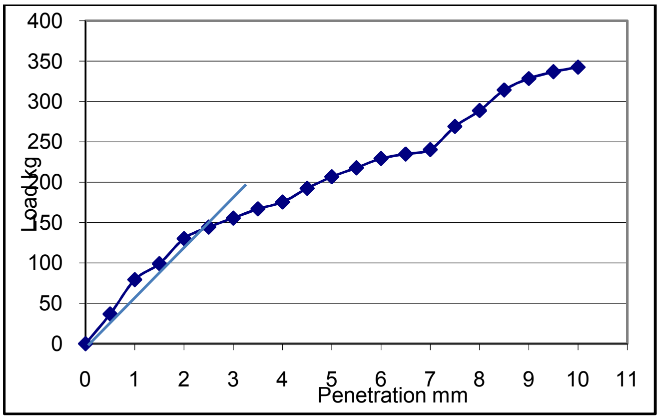

Figure 3.33CBR test for soil sample B with 10%flyash – Soaked

CBR		
2.5 mm	70	5.109489
5 mm	104	5.060827

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Table 3.45 CBR test for soil sample B with 20%flyash

Sample B + 20% Flyash										
Sl No.	Un soaked					Soaked				
	Penetration		Load			Penetration		Load		
	Div	mm	Div	N	kg	Div	mm	Div	N	kg
1	50	0.5	33	934.08	93.40815	50	0.5	11	311.36	31.13605
2	100	1	42	1188.83	118.8831	100	1	19	537.80	53.78045
3	150	1.5	49	1386.97	138.697	150	1.5	28	792.55	79.2554
4	200	2	58	1641.72	164.1719	200	2	36	1019.00	101.8998
5	250	2.5	65	1839.86	183.9858	250	2.5	40	1132.22	113.222
6	300	3	68	1924.77	192.4774	300	3	45	1273.75	127.3748
7	350	3.5	79	2236.13	223.6135	350	3.5	51	1443.58	144.3581
8	400	4	89	2519.19	251.919	400	4	54	1528.50	152.8497
9	450	4.5	101	2858.86	285.8856	450	4.5	56	1585.11	158.5108
10	500	5	116	3283.44	328.3438	500	5	58	1641.72	164.1719
11	550	5.5	117	3311.74	331.1744	550	5.5	63	1783.25	178.3247
12	600	6	117	3311.74	331.1744	600	6	69	1953.08	195.308
13	650	6.5	119	3368.35	336.8355	650	6.5	75	2122.91	212.2913
14	700	7	120	3396.66	339.666	700	7	82	2321.05	232.1051
15	750	7.5	124	3509.88	350.9882	750	7.5	89	2519.19	251.919
16	800	8	126	3566.49	356.6493	800	8	93	2632.41	263.2412
17	850	8.5	127	3594.80	359.4799	850	8.5	97	2745.63	274.5634
18	900	9	129	3651.41	365.141	900	9	99	2802.24	280.2245
19	950	9.5	131	3708.02	370.8021	950	9.5	101	2858.86	285.8856
20	1000	10	135	3821.24	382.1243	1000	10	102	2887.16	288.7161

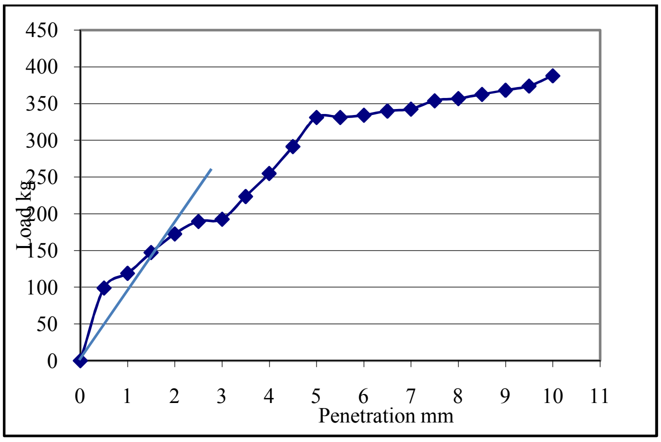

Figure 3.34 CBR test for soil sample B with 20%flyash-Un soaked

CBR		
2.5 mm	183	13.41642
5 mm	328	16.07055

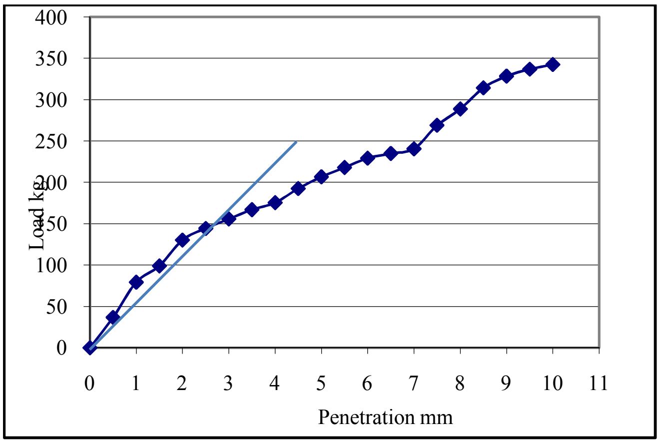

Figure 3.35 CBR test for soil sample B with 20%flyash-Soaked

CBR		
2.5 mm	113	8.248175
5 mm	165	8.029197

Table 3.46 CBR test for soil sample B with 30%flyash

Sample B + 30% Flyash										
Sl No.	Un soaked					Soaked				
	Penetration		Load			Penetration		Load		
	Div	mm	Div	N	kg	Div	mm	Div	N	kg
1	50	0.5	35	990.69	99.06925	50	0.5	13	367.97	36.79715
2	100	1	42	1188.83	118.8831	100	1	28	792.55	79.2554
3	150	1.5	52	1471.89	147.1886	150	1.5	35	990.69	99.06925
4	200	2	61	1726.64	172.6636	200	2	46	1302.05	130.2053
5	250	2.5	67	1896.47	189.6469	250	2.5	51	1443.58	144.3581
6	300	3	68	1924.77	192.4774	300	3	55	1556.80	155.6803
7	350	3.5	79	2236.13	223.6135	350	3.5	59	1670.02	167.0025
8	400	4	90	2547.50	254.7495	400	4	62	1754.94	175.4941
9	450	4.5	103	2915.47	291.5467	450	4.5	68	1924.77	192.4774
10	500	5	117	3311.74	331.1744	500	5	73	2066.30	206.6302
11	550	5.5	117	3311.74	331.1744	550	5.5	77	2179.52	217.9524
12	600	6	118	3340.05	334.0049	600	6	81	2292.75	229.2746
13	650	6.5	120	3396.66	339.666	650	6.5	83	2349.36	234.9357
14	700	7	121	3424.97	342.4966	700	7	85	2405.97	240.5968
15	750	7.5	125	3538.19	353.8188	750	7.5	95	2689.02	268.9023
16	800	8	126	3566.49	356.6493	800	8	102	2887.16	288.7161
17	850	8.5	128	3623.10	362.3104	850	8.5	111	3141.91	314.1911
18	900	9	130	3679.72	367.9715	900	9	116	3283.44	328.3438
19	950	9.5	132	3736.33	373.6326	950	9.5	119	3368.35	336.8355
20	1000	10	137	3877.85	387.7854	1000	10	121	3424.97	342.4966

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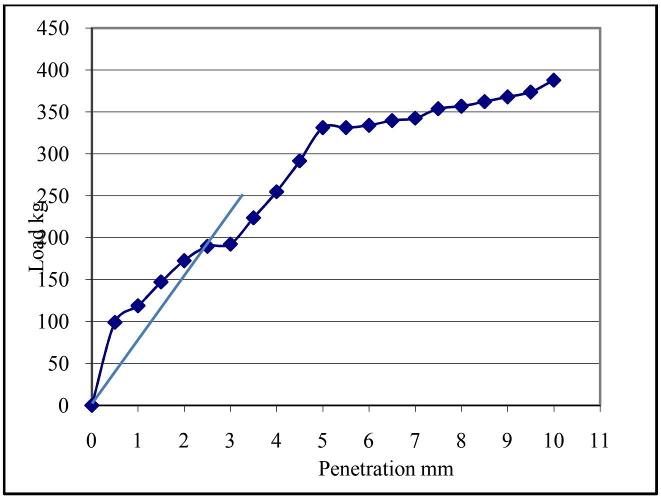

Figure 3.35 CBR test for soil sample B with 20%flyash-Soaked

CBR

2.5 mm	113	8.248175
5 mm	165	8.029197

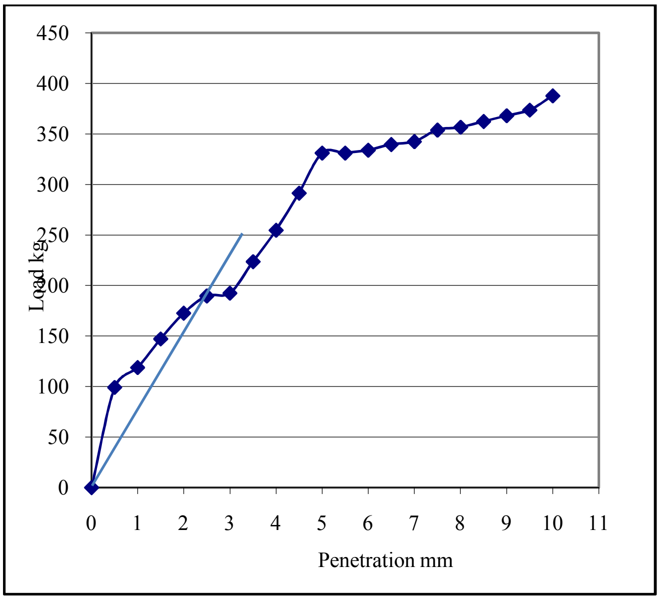

Figure 3.36 CBR test for soil sample B with 30%flyash-Un soaked

CBR

2.5 mm	189	13.8563
5 mm	331	16.21754

Table 3.46 CBR test for soil sample B with 30%flyash

Sample B + 30% Flyash										
Sl No.	Un soaked					Soaked				
	Penetration		Load			Penetration		Load		
	Div	mm	Div	N	kg	Div	mm	Div	N	kg
1	50	0.5	35	990.69	99.06925	50	0.5	13	367.97	36.79715
2	100	1	42	1188.83	118.8831	100	1	28	792.55	79.2554
3	150	1.5	52	1471.89	147.1886	150	1.5	35	990.69	99.06925
4	200	2	61	1726.64	172.6636	200	2	46	1302.05	130.2053
5	250	2.5	67	1896.47	189.6469	250	2.5	51	1443.58	144.3581
6	300	3	68	1924.77	192.4774	300	3	55	1556.80	155.6803
7	350	3.5	79	2236.13	223.6135	350	3.5	59	1670.02	167.0025
8	400	4	90	2547.50	254.7495	400	4	62	1754.94	175.4941
9	450	4.5	103	2915.47	291.5467	450	4.5	68	1924.77	192.4774
10	500	5	117	3311.74	331.1744	500	5	73	2066.30	206.6302
11	550	5.5	117	3311.74	331.1744	550	5.5	77	2179.52	217.9524
12	600	6	118	3340.05	334.0049	600	6	81	2292.75	229.2746
13	650	6.5	120	3396.66	339.666	650	6.5	83	2349.36	234.9357
14	700	7	121	3424.97	342.4966	700	7	85	2405.97	240.5968
15	750	7.5	125	3538.19	353.8188	750	7.5	95	2689.02	268.9023
16	800	8	126	3566.49	356.6493	800	8	102	2887.16	288.7161
17	850	8.5	128	3623.10	362.3104	850	8.5	111	3141.91	314.1911
18	900	9	130	3679.72	367.9715	900	9	116	3283.44	328.3438
19	950	9.5	132	3736.33	373.6326	950	9.5	119	3368.35	336.8355
20	1000	10	137	3877.85	387.7854	1000	10	121	3424.97	342.4966

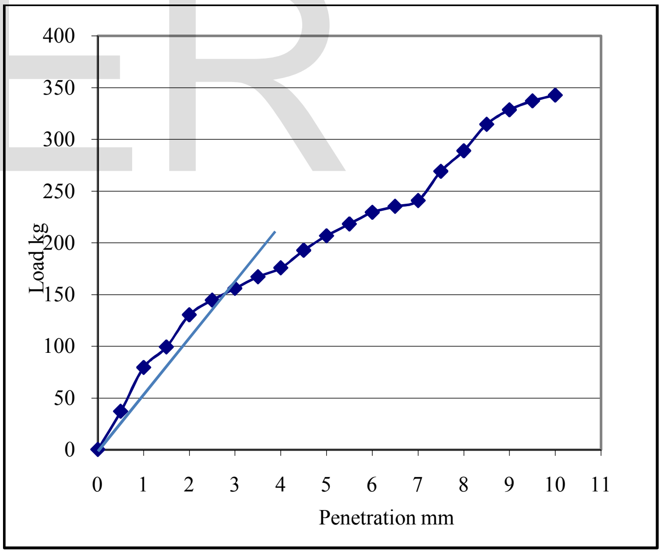

Figure 3.37 CBR test for soil sample B with 30%flyash-Soaked

CBR

2.5 mm	145	10.58394
5 mm	207	10.0729

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3.8 SOIL SAMPLE A WITH GYPSUM IN VARIOUS PROPORTIONS

3.8.1 Differential free swell index soil sample A with gypsum in various proportions

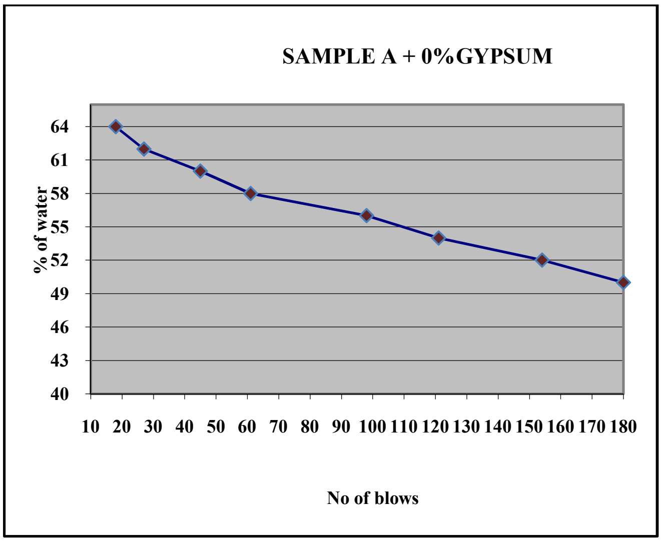
Table 3.47 Differential free swell index sample A with gypsum in various proportions

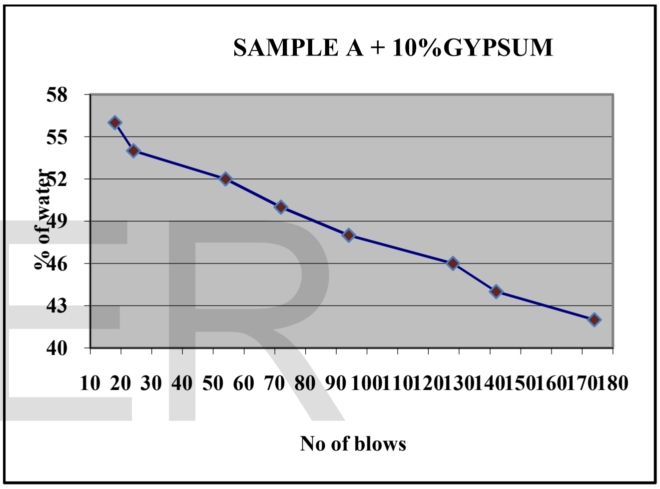
S.no	Observation	0% gypsum	10% gypsum	20% gypsum	30% gypsum
1	Volume of the soil in kerosene after swelling, V1 ml	21	21	21	21
2	Volume of soil in water after swelling,V2 ml	32	32	33	34.5
3	The free swell index of the soil (%)	52.38 %	52.38 %	57.14 %	64.28 %
Degree of expansiveness of soil: Since, The Free swell index is greater than 50.		very high	very high	very high	very high

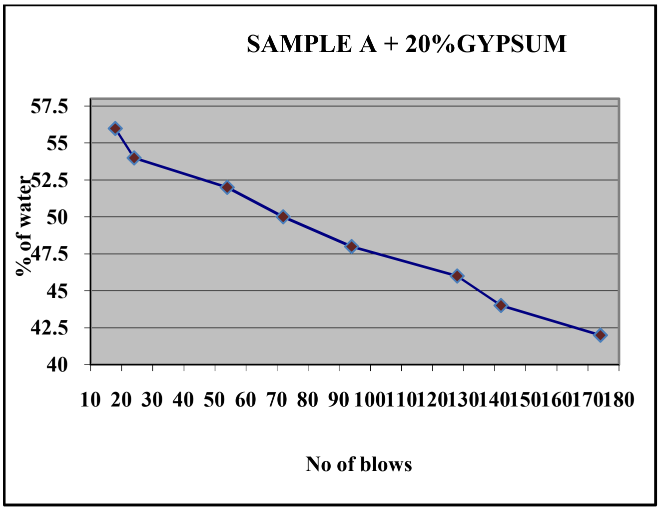
3.8.2 Liquid limit for sample A with gypsum in various proportions

Table 3.48 Liquid limit soil sample A with gypsum in various proportions

Samples	Observations									Liquid limit
0% gypsum	No of Blows	180	154	121	98	61	45	27	18	62.4 %
	Water Content	50	52	54	56	58	60	62	64	
	Wt. of soil	100	100	100	100	100	100	100	100	
10 % gypsum	No of Blows	176	140	126	93	71	55	25	19	57.6 %
	Water Content	44	46	48	50	52	54	56	58	
	Wt. of soil	100	100	100	100	100	100	100	100	
20 % gypsum	No of Blows	184	140	111	82	54	32	23	13	55.2 %
	Water Content	42	44	46	48	50	52	54	56	
	Wt. of soil	100	100	100	100	100	100	100	100	
30 % gypsum	No of Blows	144	119	75	31	19	-	-	-	41.6 %
	Water Content	34	36	38	40	42	-	-	-	
	Wt. of soil	100	100	100	100	100	-	-	-	







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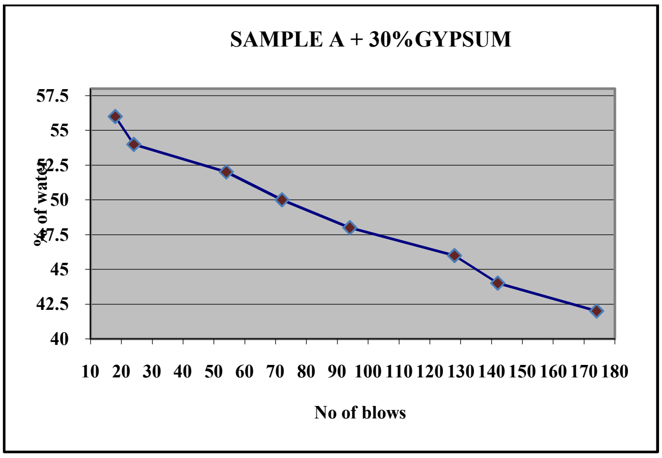

Figure 3.38 Liquid limit soil sample A with gypsum in various proportions

3.8.3 Plastic limit sample A with gypsum in various proportions

Table 3.48 Plastic limit sample a with gypsum in various proportions

S. no	Description	0% gypsum	10% gypsum	20% gypsum	30% gypsum
1	Weight of can ,W1	36	36	25	31
2	Weight of can + wet soil ,W2	46	45	34	42
3	Weight of can + dry soil , W3	43	44	33	40
4	Weight of water, (W2-W3)	3	1	1	2
5	Weight of dry soil, (W3-W1)	7	8	8	9
6	Moisture content, %	42.857	42.857	25	22.22

3.8.4 Plasticity index sample A with gypsum in various proportions

Table 3.49 Plasticity index sample a with gypsum in various proportions

Samples	Liquid limit (w_l) %	Plastic limit (w_p) %	Plasticity index (i_p) (w_l- w_p) %
0% gypsum	62.4	42.857	19.543
10% gypsum	57.6	42.857	14.743
20% gypsum	55.2	25	30.2
30% gypsum	41.6	22.22	19.38

3.8.5 Shrinkage limit sample A with gypsum in various proportions

Table 3.50 Shrinkage limit sample A with gypsum in various proportions

S.no	Determination no.	0% gypsum	10% gypsum	20% gypsum	30% gypsum
1	Wt. of container in gm,W_1	183	154	157	183
2	Wt. of container + wet soil pat in gm,W_2	229	207	205	223
3	Wt. of container + dry soil pat in gm,W_3	213	192	192	211
4	Wt. of wet soil, W4= W_2 - W_1	46	53	48	40
5	Wt. of dry soil, W5= W_3 – W_1	30	38	35	28
6	Wt. of container + mercury filling dish, W_6 in gm	588	584	525	590
7	Wt of mercury filling dish W7	405	430	368	407
8	Wt. of dish + mercury after displayed by dry pat W8 gms	250	252	253	304
9	Wt. Of mercury displayed by dry pat, W9=W6-W8	338	332	272	286
10	Volume of wet soil pat V1, in cm^3	29.8	31.6	27.06	29.9
11	Volume of dry soil pat V2,in cm^3	24.9	24.4	20	21.03
12	Shrinkage limit (W_s)	37	21	17	11.2
13	Shrinkage ratio (R) = W5 / V2	1.205	1.557	1.75	1.331
14	Volumetric shrinkage VS	0.1968	0.2949	0.354	0.4217

3.8.6 Standard proctor compaction test soil sample A with gypsum in various proportions

Table 3.50 Standard proctor compaction test soil sample A with 0%gypsum

Standard proctor compaction test - sample A + 0%gypsum					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ρ) g/cc	Dry Density($ρ_d$) g/cc
6	5907	4505	1402	1.383694	1.305371
8	5986	4505	1481	1.461662	1.353391
10	6036	4505	1531	1.511009	1.373645
12	6114	4505	1609	1.587991	1.417849
14	6202	4505	1697	1.674842	1.46916
16	6234	4505	1729	1.706424	1.471055
18	6241	4505	1736	1.713333	1.451977
20	6252	4505	1747	1.724189	1.436824
22	6243	4505	1738	1.715306	1.405989

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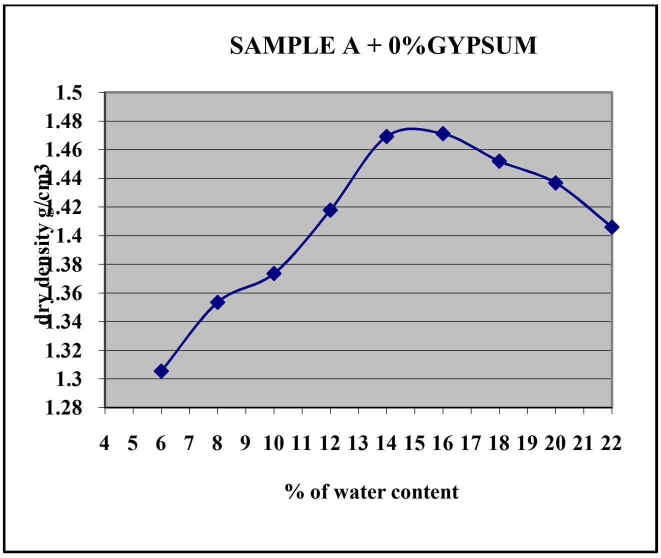

Figure 3.39 Standard proctor compaction test soil sample A with 0%gypsum

Table 3.51 Standard proctor compaction test soil sample A with 10%gypsum

Standard Proctor Compaction Test - Sample A + 10%Gypsum					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ϱ) g/cc	Dry Density(ϱd) g/cc
6	6002	4446	1556	1.535683	1.448757
8	6043	4446	1597	1.576148	1.459396
10	6101	4446	1655	1.63339	1.4849
12	6203	4446	1757	1.734058	1.548266
14	6231	4446	1785	1.761693	1.545345
16	6245	4446	1799	1.77551	1.530612
18	6254	4446	1808	1.784392	1.512197
20	6237	4446	1791	1.767614	1.473012

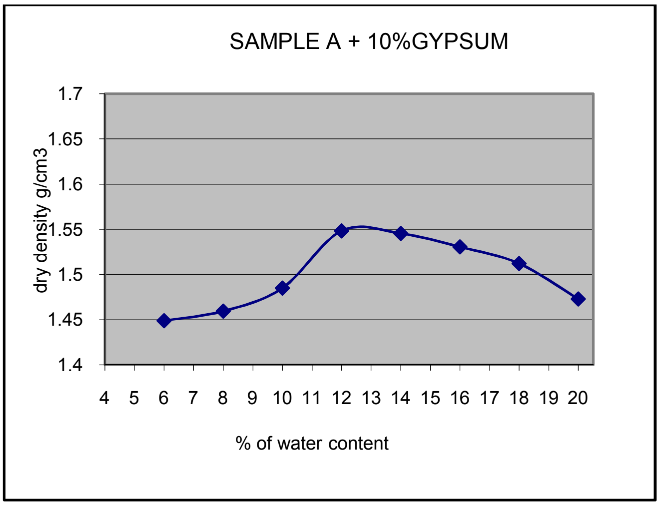

Figure 3.40 Standard proctor compaction test soil sample A with 10%gypsum

Table 3.52 Standard proctor compaction test soil sample A with20%gypsum

Standard Proctor Compaction Test - Sample A + 20%Gypsum					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ϱ) g/cc	Dry Density(ϱd) g/cc
6	6244	4446	1798	1.774523	1.674078
8	6362	4446	1916	1.890982	1.75091
10	6412	4446	1966	1.940329	1.763936
12	6478	4446	2032	2.005468	1.790596
14	6580	4446	2134	2.106136	1.847488
16	6621	4446	2175	2.1466	1.850518
18	6408	4446	1962	1.936382	1.641001

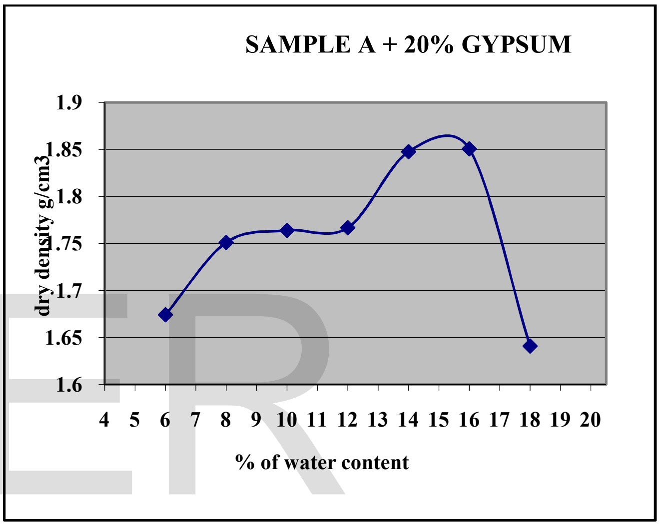

Figure 3.41 Standard proctor compaction test soil sample A with 20%gypsum

Table 3.53 Standard proctor compaction test soil sample A with 30%gypsum

Standard Proctor Compaction Test - Sample A + 30%Gypsum					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ϱ) g/cc	Dry Density(ϱd) g/cc
6	6034	4505	1529	1.509035	1.423618
8	6134	4505	1629	1.60773	1.488639
10	6176	4505	1671	1.649181	1.499256
12	6243	4505	1738	1.715306	1.531524
14	6278	4505	1773	1.749849	1.534956
16	6313	4505	1808	1.784392	1.538269
18	6361	4505	1856	1.831766	1.552344
20	6363	4505	1858	1.83374	1.528116
22	6349	4505	1844	1.819922	1.49174

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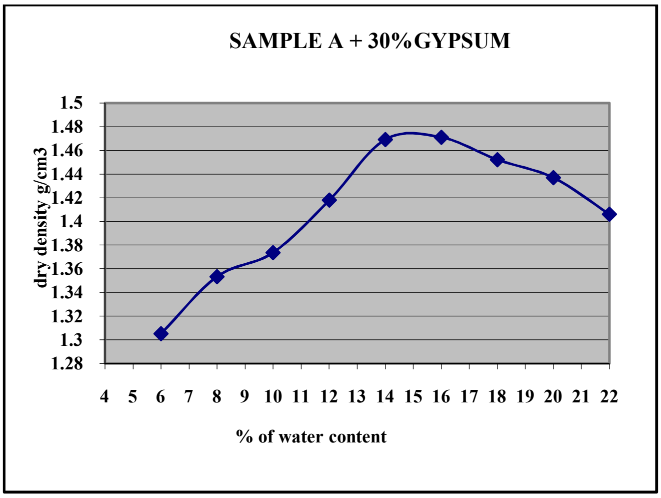

Figure 3.42 Standard proctor compaction test soil sample A with 20%gypsum

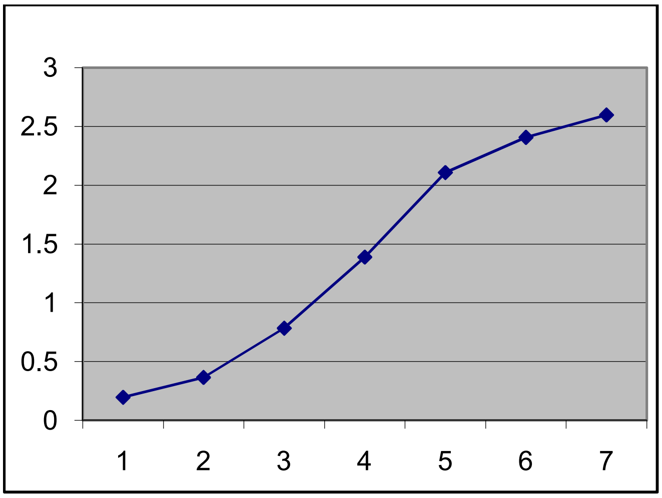

Figure 3.43 Unconfined compressive strength test for soil sample A with 0% gypsum-1

Table 3.54 Maximum dry density and OMC value for sample A

Sample And gypsum Proportions	MDD (g/cm³)	OMC %
Sample A+ 0% gypsum	1.451977	18
Sample A+ 10% gypsum	1.548266	18
Sample A+ 20% gypsum	1.850518	16
Sample A+ 30% gypsum	1.552344	18

Table 3.56 Unconfined compressive strength test for soil sample A with 0% gypsum

Unconfined compressive strength test- Sample A Second specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	0.2	1.582	11.3354	12.13367	0.130381
100	1	0.1315789	1.7	13.447	11.3354	13.05288	1.030194
150	1.5	0.1973684	4	31.64	11.3354	14.12279	2.24035
200	2	0.2631579	7.4	58.534	11.3354	15.38376	3.804922
250	2.5	0.3289474	10.2	80.682	11.3354	16.89197	4.776353
300	3	0.3947368	12	94.92	11.3354	18.72805	5.068333
350	3.5	0.4605263	13	102.83	11.3354	21.01196	4.893879

3.8.7 Unconfined compressive strength test for soil sample A with gypsum in various proportions

Table 3.55 Unconfined compressive strength test for soil sample A with 0% gypsum

Unconfined compressive strength test- sample A First specimen							
reading	dl	E(strain)	gauge reading	Load	Ao	Cor. Area	Stress
50	0.5	0.0657895	1.4	11.074	11.3354	12.13367	0.912667
100	1	0.1315789	3.6	28.476	11.3354	13.05288	2.181587
150	1.5	0.1973684	6.2	49.042	11.3354	14.12279	3.472542
200	2	0.2631579	8.4	66.444	11.3354	15.38376	4.319101
250	2.5	0.3289474	10	79.1	11.3354	16.89197	4.682699
300	3	0.3947368	10.8	85.428	11.3354	18.72805	4.561499
350	3.5	0.4605263	10.6	83.846	11.3354	21.01196	3.990394

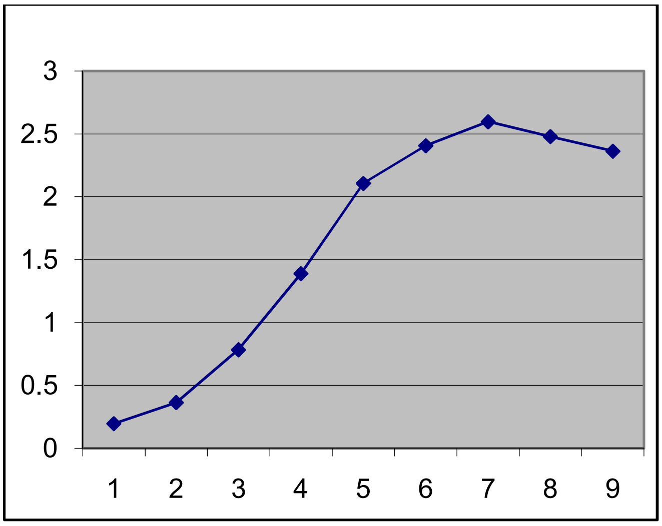

Figure 3.44 Unconfined compressive strength test for soil sample A with 0% gypsum-2

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Table 3.57 Unconfined compressive strength test for soil sample A with 10% gypsum

Unconfined compressive strength test- sample A (10% flyash) – first specimen							
reading	dl	E(strain)	pro ving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	1.4	11.074	11.3354	12.13367	0.912667
100	1	0.1315789	4.2	33.222	11.3354	13.05288	2.545184
150	1.5	0.1973684	6.6	52.206	11.3354	14.12279	3.696577
200	2	0.2631579	8.4	66.444	11.3354	15.38376	4.319101
250	2.5	0.3289474	9.8	77.518	11.3354	16.89197	4.589045
300	3	0.3947368	10.6	83.846	11.3354	18.72805	4.477027
350	3.5	0.4605263	10.8	85.428	11.3354	21.01196	4.065684
400	4	0.5263158	11	87.01	11.3354	23.93029	3.635978
450	4.5	0.5921053	10.8	85.428	11.3354	27.79001	3.074054

Table 3.58 Unconfined compressive strength test for soil sample A with 10% gypsum

Unconfined compressive strength test- sample A (10% flyash) – Second specimer							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corre cted Area	Stress
50	0.5	0.0657895	1.2	9.492	11.3354	12.13367	0.78228
100	1	0.1315789	4	31.64	11.3354	13.05288	2.42398
150	1.5	0.1973684	6.2	49.042	11.3354	14.12279	3.47254
200	2	0.2631579	8.1	64.071	11.3354	15.38376	4.16484
250	2.5	0.3289474	9.8	77.518	11.3354	16.89197	4.58904
300	3	0.3947368	10.4	82.264	11.3354	18.72805	4.39255
350	3.5	0.4605263	10.6	83.846	11.3354	21.01196	3.99039
400	4	0.5263158	10.9	86.219	11.3354	23.93029	3.60292
450	4.5	0.5921053	10.6	83.846	11.3354	27.79001	3.01712

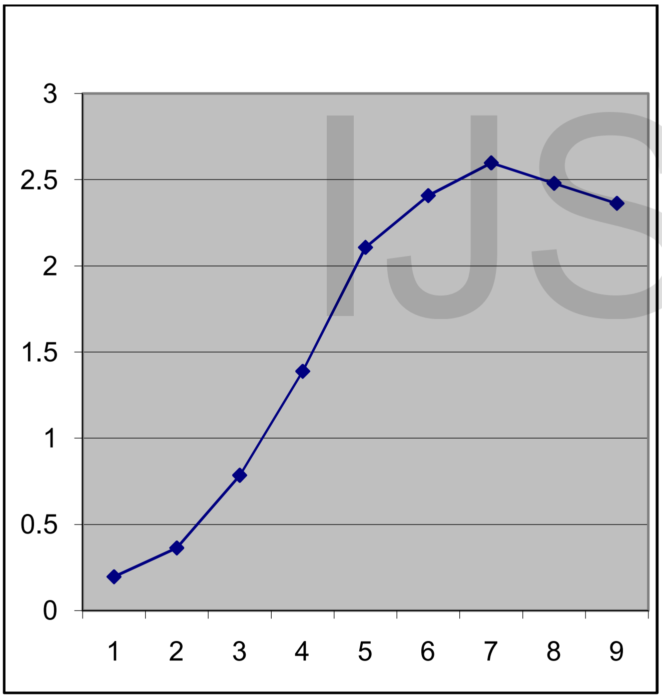

Figure 3.45 Unconfined compressive strength test for soil sample A with 0% gypsum-2

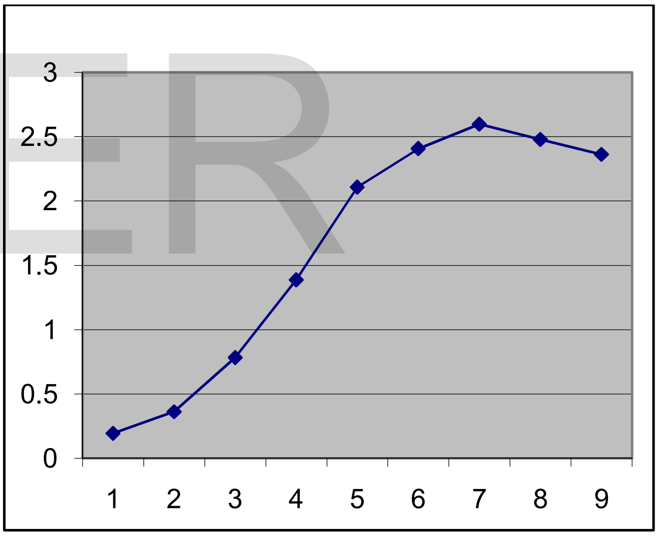

Figure 3.46 Unconfined compressive strength test for soil sample A with 10% gypsum-2

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Table 3.59 Unconfined compressive strength test for soil sample A with 20% gypsum

Unconfined compressive strength test- sample A (20% flyash) – First specimen							
reading	dl	E(strain)	gauge reading	Load	Ao	Cor. Area	Stress
50	0.5	0.0657895	2.2	17.402	11.3354	12.13367	1.434191
100	1	0.1315789	4.4	34.804	11.3354	13.05288	2.666384
150	1.5	0.1973684	6.4	50.624	11.3354	14.12279	3.58456
200	2	0.2631579	7.6	60.116	11.3354	15.38376	3.907758
250	2.5	0.3289474	8	63.28	11.3354	16.89197	3.746159
300	3	0.3947368	7.8	61.698	11.3354	18.72805	3.294416
350	3.5	0.4605263	7.6	60.116	11.3354	21.01196	2.861037

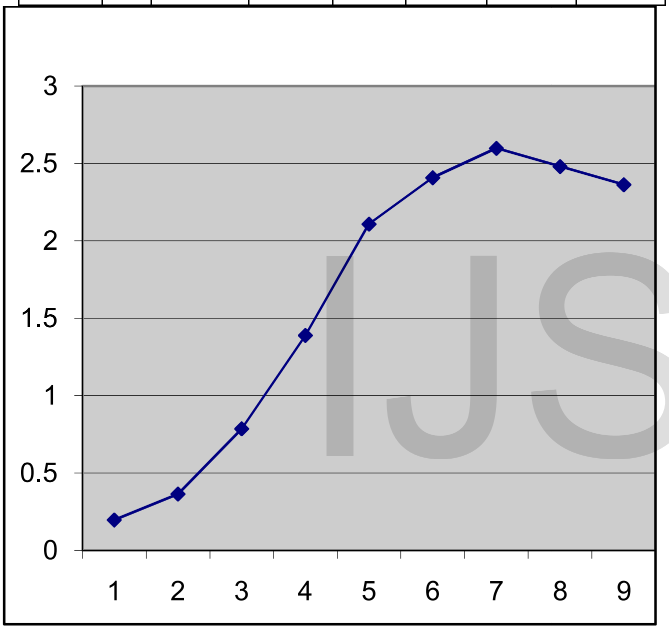

Figure 3.47 Unconfined compressive strength test for soil sample A with 20% gypsum-2

Table 3.60 Unconfined compressive strength test for soil sample A with 20% gypsum

Unconfined compressive strength test- sample A (20% flyash) – Second specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	2.6	20.566	11.3354	12.13367	1.694953
100	1	0.1315789	5.2	41.132	11.3354	13.05288	3.151181
150	1.5	0.1973684	6.8	53.788	11.3354	14.12279	3.808595
200	2	0.2631579	7.2	56.952	11.3354	15.38376	3.702087
250	2.5	0.3289474	7.8	61.698	11.3354	16.89197	3.652505
300	3	0.3947368	7.2	56.952	11.3354	18.72805	3.041
350	3.5	0.4605263	6.8	53.788	11.3354	21.01196	2.559875

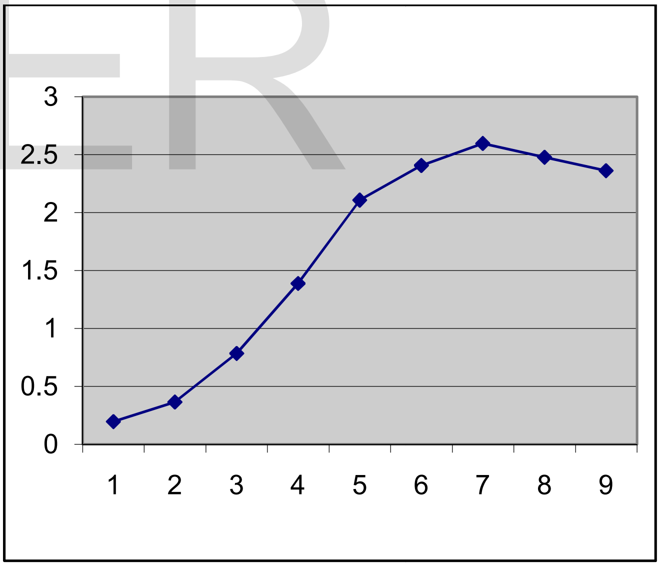

Figure 3.48 Unconfined compressive strength test for soil sample A with 20% gypsum-2

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Table 3.61 Unconfined compressive strength test for soil sample A with 30% gypsum

Unconfined compressive strength test- sample A (30% flyash) – first specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	0.2	1.582	11.3354	12.13367	0.130381
100	1	0.1315789	0.4	3.164	11.3354	13.05288	0.242399
150	1.5	0.1973684	1.2	9.492	11.3354	14.12279	0.672105
200	2	0.2631579	2.6	20.566	11.3354	15.38376	1.336865
250	2.5	0.3289474	4.2	33.222	11.3354	16.89197	1.966733
300	3	0.3947368	5.6	44.296	11.3354	18.72805	2.365222
350	3.5	0.4605263	6.6	52.206	11.3354	21.01196	2.484585
400	4	0.5263158	7.6	60.116	11.3354	23.93029	2.51213
450	4.5	0.5921053	8.4	66.444	11.3354	27.79001	2.390931
500	5	0.6578947	9	71.19	11.3354	33.13425	2.148532
550	5.5	0.7236842	9.2	72.772	11.3354	41.02335	1.773916
600	6	0.7894737	9.4	74.354	11.3354	53.84315	1.380937
650	6.5	0.8552632	9.6	75.936	11.3354	78.31731	0.969594
700	7	0.9210526	9.4	74.354	11.3354	143.5817	0.517851
750	7.5	0.9868421	9.4	74.354	11.3354	861.4904	0.086309

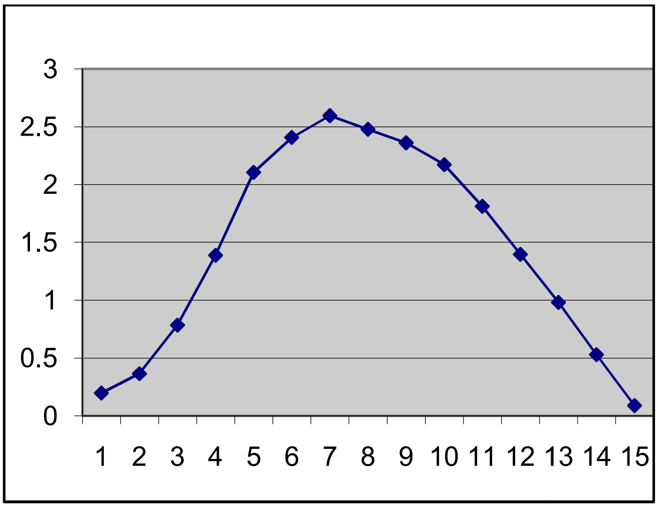

Figure 3.49 Unconfined compressive strength test for soil sample A with 30% gypsum-2

Table 3.62 Unconfined compressive strength test for soil sample A with 30% gypsum

Unconfined compressive strength test- sample A (30% flyash) – second specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	0.3	2.373	11.3354	12.13367	0.195572
100	1	0.1315789	0.6	4.746	11.3354	13.05288	0.363598
150	1.5	0.1973684	1.4	11.074	11.3354	14.12279	0.784122
200	2	0.2631579	2.7	21.357	11.3354	15.38376	1.388282
250	2.5	0.3289474	4.5	35.595	11.3354	16.89197	2.107214
300	3	0.3947368	5.7	45.087	11.3354	18.72805	2.407458
350	3.5	0.4605263	6.9	54.579	11.3354	21.01196	2.597521
400	4	0.5263158	7.5	59.325	11.3354	23.93029	2.479076
450	4.5	0.5921053	8.3	65.653	11.3354	27.79001	2.362467
500	5	0.6578947	9.1	71.981	11.3354	33.13425	2.172405
550	5.5	0.7236842	9.4	74.354	11.3354	41.02335	1.81248
600	6	0.7894737	9.5	75.145	11.3354	53.84315	1.395628
650	6.5	0.8552632	9.7	76.727	11.3354	78.31731	0.979694
700	7	0.9210526	9.6	75.936	11.3354	143.5817	0.52887
750	7.5	0.9868421	9.5	75.145	11.3354	861.4904	0.087227

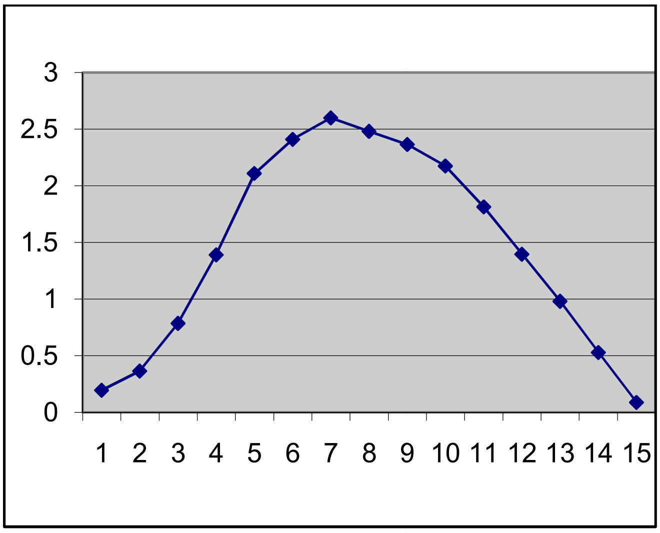

Figure 3.50 Unconfined compressive strength test for soil sample A with 30% gypsum-2

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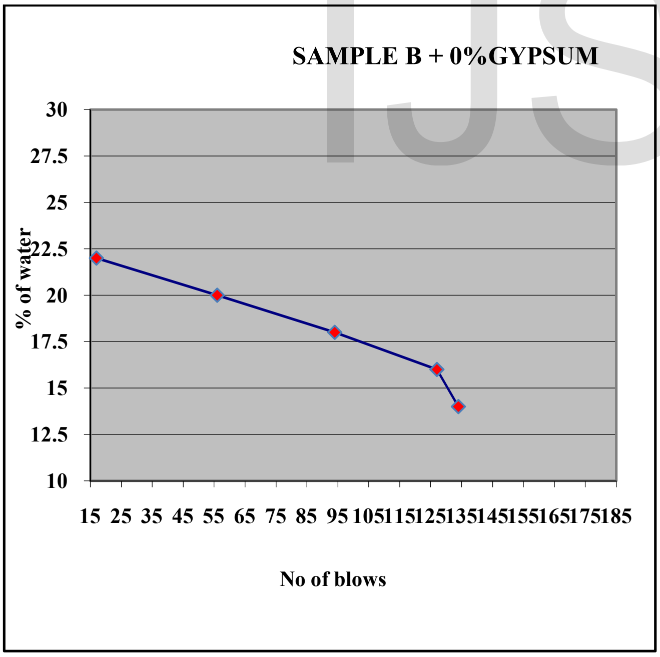
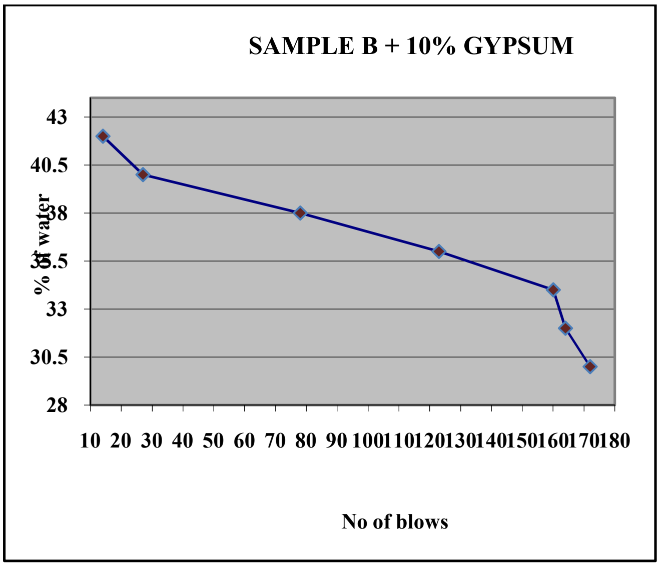
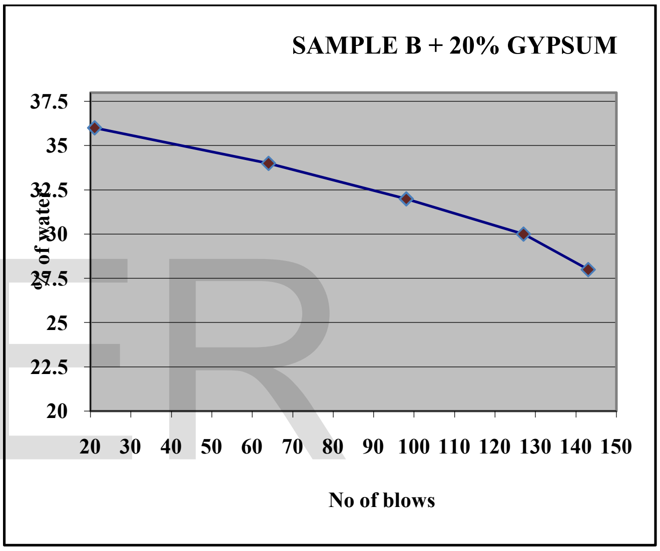
3.8.8 Liquid limit for sample B with gypsum in various proportions

Samples	Observations									Liquid limit
0% gypsum	No of Blows	180	154	121	98	61	45	27	18	62.4 %
	Water Content	50	52	54	56	58	60	62	64	
	Wt. of soil	100	100	100	100	100	100	100	100	
10% gypsum	No of Blows	169	160	157	121	75	28	18	-	43.46%
	Water Content	32	34	36	38	40	42	44	-	
	Wt. of soil	100	100	100	100	100	100	100	-	
20% gypsum	No of Blows	139	122	94	58	23	-	-	-	37.8 %
	Water Content	30	32	34	36	38	-	-	-	
	Wt. of soil	100	100	100	100	100	-	-	-	
30% gypsum	No of Blows	126	117	85	47	19	-	-	-	23.8 %
	Water Content	16	18	20	22	24	-	-	-	
	Wt. of soil	100	100	100	100	100	-	-	-	







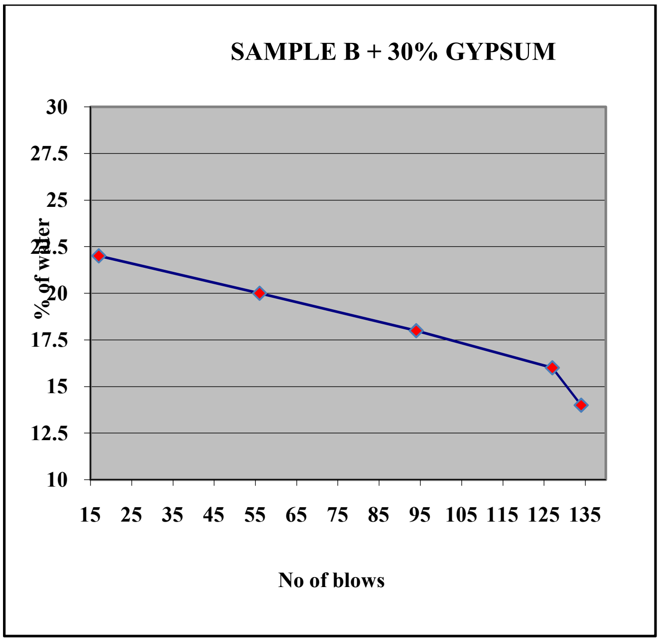


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Figure 3.51 liquid limit for soil sample B with gypsum in various proportions

3.8.9 Plastic limit for soil sample B with gypsum in various proportions

Table 3.64 Plastic limit for soil sample B with gypsum in various proportions

S.no	Description	0% gypsum	10% gypsum	20% gypsum	30% gypsum
1	Weight of can ,W1	36	36	24	32
2	Weight of can + wet soil ,W2	46	44	34	43
3	Weight of can + dry soil , W3	44	43	33	42
4	Weight of water, (W2-W3)	2	1	1	1
5	Weight of dry soil, (W3-W1)	8	7	9	10
6	Moisture content, (W2-W3)/(W3-W1) %	25 %	14.28 %	11.11 %	10 %

3.8.10 Plasticity index for soil sample B with gypsum in various proportions

Table 3.65 Plasticity index for soil sample B with gypsum in various proportions

Samples	Liquid limit (w_l) %	Plastic limit (w_p) %	Plasticity index (i_p) (w_l- w_p) %
0% gypsum	62.4	25	37.4
10% gypsum	43.46	14.28	29.18
20% gypsum	37.8	11.11	26.69
30% gypsum	23.8	10	13.8

3.8.11 Shrinkage limit for soil sample B with gypsum in various proportions

Table 3.66 shrinkage limit for soil sample B with gypsum in various proportions

S.no	Determination no.	0% gyp sum	10% gyp sum	20% gyp sum	30% gyp sum
1	Wt. of container in gm,W_1	183	154	157	162
2	Wt. of container + wet soil pat in gm,W_2	229	207	205	212
3	Wt. of container + dry soil pat in gm,W_3	213	192	192	187
4	Wt. of wet soil, W4= W_2 - W_1	46	53	48	50
5	Wt. of dry soil, W5= W_3 – W_1	30	38	35	25
6	Wt. of container + mercury filling dish, W_6 in gm	588	584	525	521
7	Wt of mercury filling dish W7= W_6 – W_1	405	430	368	359
8	Wt. of dish + mercury after displayed by dry pat W8 gms	250	252	253	249
9	Wt. Of mercury displayed by dry pat, W9=W6-W8	338	332	272	272
10	Volume of wet soil pat V1, in cm^3	29.8	31.6	27.06	26.39
11	Volume of dry soil pat V2, in cm^3	24.9	24.4	20	18.3
12	Shrinkage limit (W_S)	37	21	17	17
13	Shrinkage ratio (R)	1.205	1.557	1.75	1.366
14	Volumetric shrinkage VS	0.1968	0.2949	0.354	0.442

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3.8.12 Standard proctor compaction test for soil sample B with gypsum in various proportions

Table 3.67 Standard proctor compaction test for soil sample B with 0%gypsum

Standard proctor compaction test - sample B (0%gypsum)					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density (ϱ) g/cc	Dry Density(ϱd) g/cc
6	6178	4446	1732	1.709385	1.612627
8	6253	4446	1807	1.783406	1.651301
10	6402	4446	1956	1.93046	1.754964
12	6534	4446	2088	2.060736	1.839943
14	6560	4446	2114	2.086397	1.830173
16	6504	4446	2058	2.031128	1.750973

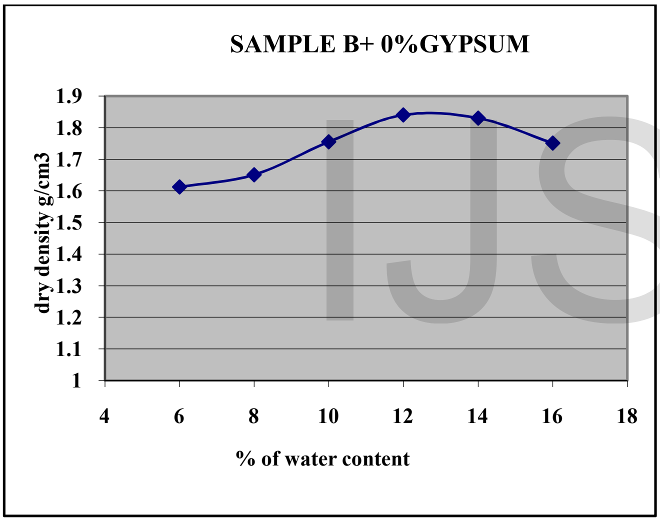

Figure 3.52 liquid limit for soil sample B with gypsum in various proportions

Table 3.68 Standard proctor compaction test for soil sample B with 10%gypsum

Standard Proctor Compaction Test - Sample B (10 % Gypsum)					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ϱ) g/cc	Dry Density(ϱd) g/cc
6	6065	4446	1619	1.59786	1.507415
8	6132	4446	1686	1.663985	1.540727
10	6213	4446	1767	1.743928	1.585389
12	6289	4446	1843	1.818935	1.62405
14	6348	4446	1902	1.877165	1.646636
16	6391	4446	1945	1.919604	1.654831
18	6300	4446	1854	1.829792	1.550671

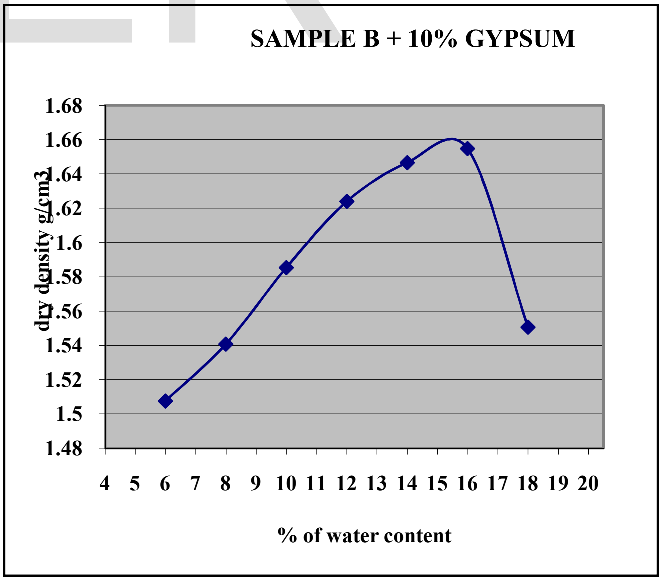

Figure 3.53 Liquid limit for soil sample B with gypsum in various proportions

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Table 3.69 Standard proctor compaction test for soil sample B with 20%gypsum

Standard Proctor Compaction Test - Sample B (20 % Gypsum)					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ϱ) g/cc	Dry Density(ϱ_d) g/cc
6	6027	4446	1581	1.560356	1.472034
8	6117	4446	1671	1.649181	1.52702
10	6272	4446	1826	1.802157	1.638325
12	6390	4446	1944	1.918617	1.713051
14	6440	4446	1994	1.967964	1.726284
16	6482	4446	2036	2.009415	1.732255
18	6459	4446	2013	1.986716	1.683657

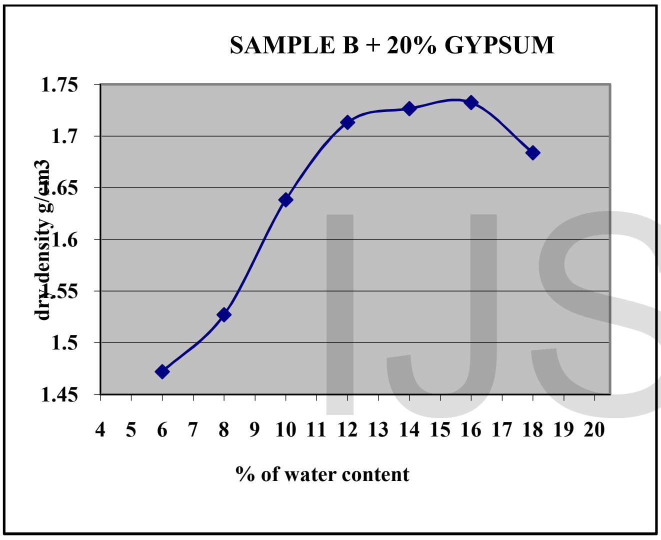

Figure 3.54 Liquid limit for soil sample B with gypsum in various proportions

Table 3.70 Standard proctor compaction test for soil sample B with 20%gypsum

Standard Proctor Compaction Test - Sample B (30 % Gypsum)					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ϱ) g/cc	Dry Density (ϱ_d) g/cc
6	6497	4552	1945	1.919604	1.810947
8	6589	4552	2037	2.010402	1.861484
10	6699	4552	2147	2.118966	1.926333
12	6593	4552	2041	2.01435	1.798527

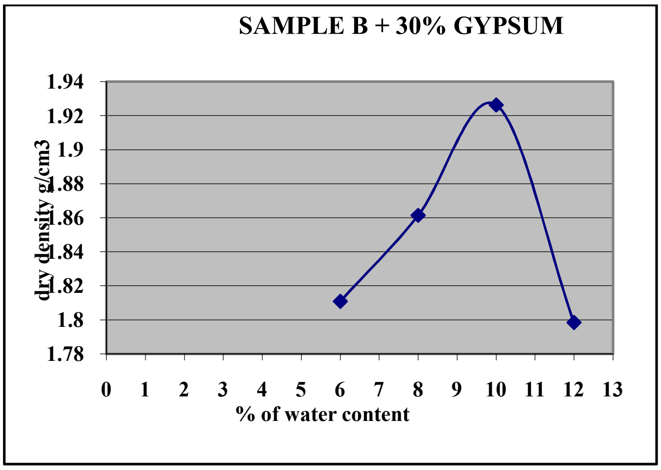

Figure 3.55 Liquid limit for soil sample B with gypsum in various proportions

Table 3.71 Maximum dry density and OMC value for sample B

Sample and gypsum proportions	MDD (g/cm³)	OMC %
Sample B+ 0% gypsum	1.839943	12
Sample B+ 10% gypsum	1.654831	16
Sample B+ 20% gypsum	1.732255	16
Sample B+ 30% gypsum	1.926333	10

3.8.13 CBR test for soil sample B with gypsum in various proportions

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Table 3.72 CBR test for soil sample B with 0% gypsum

Sample B + 0%Gypsum										
Sl No.	Un soaked					Soaked				
	Penetration		Load			Penetration		Load		
	Div	mm	Div	N	kg	Div	mm	Div	N	kg
1	50	0.5	26	735.94	73.59	50	0.5	7	198.14	19.81
2	100	1	34	962.39	96.23	100	1	16	452.89	45.28
3	150	1.5	40	1132.22	113.22	150	1.5	20	566.11	56.61
4	200	2	44	1245.44	124.54	200	2	22	622.72	62.27
5	250	2.5	50	1415.28	141.52	250	2.5	23	651.03	65.10
6	300	3	64	1811.55	181.15	300	3	24	679.33	67.93
7	350	3.5	70	1981.39	198.13	350	3.5	25	707.64	70.76
8	400	4	82	2321.05	232.10	400	4	26	735.94	73.59
9	450	4.5	90	2547.50	254.74	450	4.5	28	792.55	79.25
10	500	5	102	2887.16	288.71	500	5	30	849.17	84.91
11	550	5.5	104	2943.77	294.37	550	5.5	31	877.47	87.74
12	600	6	108	3056.99	305.69	600	6	33	934.08	93.40
13	650	6.5	116	3283.44	328.34	650	6.5	34	962.39	96.23
14	700	7	118	3340.05	334.00	700	7	34	962.39	96.23
15	750	7.5	120	3396.66	339.66	750	7.5	35	990.69	99.06
16	800	8	122	3453.27	345.32	800	8	35	990.69	99.06
17	850	8.5	124	3509.88	350.98	850	8.5	36	1019.00	101.89
18	900	9	126	3566.49	356.64	900	9	36	1019.00	101.89
19	950	9.5	128	3623.10	362.31	950	9.5	37	1047.30	104.73
20	1000	10	130	3679.72	367.97	1000	10	37	1047.30	104.73

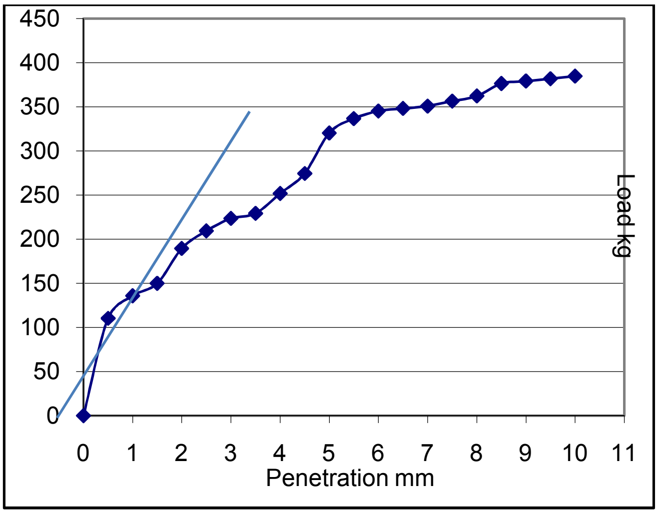

Figure 3.56 CBR test for soil sample B with 0% gypsum

CBR

2.5 mm	141	10.33724
5 mm	289	14.15973

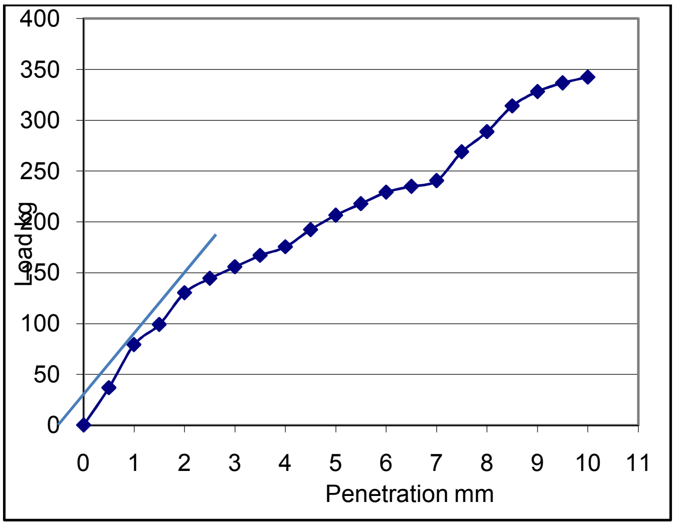

Figure 3.57 CBR test for soil sample B with 0% gypsum

CBR

2.5 mm	57	4.160584
5 mm	84	4.087591

Table 3.73 CBR test for soil sample B with 10% gypsum

Sample B + 10% Gypsum										
Sl No.	Un soaked					Soaked				
	Penetration		Load			Penetration		Load		
	Div	mm	Div	N	kg	Div	mm	Div	N	kg
1	50	0.5	22	622.72	62.27	50	0.5	6	169.83	16.9833
2	100	1	28	792.55	79.25	100	1	9	254.75	25.47495
3	150	1.5	33	934.08	93.40	150	1.5	13	367.97	36.79715
4	200	2	45	1273.75	127.37	200	2	19	537.80	53.78045
5	250	2.5	51	1443.58	144.35	250	2.5	25	707.64	70.76375
6	300	3	59	1670.02	167.00	300	3	29	820.86	82.08595
7	350	3.5	70	1981.39	198.13	350	3.5	32	905.78	90.5776
8	400	4	79	2236.13	223.61	400	4	34	962.39	96.2387
9	450	4.5	93	2632.41	263.24	450	4.5	36	1019.00	101.8998
10	500	5	98	2773.94	277.39	500	5	37	1047.30	104.7304
11	550	5.5	111	3141.91	314.19	550	5.5	39	1103.91	110.3915
12	600	6	120	3396.66	339.66	600	6	39	1103.91	110.3915
13	650	6.5	132	3736.33	373.63	650	6.5	40	1132.22	113.222
14	700	7	141	3991.08	399.10	700	7	40	1132.22	113.222
15	750	7.5	149	4217.52	421.75	750	7.5	41	1160.53	116.0526
16	800	8	158	4472.27	447.22	800	8	41	1160.53	116.0526
17	850	8.5	159	4500.57	450.05	850	8.5	42	1188.83	118.8831
18	900	9	160	4528.88	452.88	900	9	42	1188.83	118.8831
19	950	9.5	162	4585.49	458.54	950	9.5	43	1217.14	121.7137
20	1000	10	164	4613.80	461.37	1000	10	43	1217.14	121.7137

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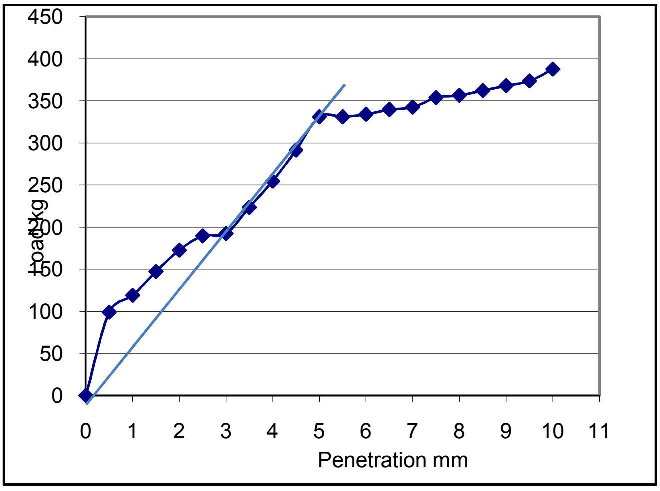

Figure 3.58 CBR test for soil sample B with 10% gypsum-Un soaked

CBR

2.5 mm	144	10.55718
5 mm	277	13.57178

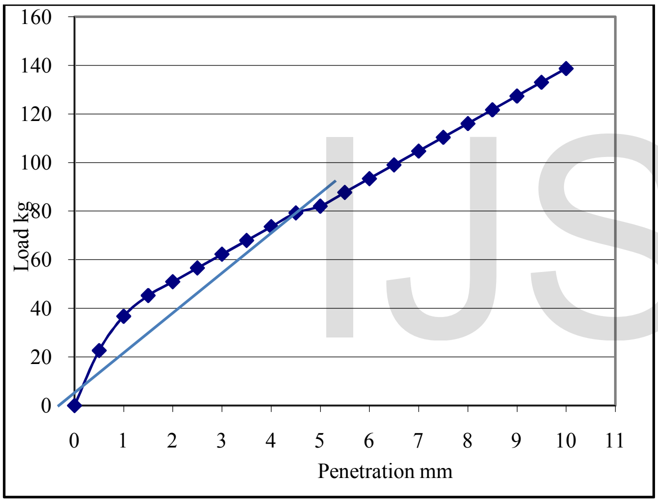

Figure 3.59 CBR test for soil sample B with 10% gypsum- Soaked

CBR

2.5 mm	71	5.182482
5 mm	105	5.109489

Table 3.74 CBR test for soil sample B with 20% gypsum

Sample B + 20% Gypsum										
Sl No.	Un soaked					Soaked				
	Penetration		Load			Penetration		Load		
	Div	mm	Div	N	kg	Div	mm	Div	N	kg
1	50	0.5	24	679.33	67.9332	50	0.5	7	198.14	19.81385
2	100	1	30	849.17	84.9165	100	1	12	339.67	33.9666
3	150	1.5	34	962.39	96.2387	150	1.5	21	594.42	59.44155
4	200	2	44	1245.44	124.5442	200	2	27	764.25	76.42485
5	250	2.5	53	1500.19	150.0192	250	2.5	35	990.69	99.06925
6	300	3	62	1754.94	175.4941	300	3	41	1160.53	116.0526
7	350	3.5	70	1981.39	198.1385	350	3.5	46	1302.05	130.2053
8	400	4	81	2292.75	229.2746	400	4	49	1386.97	138.697
9	450	4.5	92	2604.11	260.4106	450	4.5	51	1443.58	144.3581
10	500	5	104	2943.77	294.3772	500	5	52	1471.89	147.1886
11	550	5.5	122	3453.27	345.3271	550	5.5	54	1528.50	152.8497
12	600	6	131	3708.02	370.8021	600	6	56	1585.11	158.5108
13	650	6.5	138	3906.16	390.6159	650	6.5	57	1613.41	161.3414
14	700	7	144	4075.99	407.5992	700	7	59	1670.02	167.0025
15	750	7.5	151	4274.13	427.4131	750	7.5	61	1726.64	172.6636
16	800	8	153	4330.74	433.0742	800	8	62	1754.94	175.4941
17	850	8.5	160	4528.88	452.888	850	8.5	63	1783.25	178.3247
18	900	9	161	4557.19	455.7186	900	9	64	1811.55	181.1552
19	950	9.5	163	4613.80	461.3797	950	9.5	65	1839.86	183.9858
20	1000	10	164	4642.10	464.2102	1000	10	66	1868.16	186.8163

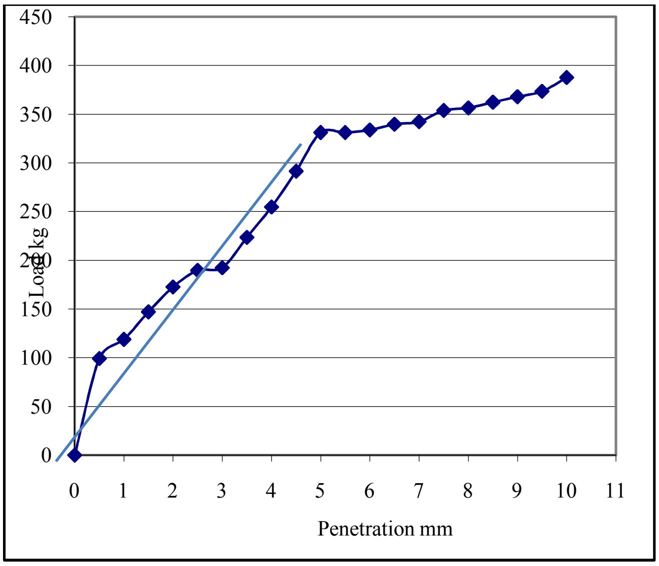

Figure 3.60 CBR test for soil sample B with 20% gypsum – Unsoaked

CBR

2.5 mm	150	10.99707
5 mm	294	14.4047

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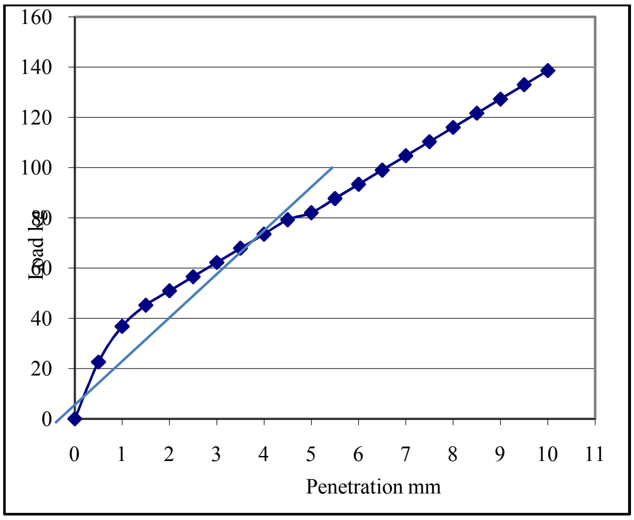

Figure 3.61 CBR test for soil sample B with 20% gypsum – Soaked

CBR		
2.5 mm	99	7.226277
5 mm	147	7.153285

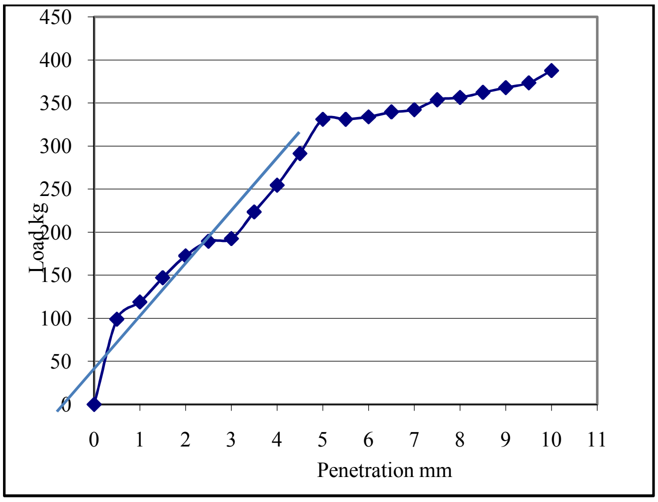

Figure 3.62 CBR test for soil sample B with 30% gypsum – Un soaked

CBR		
2.5 mm	161	11.80352
5 mm	300	14.69868

Table 3.75 CBR test for soil sample B with 30% gypsum

Sample B + 30% Gypsum										
Sl No.	Un soaked					Soaked				
	Penetration		Load			Penetration		Load		
	Div	mm	Div	N	kg	Div	mm	Div	N	kg
1	50	0.5	26	735.94	73.5943	50	0.5	11	311.36	31.13605
2	100	1	32	905.78	90.5776	100	1	19	537.80	53.78045
3	150	1.5	38	1075.61	107.5609	150	1.5	28	792.55	79.2554
4	200	2	46	1302.05	130.2053	200	2	36	1019.00	101.8998
5	250	2.5	57	1613.41	161.3414	250	2.5	45	1273.75	127.3748
6	300	3	63	1783.25	178.3247	300	3	54	1528.50	152.8497
7	350	3.5	72	2038.00	203.7996	350	3.5	59	1670.02	167.0025
8	400	4	82	2321.05	232.1051	400	4	63	1783.25	178.3247
9	450	4.5	93	2632.41	263.2412	450	4.5	65	1839.86	183.9858
10	500	5	106	3000.38	300.0383	500	5	67	1896.47	189.6469
11	550	5.5	124	3509.88	350.9882	550	5.5	69	1953.08	195.308
12	600	6	132	3736.33	373.6326	600	6	71	2009.69	200.9691
13	650	6.5	140	3962.77	396.277	650	6.5	73	2066.30	206.6302
14	700	7	146	4132.60	413.2603	700	7	74	2094.61	209.4607
15	750	7.5	153	4330.74	433.0742	750	7.5	75	2122.91	212.2913
16	800	8	157	4443.96	444.3964	800	8	76	2151.22	215.1218
17	850	8.5	160	4528.88	452.888	850	8.5	77	2179.52	217.9524
18	900	9	162	4585.49	458.5491	900	9	77	2179.52	217.9524
19	950	9.5	164	4642.10	464.2102	950	9.5	78	2207.83	220.7829
20	1000	10	166	4698.71	469.8713	1000	10	78	2207.83	220.7829

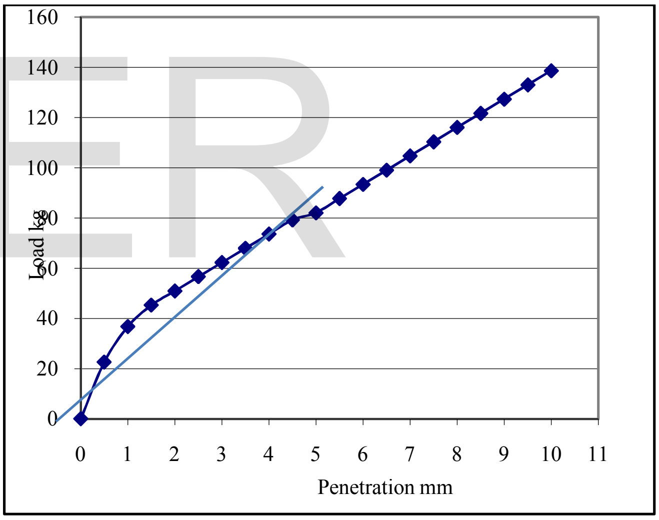

Figure 3.63 CBR test for soil sample B with 30% gypsum – Soaked

CBR		
2.5 mm	127	9.270073
5 mm	189	9.19708

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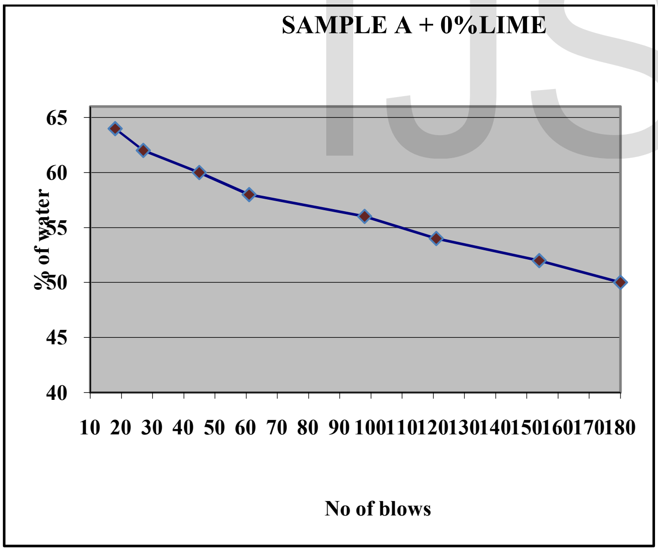
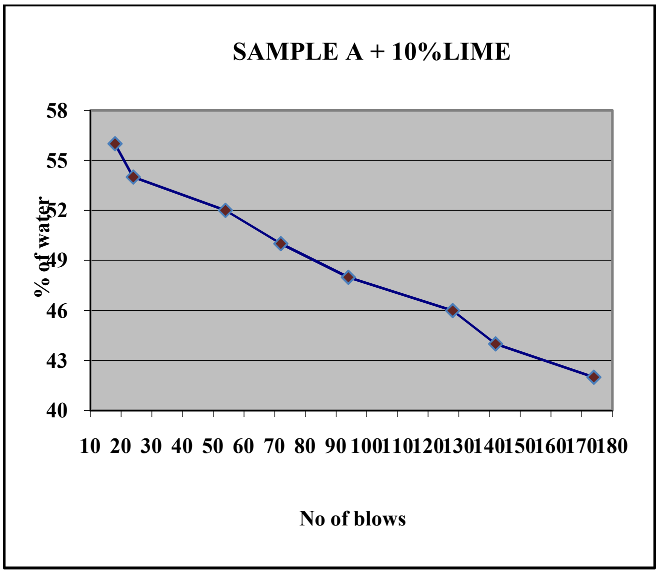
3.9 SOIL SAMPLE A WITH LIME IN VARIOUS PROPORTIONS

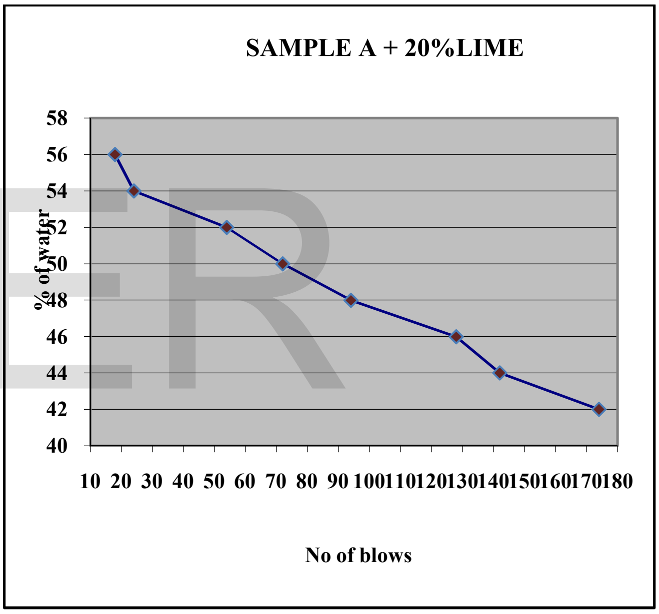
3.9.1 Liquid limit for soil sample A with lime in various proportions

Table 3.76 Liquid limit for soil sample A with lime in various proportions

Samples	Observations									Liquid limit
0% lime	No of blows	180	154	121	98	61	45	27	18	62.4%
	Water content	50	52	54	56	58	60	62	64	
	Wt. Of soil	100	100	100	100	100	100	100	100	
10% lime	No of blows	178	142	128	94	72	54	24	18	54%
	Water content	42	44	46	48	50	52	54	56	
	Wt. Of soil	100	100	100	100	100	100	100	100	
20% lime	No of blows	182	140	110	86	54	32	21	18	53%
	Water content	40	42	44	46	48	50	52	54	
	Wt. Of soil	100	100	100	100	100	100	100	100	
30% lime	No of blows	142	121	78	28	17	-	-	-	37%
	Water content	30	32	34	36	38	-	-	-	
	Wt. Of soil	100	100	100	100	100	-	-	-	







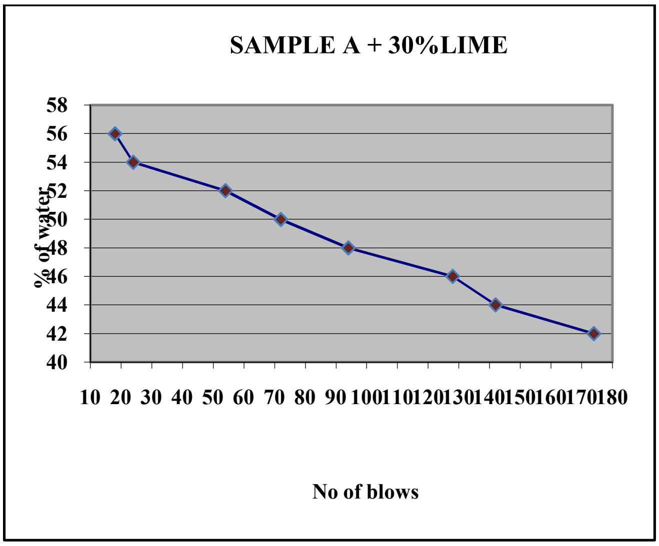


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Figure 3.64 liquid limit for soil sample A with lime in various proportions

3.9.2 Plastic limit for soil sample A with lime in various proportions

Table 3.77 Plastic limit for soil sample A with lime in various proportions

S.no	Description	0% lime	10% lime	20% lime	30% lime
1	Weight of can ,W1	36	23	36	25
2	Weight of can + wet soil ,W2	46	33	46	35
3	Weight of can + dry soil , W3	43	30	43	33
4	Weight of water, (W2-W3)	3	3	3	2
5	Weight of dry soil, (W3-W1)	7	7	7	8
6	Moisture content,	42.857 %	42.857	42.857	25

3.9.3 Plasticity Index for soil sample A with lime in various proportions

Table 3.78 Plasticity Index for soil sample A with lime in various proportions

Samples	Liquid limit (w_l) %	Plastic limit (w_p) %	Plasticity index (i_p) (w_l- w_p) %
0% lime	62.4	42.857	19.543
10% lime	54	42.857	11.143
20% lime	53	42.857	10.143
30% lime	37	25	12

3.9.4 Shrinkage limit for soil sample A with lime in various proportions

Table 3.79 shrinkage limit for soil sample A with lime in various proportions

	Determination no.	0% lime	10% lime	20% lime	30% lime
1	Wt. of container in gm,W_1	183	183	154	157
2	Wt. of container + wet soil pat in gm,W_2	229	223	207	205
3	Wt. of container + dry soil pat in gm,W_3	213	211	192	192
4	Wt. of wet soil, W4= W_2 - W_1	46	40	53	48
5	Wt. of dry soil, W5= W_3 – W_1	30	28	38	35
6	Wt. of container + mercury filling dish, W_6 in gm	588	590	584	525
7	Wt of mercury filling dish W7= W_6 – W_1	405	407	430	368
8	Wt. of dish + mercury after displayed by dry pat W8 gms	250	304	252	253
9	Wt. Of mercury displayed by dry pat, W9=W6-W8	338	286	332	272
10	Volume of wet soil pat V1in cm^3	29.8	29.9	31.6	27.06
11	Volume of dry soil pat V2, in cm^3	24.9	21.03	24.4	20
12	Shrinkage limit (W_S)	37	11.17	20.53	16.9
13	Shrinkage ratio (R) = W5 / V2	1.205	1.331	1.557	1.75
14	Volumetric shrinkage VS	0.1968	0.4217	0.2949	0.354

3.9.5 Differential free swell index for soil sample A

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with lime in various proportions

Table 3.80 Differential free swell index for soil sample A with lime in various proportions

S.no	Observation	0% lime	10% lime	20% lime	30% lime
1	Volume of the soil in kerosene after swelling, V1 ml	21	21	21	21
2	Volume of soil in water after swelling,V2 ml	32	33	34	35
3	The free swell index of the soil (%) [(V1-V2)/V1] X 100 %	52.38 %	57.14 %	61.9 %	66.67 %
Degree of expansiveness of soil: Since, The Free swell index is greater than 50.		very high	very high	very high	very high

3.9.6 Standard proctor compaction test for soil sample A with lime in various proportions

Table 3.81 Standard proctor compaction test for soil sample A with 0% lime

Standard Proctor Compaction Test - Sample A					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ρ) g/cc	Dry Density(ρ_d) g/cc
6	5907	4505	1402	1.383694	1.305371
8	5986	4505	1481	1.461662	1.353391
10	6036	4505	1531	1.511009	1.373645
12	6114	4505	1609	1.587991	1.417849
14	6202	4505	1697	1.674842	1.46916
16	6234	4505	1729	1.706424	1.471055
18	6241	4505	1736	1.713333	1.451977
20	6252	4505	1747	1.724189	1.436824
22	6243	4505	1738	1.715306	1.405989

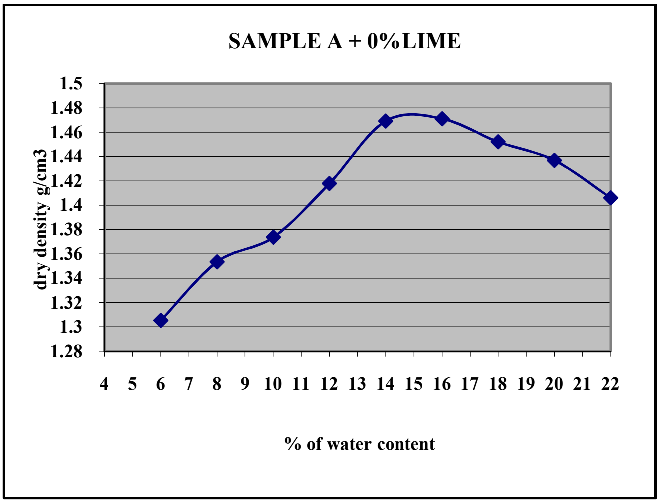

Figure 3.65 Standard proctor compaction test for soil sample A with 0% lime

Table 3.82 Standard proctor compaction test for soil sample A with 10% lime

Standard proctor compaction test - sample A-10%lime					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ρ) g/cc	Dry Density(ρ_d) g/cc
6	5814	4446	1368	1.350138	1.273715
8	5934	4446	1488	1.468571	1.359788
10	6102	4446	1656	1.634377	1.485797
12	6149	4446	1703	1.680763	1.500682
14	6183	4446	1737	1.71432	1.503789
16	6243	4446	1797	1.773536	1.52891
18	6298	4446	1852	1.827818	1.548998
20	6435	4446	1989	1.963029	1.635858
22	6404	4446	1958	1.932434	1.583962
24	6364	4446	1918	1.892956	1.526578

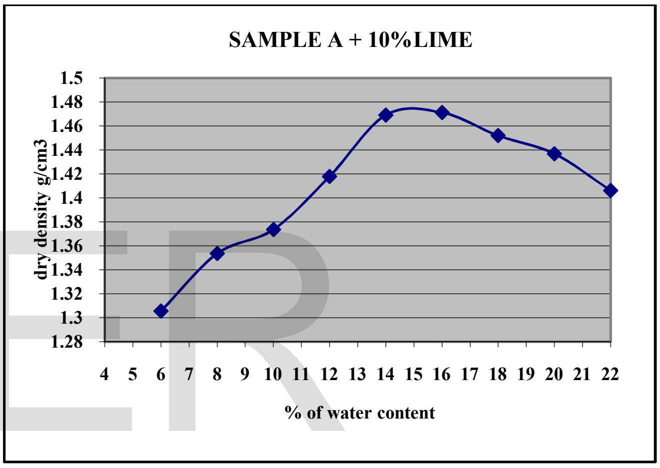

Figure 3.66 Standard proctor compaction test for soil sample A with 10% lime

Table 3.83 Standard proctor compaction test for soil sample A with 20% lime

Standard proctor compaction test – sample A-20%lime					
Water content %	Wt of mould + Soil (g)	Empty wt. Of mould (g)	Wt of compacted soil (g)	Wet Density(ρ) g/cc	Dry Density(ρ_d) g/cc
6	5978	4446	1532	1.511996	1.426412
8	6067	4446	1621	1.599834	1.481328
10	6207	4446	1761	1.738006	1.580006
12	6274	4446	1828	1.804131	1.610832
14	6308	4446	1862	1.837687	1.612006
16	6384	4446	1938	1.912695	1.648875
18	6435	4446	1989	1.963029	1.663584
20	6489	4446	2043	2.016324	1.68027
22	6463	4446	2017	1.990664	1.631691
24	6364	4446	1918	1.892956	1.526578

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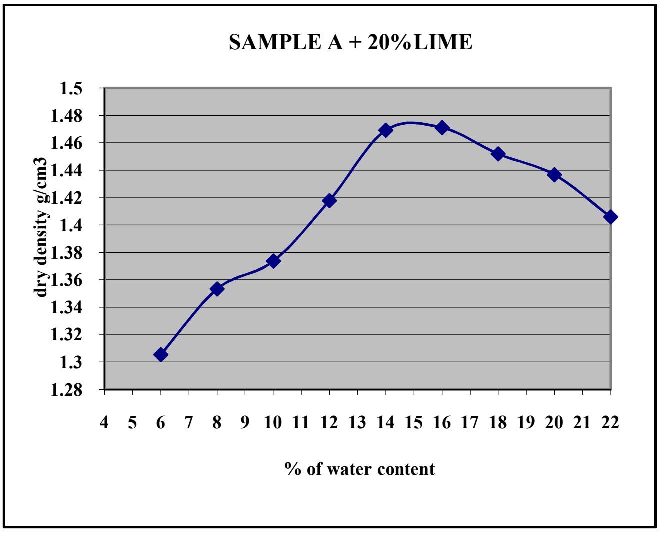

Figure 3.67 Standard proctor compaction test for soil sample A with 20% lime

Table 3.84 Standard proctor compaction test for soil sample A with 30% lime

Standard proctor compaction test - sample A-30%lime					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density(ϱ) g/cc	Dry Density(ϱd) g/cc
6	6034	4446	1588	1.567265	1.478552
8	6137	4446	1691	1.66892	1.545296
10	6289	4446	1843	1.818935	1.653578
12	6388	4446	1942	1.916643	1.711288
14	6459	4446	2013	1.986716	1.742733
16	6493	4446	2047	2.020272	1.741614
18	6449	4446	2003	1.976846	1.675293
20	6457	4446	2011	1.984742	1.653952

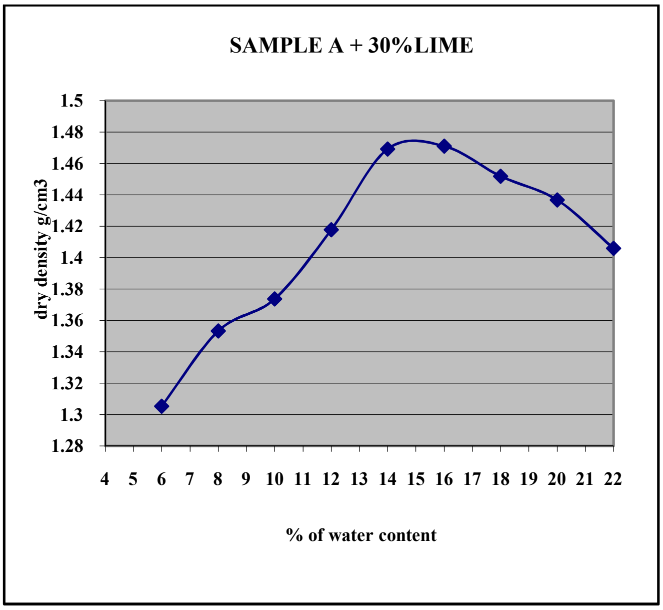

Figure 3.68 Standard proctor compaction test for soil sample A with 30% lime

Table 3.85 MDD and OMC value for sample A with lime in various proportions

Sample and flyash proportions	MDD (g/cm³)	OMC %
Sample A+ 0% lime	1.471055	16
Sample A+ 10% lime	1.635858	20
Sample A+ 20% lime	1.68027	20
Sample A+ 30% lime	1.742733	14

3.9.7 Unconfined compressive strength test for soil sample A with lime in various proportions

Table 3.86 Unconfined compressive strength test for soil sample A with 0%lime

Unconfined compressive strength test- sample A+ 0% lime first specimen							
rea ding	dl	E(strain)	pro ving gauge rea ding	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	1.4	11.074	11.3354	12.13367	0.912667
100	1	0.1315789	3.6	28.476	11.3354	13.05288	2.181587
150	1.5	0.1973684	6.2	49.042	11.3354	14.12279	3.472542
200	2	0.2631579	8.4	66.444	11.3354	15.38376	4.319101
250	2.5	0.3289474	10	79.1	11.3354	16.89197	4.682699
300	3	0.3947368	10.8	85.428	11.3354	18.72805	4.561499
350	3.5	0.4605263	10.6	83.846	11.3354	21.01196	3.990394

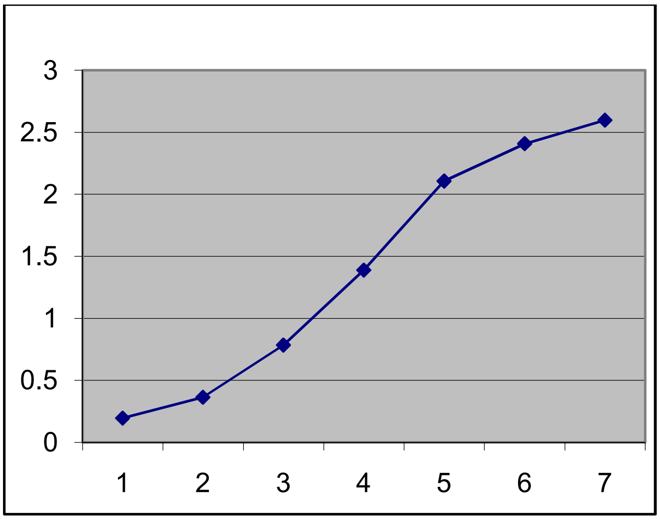

Figure 3.69 Unconfined compressive strength test for soil sample A with 0%lime-1

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Table 3.87 Unconfined compressive strength test for soil sample A with 0%lime-2

Unconfined compressive strength test- sample A second specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	0.2	1.582	11.3354	12.13367	0.130381
100	1	0.1315789	1.7	13.447	11.3354	13.05288	1.030194
150	1.5	0.1973684	4	31.64	11.3354	14.12279	2.24035
200	2	0.2631579	7.4	58.534	11.3354	15.38376	3.804922
250	2.5	0.3289474	10.2	80.682	11.3354	16.89197	4.776353
300	3	0.3947368	12	94.92	11.3354	18.72805	5.068333
350	3.5	0.4605263	13	102.83	11.3354	21.01196	4.893879
400	4	0.5263158	12.8	101.248	11.3354	23.93029	4.230956
450	4.5	0.5921053	12.4	98.084	11.3354	27.79001	3.529469

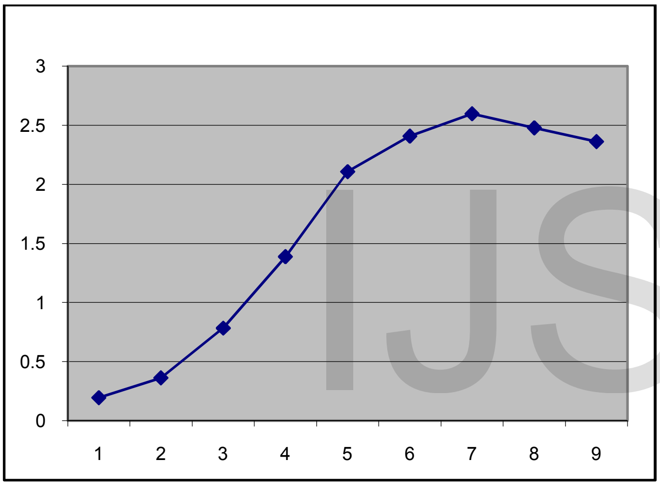

Figure 3.71Unconfined compressive strength test for soil sample A with 0% lime-1

Table 3.89 Unconfined compressive strength test for soil sample A with 0% lime-2

Unconfined compressive strength test- sample A 10%lime - second specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	4.2	33.222	11.3354	12.13367	2.738001
100	1	0.1315789	5.7	45.087	11.3354	13.05288	3.454179
150	1.5	0.1973684	7.3	57.743	11.3354	14.12279	4.088639
200	2	0.2631579	10.7	84.637	11.3354	15.38376	5.501712
250	2.5	0.3289474	16.9	133.679	11.3354	16.89197	7.913761
300	3	0.3947368	16.8	132.888	11.3354	18.72805	7.095666
350	3.5	0.4605263	16.8	132.888	11.3354	21.01196	6.324398
400	4	0.5263158	9.9	78.309	11.3354	23.93029	3.27238

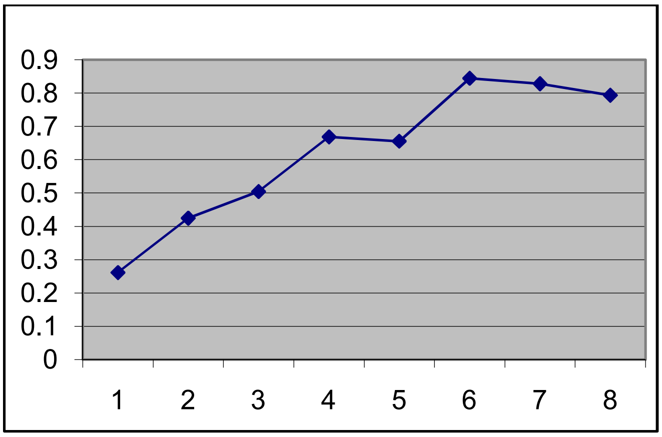

Figure 3.72 Unconfined compressive strength test for soil sample A with 10% lime-2

Table 3.90 Unconfined compressive strength test for soil sample A with 10% lime-1

Unconfined compressive strength test- sample A 20%lime – first specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	0.2	1.582	11.3354	12.13367	0.130381
100	1	0.1315789	0.6	4.746	11.3354	13.05288	0.363598
150	1.5	0.1973684	1.2	9.492	11.3354	14.12279	0.672105
200	2	0.2631579	1.6	12.656	11.3354	15.38376	0.822686
250	2.5	0.3289474	2	15.82	11.3354	16.89197	0.93654
300	3	0.3947368	2.4	18.984	11.3354	18.72805	1.013667
350	3.5	0.4605263	2.8	22.148	11.3354	21.01196	1.054066
400	4	0.5263158	3.1	24.521	11.3354	23.93029	1.024685
450	4.5	0.5921053	3.4	26.894	11.3354	27.79001	0.967758
500	5	0.6578947	3.6	28.476	11.3354	33.13425	0.859413
550	5.5	0.7236842	3.6	28.476	11.3354	41.02335	0.694141
600	6	0.7894737	2.8	22.148	11.3354	53.84315	0.411343

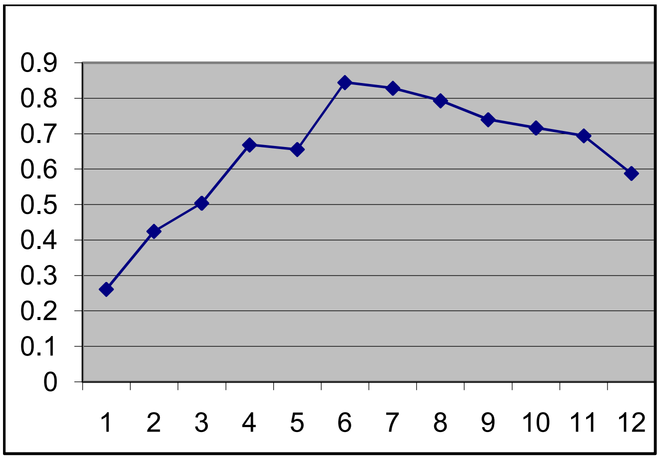

Figure 3.73 Unconfined compressive strength test for soil sample A with 20% lime-1

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Table 3.91 Unconfined compressive strength test for soil sample A with 20% lime-2

Unconfined compressive strength test- sample A 20% lime - second specimen							
rea ding	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	0.1	0.791	11.3354	12.13367	0.065191
100	1	0.1315789	0.5	3.955	11.3354	13.05288	0.302998
150	1.5	0.1973684	0.9	7.119	11.3354	14.12279	0.504079
200	2	0.2631579	1.4	11.074	11.3354	15.38376	0.71985
250	2.5	0.3289474	1.6	12.656	11.3354	16.89197	0.749232
300	3	0.3947368	2.1	16.611	11.3354	18.72805	0.886958
350	3.5	0.4605263	2.4	18.984	11.3354	21.01196	0.903485
400	4	0.5263158	2.9	22.939	11.3354	23.93029	0.958576
450	4.5	0.5921053	3.4	26.894	11.3354	27.79001	0.967758
500	5	0.6578947	3.4	26.894	11.3354	33.13425	0.811668
550	5.5	0.7236842	3.3	26.103	11.3354	41.02335	0.636296
600	6	0.7894737	3.1	24.521	11.3354	53.84315	0.455415
650	6.5	0.8552632	4.8	37.968	11.3354	78.31731	0.484797
700	7	0.9210526	4.8	37.968	11.3354	143.5817	0.264435
750	7.5	0.9868421	4.8	37.968	11.3354	861.4904	0.044072
800	8	1.0526316	3.2	25.312	11.3354	-215.373	-0.11753

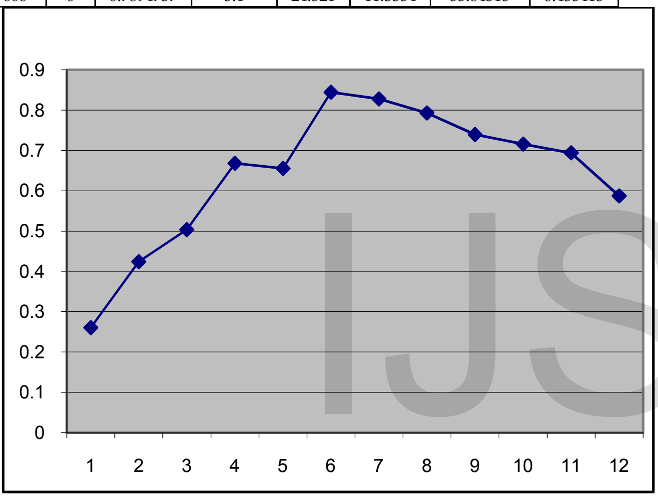

Figure 3.74 Unconfined compressive strength test for soil sample A with 20% lime-2

Table 3.92 Unconfined compressive strength test for soil sample A with 20% lime-1

Unconfined compressive strength test- sample A 30%lime - first specimen							
rea ding	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	0.2	1.582	11.3354	12.13367	0.130381
100	1	0.1315789	0.3	2.373	11.3354	13.05288	0.181799
150	1.5	0.1973684	0.8	6.328	11.3354	14.12279	0.44807
200	2	0.2631579	1.2	9.492	11.3354	15.38376	0.617014
250	2.5	0.3289474	1.6	12.656	11.3354	16.89197	0.749232
300	3	0.3947368	2.1	16.611	11.3354	18.72805	0.886958
350	3.5	0.4605263	2.4	18.984	11.3354	21.01196	0.903485
400	4	0.5263158	2.8	22.148	11.3354	23.93029	0.925522
450	4.5	0.5921053	3.2	25.312	11.3354	27.79001	0.910831
500	5	0.6578947	3.8	30.058	11.3354	33.13425	0.907158
550	5.5	0.7236842	4.2	33.222	11.3354	41.02335	0.809831
600	6	0.7894737	4.6	36.386	11.3354	53.84315	0.675778

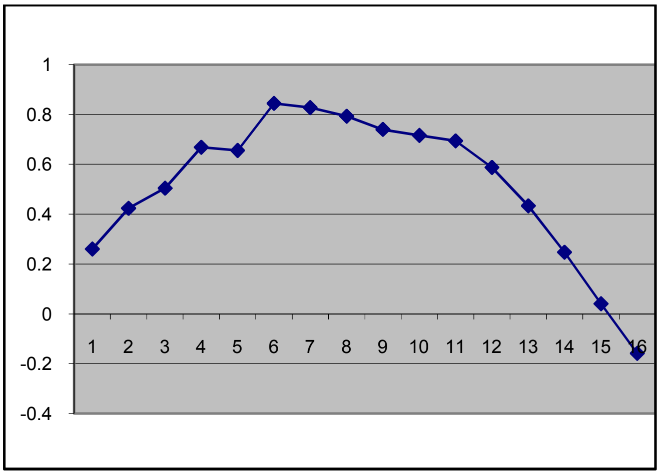

Figure 3.75 Unconfined compressive strength test for soil sample A with 20% lime-1

Table 3.93 Unconfined compressive strength test for soil sample A with 30% lime-2

Unconfined compressive strength test- sample A 30%lime - second specimen							
reading	dl	E(strain)	proving gauge reading	Load	Ao	Corrected Area	Stress
50	0.5	0.0657895	0.4	3.164	11.3354	12.13367	0.260762
100	1	0.1315789	0.7	5.537	11.3354	13.05288	0.424197
150	1.5	0.1973684	0.9	7.119	11.3354	14.12279	0.504079
200	2	0.2631579	1.3	10.283	11.3354	15.38376	0.668432
250	2.5	0.3289474	1.4	11.074	11.3354	16.89197	0.655578
300	3	0.3947368	2	15.82	11.3354	18.72805	0.844722
350	3.5	0.4605263	2.2	17.402	11.3354	21.01196	0.828195
400	4	0.5263158	2.4	18.984	11.3354	23.93029	0.793304
450	4.5	0.5921053	2.6	20.566	11.3354	27.79001	0.74005
500	5	0.6578947	3	23.73	11.3354	33.13425	0.716177
550	5.5	0.7236842	3.6	28.476	11.3354	41.02335	0.694141
600	6	0.7894737	4	31.64	11.3354	53.84315	0.587633

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650	6.5	0.8552632	4.3	34.013	11.3354	78.31731	0.434297
700	7	0.9210526	4.5	35.595	11.3354	143.5817	0.247908
750	7.5	0.9868421	4.5	35.595	11.3354	861.4904	0.041318
800	8	1.0526316	4.3	34.013	11.3354	-215.373	-0.15793

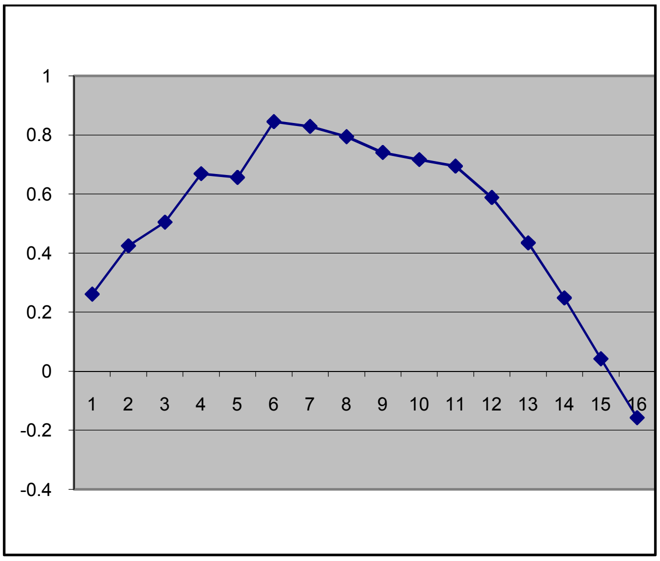


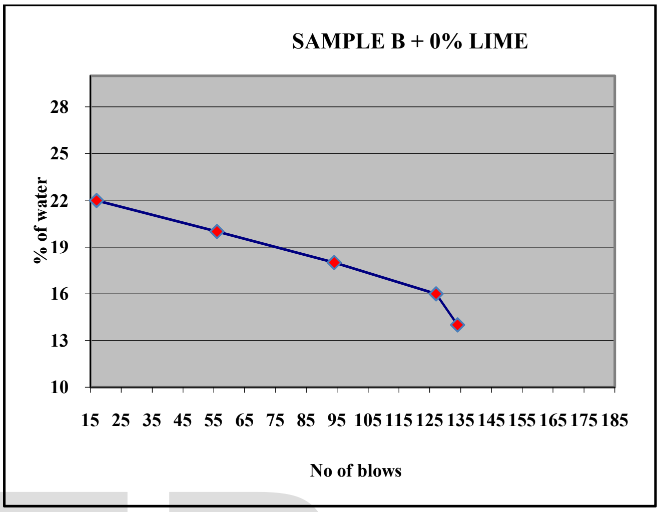
Figure 3.76 Unconfined compressive strength test for soil sample A with 30% lime-2

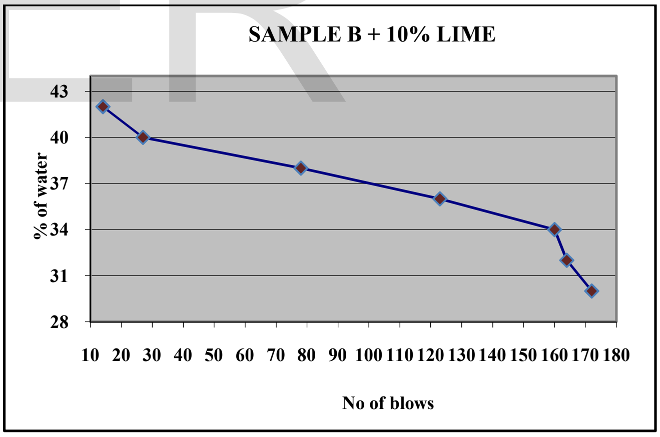
3.10 SOIL SAMPLE B WITH LIME IN VARIOUS PROPORTIONS

3.10.1 Liquid limit for sample B with lime in various proportions

Table 3.94 Liquid limit for sample B with lime in various proportions

Samples	Observations									Liquid limit (from graph)
0% lime	No of Blows	180	154	121	98	61	45	27	18	62.4%
	Water Content	50	52	54	56	58	60	62	64	
	Wt. of soil	100	100	100	100	100	100	100	100	
10% lime	No of Blows	172	164	160	123	78	27	14	-	41%
	Water Content	30	32	34	36	38	40	42	-	
	Wt. of soil	100	100	100	100	100	100	100	-	
20%	No of	14	12	98	64	21	-	-	-	35.64%
lime	Blows	3	7							
	Water Content	28	30	32	34	36	-	-	-	
	Wt. of soil	100	100	100	100	100	-	-	-	
30% lime	No of Blows	134	127	94	56	17	-	-	-	21.78%
	Water Content	14	16	18	20	22	-	-	-	
	Wt. of soil	100	100	100	100	100	-	-	-	





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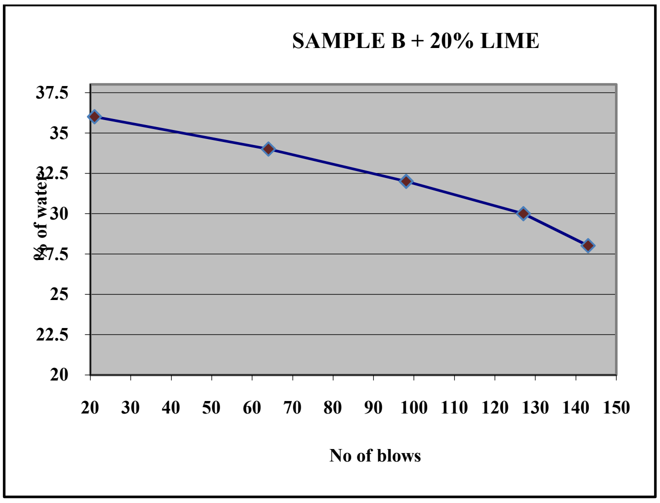

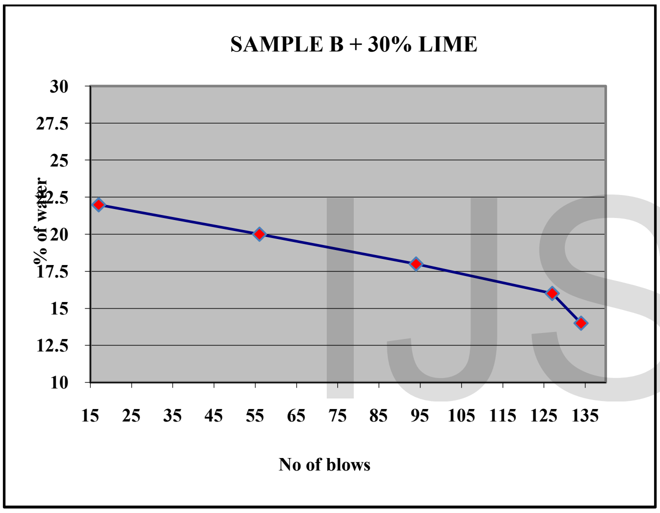

Figure 3.77 Liquid limit for soil sample B with lime in various proportions

3.10.2 Plastic Limit for sample B with lime in various proportions

Table 3.95 Plastic Limit for sample B with lime in various proportions

S.no	Description	0% lime	10% lime	20% lime	30% lime
1	Weight of can ,W1	36	36	24	32
2	Weight of can + wet soil ,W2	46	45	34	42
3	Weight of can + dry soil , W3	44	44	33	41
4	Weight of water, (W2-W3)	2	1	1	1
5	Weight of dry soil, (W3-W1)	8	8	9	9
6	Moisture content,	25 %	12.5 %	11.11 %	11.11 %

3.10.3 Plasticity Index for sample B with lime in various proportions

Table 3.96 Plasticity Index for sample B with lime in various proportions

Samples	Liquid limit (w_l) %	Plastic limit (w_p) %	Plasticity index (i_p) (w_l- w_p) %
0% lime	62.4	25	37.4
10% lime	41	12.5	28.5
20% lime	35.64	11.11	24.53
30% lime	21.78	11.11	10.67

3.10.4 Shrinkage limit for sample B with lime in various proportions

Table 3.97 Shrinkage limit for sample B with lime in various proportions

S.no	Determination no.	0% lime	10% lime	20% lime	30% lime
1	Wt. of container in gm,W_1	183	154	157	162
2	Wt. of container + wet soil pat in gm,W_2	229	207	205	212
3	Wt. of container + dry soil pat in gm,W_3	213	192	192	187
4	Wt. of wet soil, W4= W_2 - W_1	46	53	48	50
5	Wt. of dry soil, W5= W_3 – W_1	30	38	35	25
6	Wt. of container + mercury filling dish, W_6	588	584	525	521
7	Wt of mercury filling dish W7= W_6 – W_1	405	430	368	359
8	Wt. of dish + mercury after displayed by dry pat W8	250	252	253	249
9	Wt. Of mercury displayed by dry pat, W9	338	332	272	272
10	Volume of wet soil pat (V1=W7/13.6), in cm^3	29.8	31.6	27.06	26.39
11	Volume of dry soil pat (V2=W8/13.6), in cm^3	24.9	24.4	20	18.3
12	Shrinkage limit (W_s)	37	20.53	16.9	16.64
13	Shrinkage ratio (R) = W5 / V2	1.205	1.557	1.75	1.366
14	Volumetric shrinkage VS	0.1968	0.2949	0.354	0.442

3.10.5 CBR Test for soil sample B with lime in various proportions

Table 3.98 CBR Test for soil sample B with 0% lime

Sl No.	Un soaked					Soaked				
	Penetration		Load			Penetration		Load		
	Div	mm	Div	N	kg	Div	mm	Div	N	Kg
1	50	0.5	26	735.94	73.59	50	0.5	7	198.14	19.81
2	100	1	34	962.39	96.23	100	1	16	452.89	45.28
3	150	1.5	40	1132.22	113.22	150	1.5	20	566.11	56.61
4	200	2	44	1245.44	124.54	200	2	22	622.72	62.27
5	250	2.5	50	1415.28	141.52	250	2.5	23	651.03	65.10
6	300	3	64	1811.55	181.15	300	3	24	679.33	67.93
7	350	3.5	70	1981.39	198.13	350	3.5	25	707.64	70.76
8	400	4	82	2321.05	232.10	400	4	26	735.94	73.59
9	450	4.5	90	2547.50	254.74	450	4.5	28	792.55	79.25
10	500	5	102	2887.16	288.71	500	5	30	849.17	84.91
11	550	5.5	104	2943.77	294.37	550	5.5	31	877.47	87.74
12	600	6	108	3056.99	305.69	600	6	33	934.08	93.40
13	650	6.5	116	3283.44	328.34	650	6.5	34	962.39	96.23
14	700	7	118	3340.05	334.00	700	7	34	962.39	96.23

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15	750	7.5	120	3396.66	339.66	750	7.5	35	990.69	99.06
16	800	8	122	3453.27	345.32	800	8	35	990.69	99.06
17	850	8.5	124	3509.88	350.98	850	8.5	36	1019.00	101.89
18	900	9	126	3566.49	356.64	900	9	36	1019.00	101.89
19	950	9.5	128	3623.10	362.31	950	9.5	37	1047.30	104.73
20	1000	10	130	3679.72	367.97	1000	10	37	1047.30	104.73

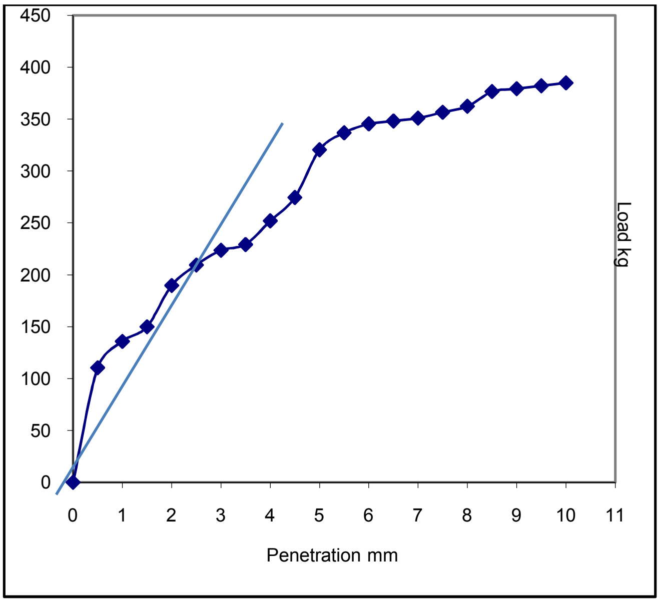

Figure 3.78 CBR Test for soil sample B with 0% lime-Un soaked

CBR		
2.5 mm	141	10.33724
5 mm	289	14.15973

Table 3.99 CBR Test for soil sample B with 10% lime

Sample B + 10% Lime										
Sl No.	Un soaked					Soaked				
	Penetration		Load			Penetration		Load		
	Div	mm	Div	N	kg	Div	mm	Div	N	Kg
1	50	0.5	36	1019.00	101.8998	50	0.5	11	311.36	31.136 5
2	100	1	44	1245.44	124.5442	100	1	17	481.19	48.119 5
3	150	1.5	55	1556.80	155.6803	150	1.5	22	622.72	62.272
4	200	2	64	1811.55	181.1552	200	2	26	735.94	73.594
5	250	2.5	72	2038.00	203.7996	250	2.5	30	849.17	84.916
6	300	3	78	2207.83	220.7829	300	3	34	962.39	96.238
7	350	3.5	84	2377.66	237.7662	350	3.5	38	1075.61	107.56 9
8	400	4	90	2547.50	254.7495	400	4	40	1132.22	113.22
9	450	4.5	95	2689.02	268.9023	450	4.5	42	1188.83	118.88 1
10	500	5	107	3028.69	302.8689	500	5	44	1245.44	124.54 2
11	550	5.5	109	3085.30	308.53	550	5.5	46	1302.05	130.20 3
12	600	6	112	3170.22	317.0216	600	6	48	1358.66	135.86 4
13	650	6.5	114	3226.83	322.6827	650	6.5	50	1415.28	141.52 5
14	700	7	117	3311.74	331.1744	700	7	52	1471.89	147.18 6
15	750	7.5	119	3368.35	336.8355	750	7.5	54	1528.50	152.84 7
16	800	8	120	3396.66	339.666	800	8	55	1556.80	155.68 3
17	850	8.5	125	3538.19	353.8188	850	8.5	57	1613.41	161.34 4
18	900	9	129	3651.41	365.141	900	9	59	1670.02	167.00 5
19	950	9.5	131	3708.02	370.8021	950	9.5	61	1726.64	172.66 6
20	1000	10	134	3792.94	379.2937	1000	10	63	1783.25	178.32 7

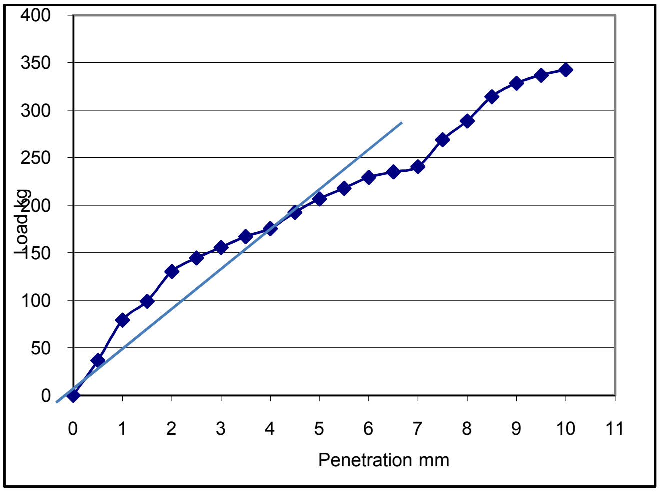

Figure 3.79 CBR Test for soil sample B with 0% lime -soaked

CBR		
2.5 mm	57	4.160584
5 mm	84	4.087591

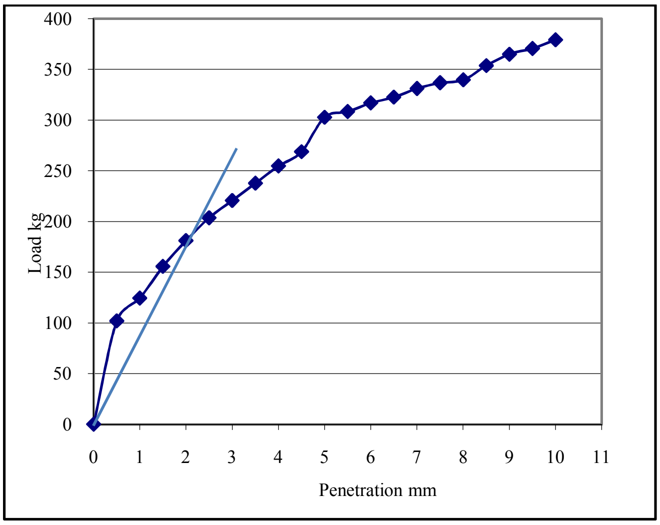

Figure 3.80 CBR Test for soil sample B with 10% lime – Un soaked

CBR		
2.5 mm	204	14.95601
5 mm	306	14.99667

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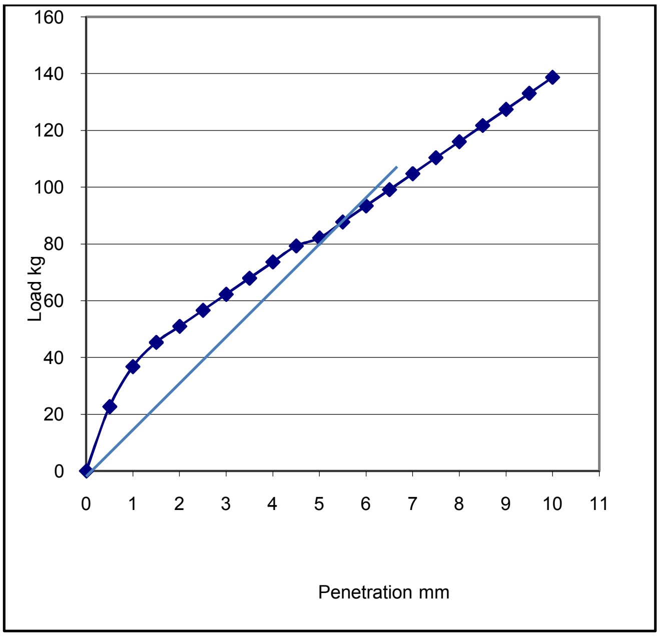

Figure 3.81 CBR Test for soil sample B with 10% lime - soaked

CBR
2.5 mm 84 6.131387
5 mm 125 6.082725

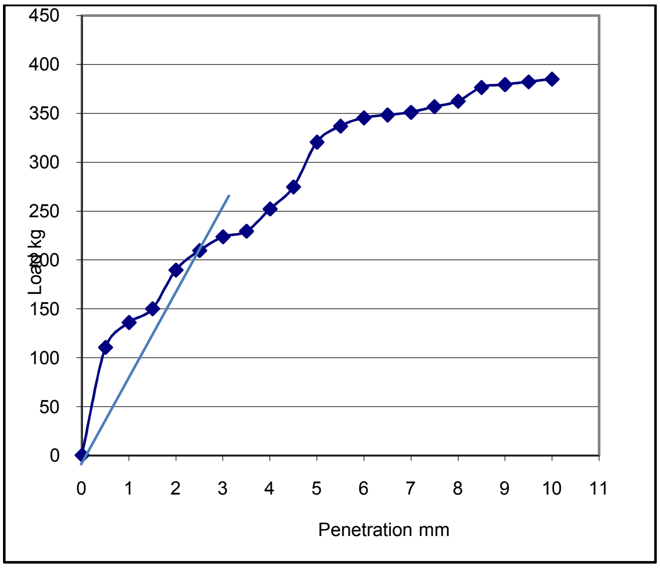

Figure 3.82 CBR Test for soil sample B with 20% lime – Un soaked

CBR
2.5 mm 207 15.17595
5 mm 311 15.23763

Table 3.100 CBR Test for soil sample B with 20% lime

SAMPLE B + 20% LIME										
Sl No.	Un soaked					Soaked				
	Penetration		Load			Penetration		Load		
	Div	mm	Div	N	kg	Div	mm	Div	N	Kg
1	50	0.5	38	1075.61	107.5609	50	0.5	11	311.36	31.13605
2	100	1	46	1302.05	130.2053	100	1	17	481.19	48.11935
3	150	1.5	49	1386.97	138.697	150	1.5	22	622.72	62.2721
4	200	2	66	1868.16	186.8163	200	2	26	735.94	73.5943
5	250	2.5	74	2094.61	209.4607	250	2.5	30	849.17	84.9165
6	300	3	78	2207.83	220.7829	300	3	34	962.39	96.2387
7	350	3.5	84	2377.66	237.7662	350	3.5	38	1075.61	107.5609
8	400	4	90	2547.50	254.7495	400	4	40	1132.22	113.222
9	450	4.5	95	2689.02	268.9023	450	4.5	42	1188.83	118.8831
10	500	5	110	3113.61	311.3605	500	5	44	1245.44	124.5442
11	550	5.5	119	3368.35	336.8355	550	5.5	46	1302.05	130.2053
12	600	6	120	3396.66	339.666	600	6	48	1358.66	135.8664
13	650	6.5	122	3453.27	345.3271	650	6.5	50	1415.28	141.5275
14	700	7	124	3509.88	350.9882	700	7	52	1471.89	147.1886
15	750	7.5	126	3566.49	356.6493	750	7.5	54	1528.50	152.8497
16	800	8	128	3623.10	362.3104	800	8	55	1556.80	155.6803
17	850	8.5	130	3679.72	367.9715	850	8.5	57	1613.41	161.3414
18	900	9	131	3708.02	370.8021	900	9	59	1670.02	167.0025
19	950	9.5	132	3736.33	373.6326	950	9.5	61	1726.64	172.6636
20	1000	10	134	3792.94	379.2937	1000	10	63	1783.25	178.3247

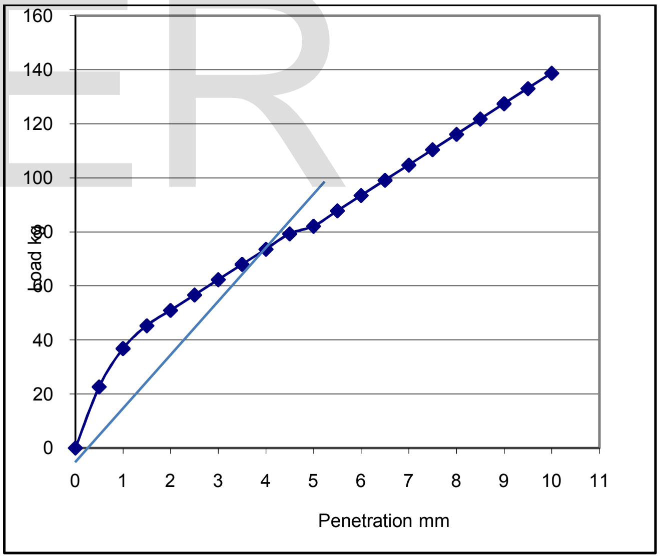

Figure 3.83 CBR Test for soil sample B with 20% lime - soaked

CBR
2.5 mm 116 8.467153
5 mm 173 8.418491

Table 3.101 CBR Test for soil sample B with 30% lime

Sample B + 30% Lime										
Sl No.	Un soaked					Soaked				
	Penetration		Load			Penetration		Load		
	Div	mm	Div	N	kg	Div	mm	Div	N	Kg
1	50	0.5	39	1103.91	110.3915	50	0.5	13	367.97	36.79 715

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2	100	1	48	1358.66	135.8664	100	1	21	594.42	59.44155
3	150	1.5	53	1500.19	150.0192	150	1.5	32	905.78	90.5776
4	200	2	67	1896.47	189.6469	200	2	44	1245.44	124.5442
5	250	2.5	74	2094.61	209.4607	250	2.5	52	1471.89	147.1886
6	300	3	79	2236.13	223.6135	300	3	59	1670.02	167.0025
7	350	3.5	81	2292.75	229.2746	350	3.5	65	1839.86	183.9858
8	400	4	89	2519.19	251.919	400	4	71	2009.69	200.9691
9	450	4.5	97	2745.63	274.5634	450	4.5	75	2122.91	212.2913
10	500	5	112	3170.22	320.436	500	5	78	2207.83	220.7829
11	550	5.5	119	3368.35	336.8355	550	5.5	81	2292.75	229.2746
12	600	6	122	3453.27	345.3271	600	6	84	2377.66	237.7662
13	650	6.5	123	3481.58	348.1577	650	6.5	86	2434.27	243.4273
14	700	7	124	3509.88	350.9882	700	7	88	2490.88	249.0884
15	750	7.5	126	3566.49	356.6493	750	7.5	90	2547.50	254.7495
16	800	8	128	3623.10	362.3104	800	8	92	2604.11	260.4106
17	850	8.5	133	3764.63	376.4632	850	8.5	92	2604.11	260.4106
18	900	9	134	3792.94	379.2937	900	9	93	2632.41	263.2412
19	950	9.5	135	3821.24	382.1243	950	9.5	93	2632.41	263.2412
20	1000	10	136	3849.55	384.9548	1000	10	94	2660.72	266.0717

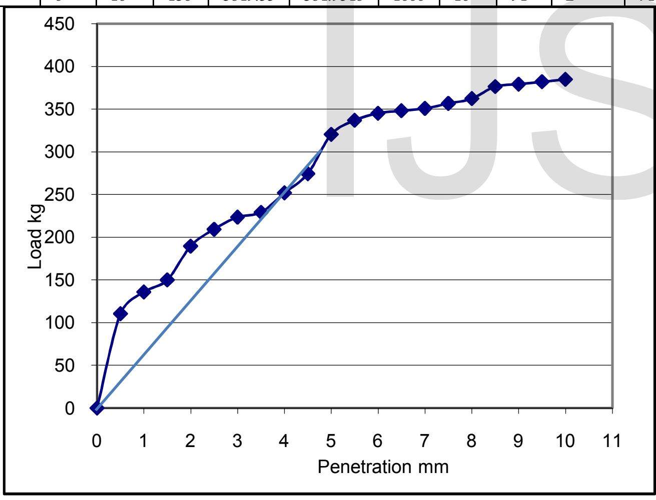

Figure 3.84 CBR Test for soil sample B with 30% lime – Un soaked

CBR

2.5 mm	210	15.39589
5 mm	320	15.67859

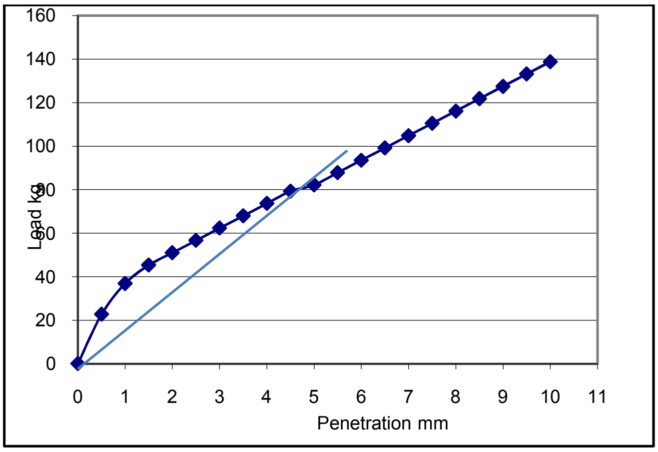

Figure 3.85 CBR Test for soil sample B with 30% lime - soaked

CBR

2.5 mm	147	10.72993
5 mm	220	10.7056

3.10.6 Standard proctor compaction test for sample B with lime in various proportions

Table 3.102 Standard proctor compaction test for sample B with 0% lime

Standard Proctor Compaction Test - Sample B (0%Lime)					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density (ρ) g/cc	Dry Density (ρ_d) g/cc
6	6178	4446	1732	1.709385	1.612627
8	6253	4446	1807	1.783406	1.651301
10	6402	4446	1956	1.93046	1.754964
12	6534	4446	2088	2.060736	1.839943
14	6560	4446	2114	2.086397	1.830173
16	6504	4446	2058	2.031128	1.750973

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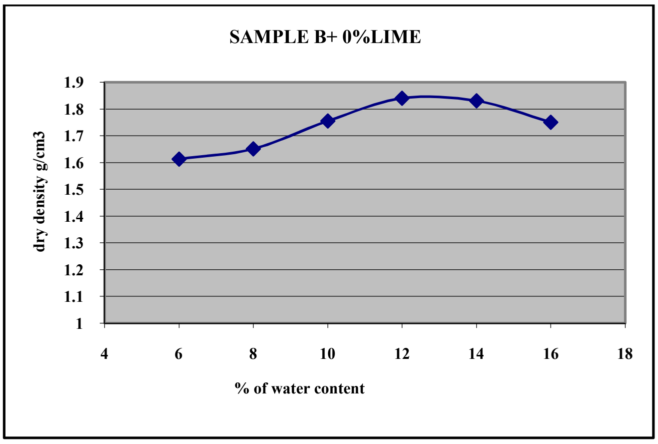

Figure 3.86 Standard proctor compaction test for sample B with 0% lime

Table 3.103 Standard proctor compaction test for sample B with 0% lime

Standard Proctor Compaction Test - Sample B-10%Lime					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil	Wet Density	Dry Density
6	6041	4446	1595	1.574174	1.48507
8	6121	4446	1675	1.653129	1.530675
10	6213	4446	1767	1.743928	1.585389
12	6289	4446	1843	1.818935	1.62405
14	6345	4446	1899	1.874204	1.644039
16	6384	4446	1938	1.912695	1.648875
18	6312	4446	1866	1.841635	1.560708

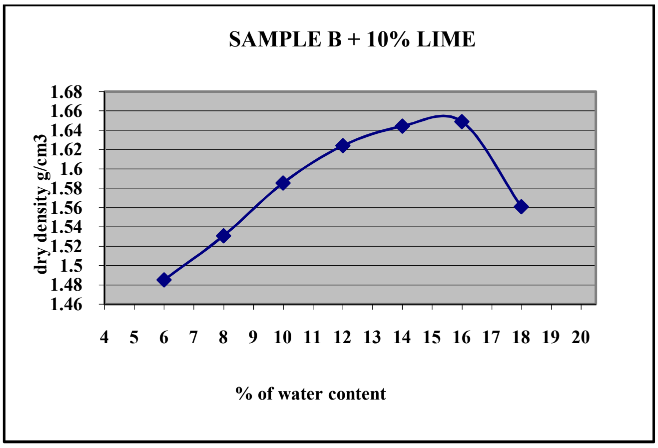

Figure 3.87 Standard proctor compaction test for sample B with 10% lime

Table 3.104 Standard proctor compaction test for sample B with 20% lime

Standard Proctor Compaction Test - Sample B-20%Lime					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of comp. soil (g)	Wet Den. (ϱ) g/cc	Dry Den. ($ϱ_d$) g/cc
6	6041	4446	1595	1.574174	1.48507
8	6137	4446	1691	1.66892	1.545296
10	6297	4446	1851	1.826831	1.660755
12	6390	4446	1944	1.918617	1.713051
14	6438	4446	1992	1.96599	1.724553
16	6487	4446	2041	2.01435	1.736509
18	6467	4446	2021	1.994611	1.690349

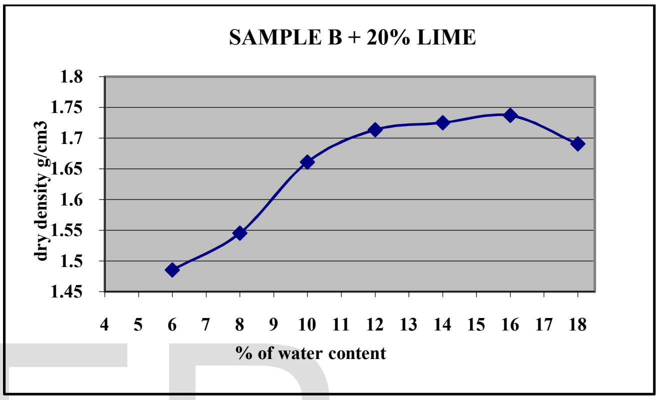

Figure 3.88 Standard proctor compaction test for sample B with 10% lime

Table 3.105 Standard proctor compaction test for sample A with 30% lime

Standard Proctor Compaction Test - Sample B-30%Lime					
Water content %	Wt of mould + Soil (g)	Empty wt. of mould (g)	Wt of compacted soil (g)	Wet Density (ϱ) g/cc	Dry Density ($ϱ_d$) g/cc
6	6523	4552	1971	1.945264	1.835155
8	6689	4552	2137	2.109097	1.952867
10	6743	4552	2191	2.162392	1.965811
12	6641	4552	2089	2.061723	1.840824

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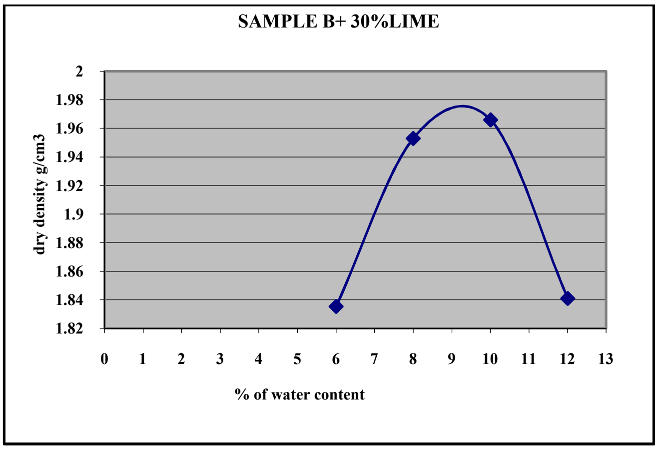

Figure 3.89 Standard proctor compaction test for sample B with 10% lime

Table 3.106 MDD and OMC value for sample B with lime various proportions

Sample and flyash proportions	MDD (g/cm^3)	OMC %
Sample B+ 0% lime	1.839943	12
Sample B+ 10% lime	1.648875	16
Sample B+ 20% lime	1.736509	16
Sample B+ 30% lime	1.965811	10

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CHAPTER-4

RESULTS AND DISCUSSION

4.1 RESULT FOR SOIL SAMPLE A

Table 4.1Result for soil sample A

S.No.	Laboratory Test	Result
1	Grain Size Distribution	74.69% fine
2	Specific Gravity (G)	2.5
3	Water Content (Natural) (w)	18.4%
4	Liquid Limit (WL)	62.5%
5	Plastic Limit (WP)	42.857%
6	Plasticity Index (IP or P.I)	19.643%
7	Shrinkage limit	37%
8	Free Swell Index (F.S.I)	52.38 %
9	Optimum Moisture Content (O.M.C.)	16%
10	Maximum Dry Density (M.D.D.)	1.464g/cc
11	Unconfined Compressive Strength (U.C.S.) at OMC	4.231Kg/cm2

4.2 RESULT FOR SOIL SAMPLE B

Table 4.2 Result for soil sample B

S.No.	Laboratory Test	Result
1	Grain Size Distribution	74.69% fine
2	Specific Gravity (G)	2.2
3	Water Content (Natural) (w)	25.1%
4	Liquid Limit (WL)	42%
5	Plastic Limit (WP)	25%
6	Plasticity Index (IP or P.I)	17%
7	Shrinkage limit	14.2%
8	Free Swell Index (F.S.I)	20 %
9	Optimum Moisture Content (O.M.C.)	12%
10	Maximum Dry Density (M.D.D.)	1.839943g/cc
11	Unconfined Compressive Strength (U.C.S.) at OMC	4.231Kg/cm2
12	California Bearing Ratio (C.B.R.)	4.087591%

4.3 THE RESULT FOR CONSISTENCY LIMITS OF SOIL SAMPLE A WITH VARIOUS STABILIZERS

It is observed that the liquid limit (LL), plastic limit and plasticity index decreases with increase in the flyash content from 10% to 30%. It is described in the Table 4.3.1 and fig.4.1

Table 4.3 Sample A – Stabilizer Flyash

Samples	Liquid limit (w_l) %	Plastic limit (w_p) %	Plasticity index (i_p) (w_l- w_p) %
0% flyash	62.4	42.857	19.543
10% flyash	57	22.22	34.78
20% flyash	47	20	27
30% flyash	48	33.33	14.67

It is observed that the liquid limit (LL), plastic limit and plasticity index decreases with increase in the lime content from 10% to 30%. It is described in the Table 4.3.2 and figure 4.2

Table 4.4 sample A – Stabilizer Lime

Samples	Liquid limit (w_l) %	Plastic limit (w_p) %	Plasticity index (i_p) (w_l- w_p) %
0% lime	62.4	42.857	19.543
10% lime	54	42.857	11.143
20% lime	53	42.857	10.143
30% lime	37	25	12

It is observed that the liquid limit (LL), plastic limit and plasticity index decreases with increase in the gypsum content from 10% to 30%. It is described in the Table 4.3.3 and fig.4.3

Table 4.5 Sample A – Stabilizer Gypsum

Samples	Liquid limit (w_l) %	Plastic limit (w_p) %	Plasticity index (i_p) (w_l- w_p) %
0% gypsum	62.4	42.857	19.543
10% gypsum	57.6	42.857	14.743
20% gypsum	55.2	25	30.2
30% gypsum	41.6	22.22	19.38

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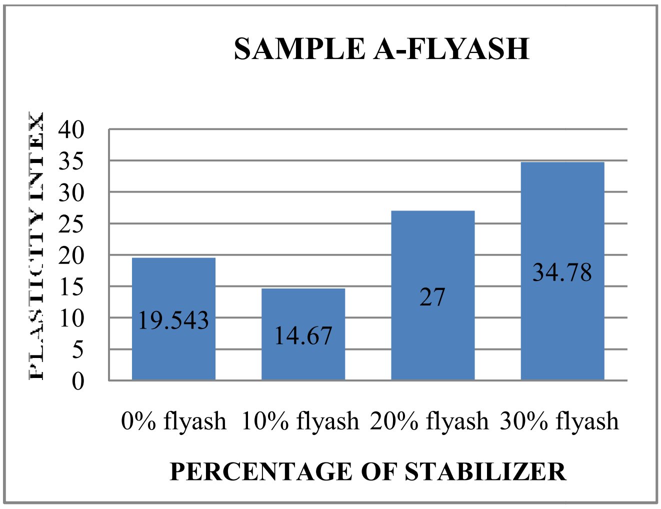

Figure 4.1 Bar chart for soil sample A with flyash in various proportions

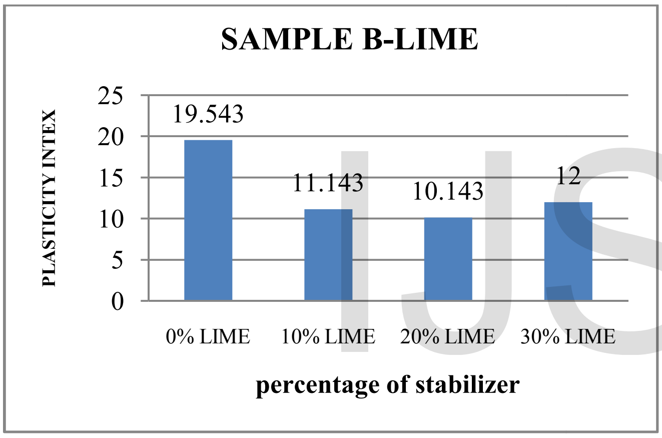

Figure 4.2 Bar chart for soil sample A with lime in various proportions

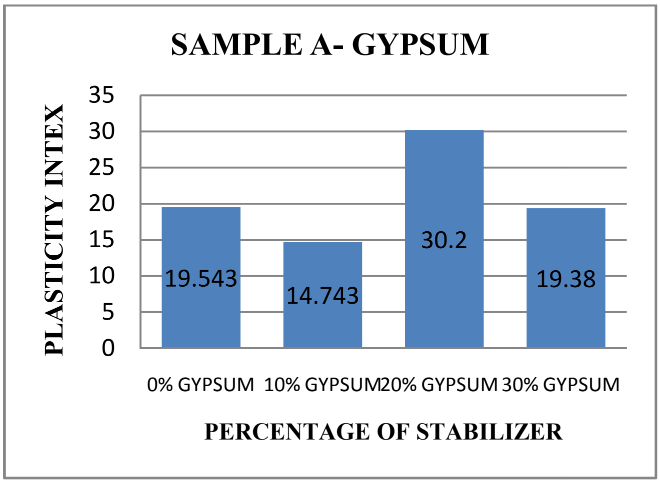

Figure 4.3 Bar chart for soil sample A with gypsum in various proportions

4.4 THE RESULT FOR CONSISTENCY LIMITS OF SOIL SAMPLE B WITH VARIOUS STABILIZERS

It is observed that the liquid limit (LL), plastic limit and plasticity index decreases with increase in the flyash content from 10% to 30%. It is described in the Table 4.4.1 and fig.4.4

Table 4.6 Sample B– Stabilizer Flyash

Samples	Liquid limit (w_l) %	Plastic limit (w_p) %	Plasticity index (i_p) (w_l- w_p) %
0% flyash	42	25	17
10% flyash	41.45	20	21.45
20% flyash	35.73	11.11	24.26
30% flyash	21.79	10	11.79

It is observed that the liquid limit (LL), plastic limit and plasticity index decreases with increase in the lime content from 10% to 30%. It is described in the Table 4.4.2 and figure 4.5

Table 4.7 Sample B– Stabilizer Lime

Samples	Liquid limit (w_l) %	Plastic limit (w_p) %	Plasticity index (i_p) (w_l- w_p) %
0% lime	42	25	17
10% lime	41	12.5	28.5
20% lime	35.64	11.11	24.53
30% lime	21.78	11.11	10.67

It is observed that the liquid limit (LL), plastic limit and plasticity index decreases with increase in the gypsum content from 10% to 30%. It is described in the Table 4.4.3 and fig.4.6

Table 4.8 Sample B– Stabilizer Gypsum

Samples	Liquid limit (w_l) %	Plastic limit (w_p) %	Plasticity index (i_p) (w_l- w_p) %
0% gypsum	42	25	17
10% gypsum	43.46	14.28	29.18
20% gypsum	37.8	11.11	26.69
30% gypsum	23.8	10	13.8

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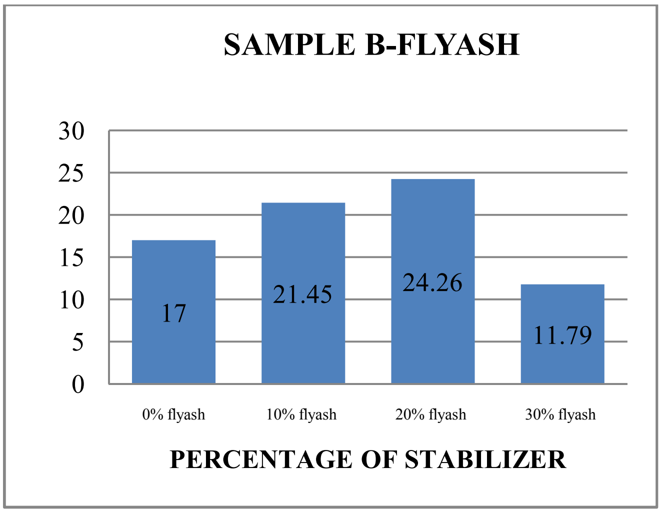

Figure 4.4 Bar chart for soil sample B with flyash in various proportions

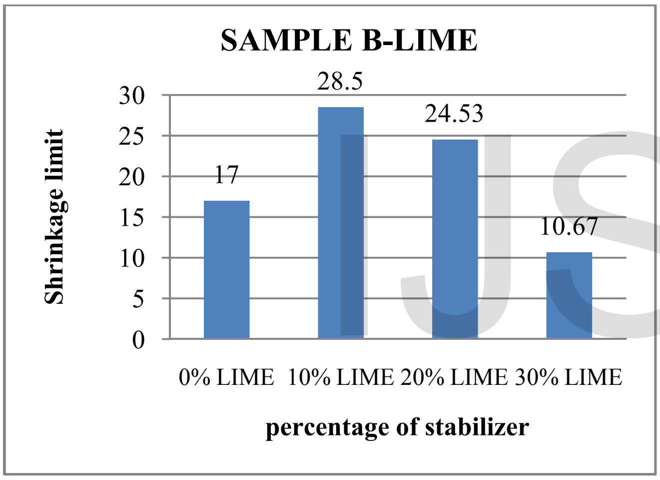

Figure 4.5 Bar chart for soil sample B with lime in various proportions

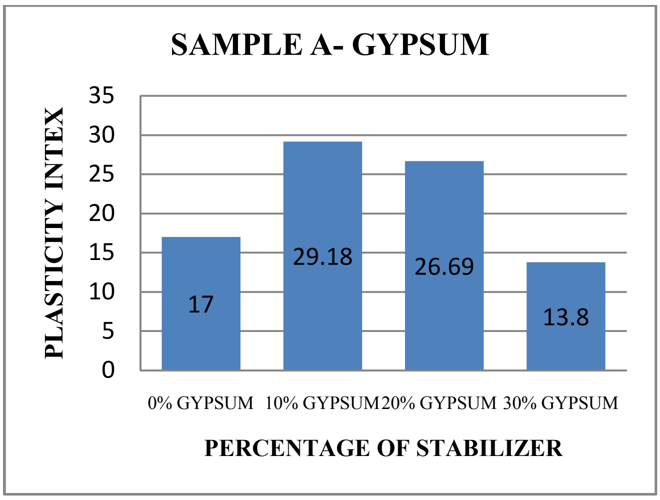

Figure 4.6 Bar chart for soil sample B with gypsum in various proportions

4.5 SHRINKAGE LIMIT FOR SAMPLE A WITH STABLILIZERS IN VARIOUS PROPORTIONS

Table 4.9 shrinkage limit for soil sample A with stabilizers in various proportions

Stabilizer	Percentage of stabilizer	Shrinkage limit
Soil A	0%	37
Flyash	10%	8.72
	20%	16.9
	30%	19.56
Lime	10%	11.17
	20%	20.53
	30%	16.9
Gypsum	10%	21
	20%	17
	30%	11.2

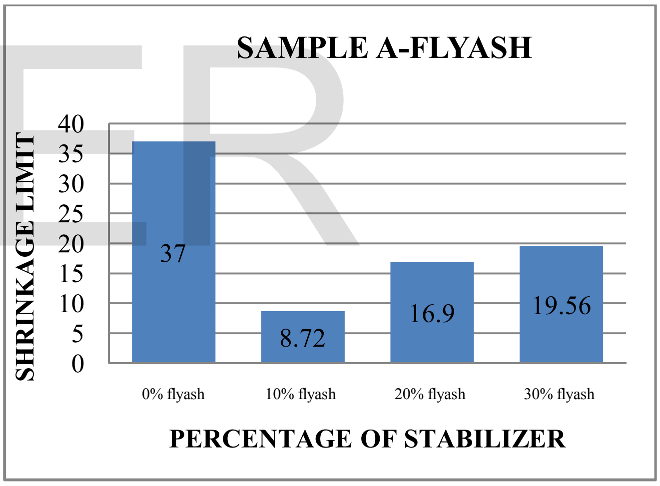

Figure 4.7 Bar chart for shrinkage limit of sample A with flyash in various proportions

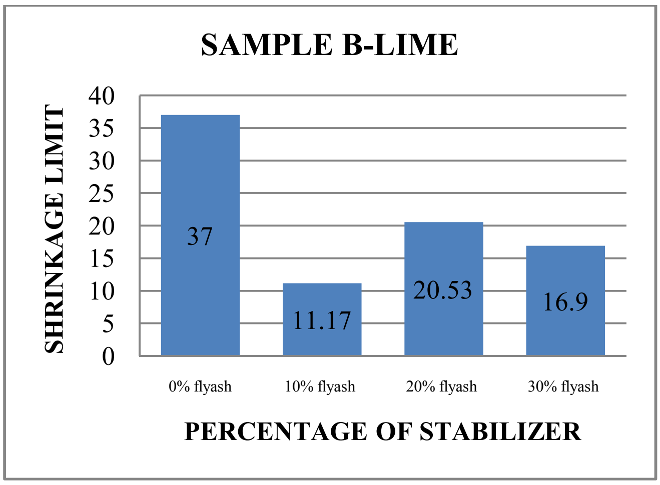

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Figure 4.8 Bar chart for shrinkage limit of soil sample A with lime in various proportions

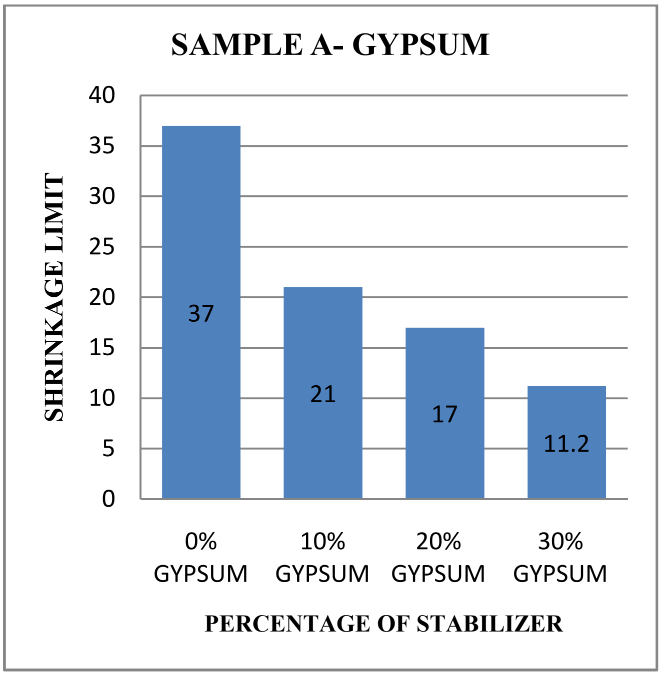

Figure 4.9 Bar chart for shrinkage limit of sample A with gypsum in various proportions

4.6 SHRINKAGE LIMIT FOR SAMPLE B WITH STABLILIZERS IN VARIOUS PROPORTIONS

Table 4.10 Shrinkage limit for sample B with stabilizers in various proportions

Stabilizer	Percentage of stabilizer	Shrinkage limit
Soil B	0%	14.2
Flyash	10%	19.56
	20%	16.9
	30%	8.72
Lime	10%	21
	20%	17
	30%	17
Gypsum	10%	20.53
	20%	16.9
	30%	16.64

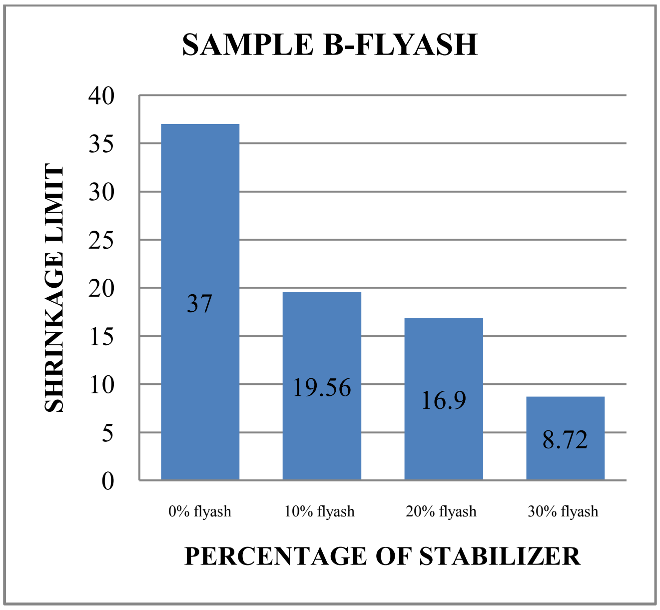

Figure 4.9 Bar chart for shrinkage limit sample B with flyash in various proportions

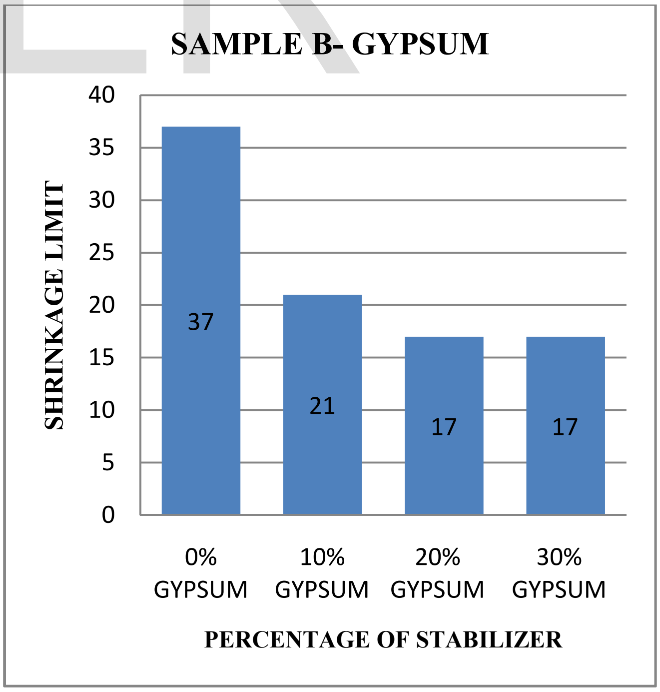

Figure 4.10 Bar chart for shrinkage limit sample B with gypsum in various proportions

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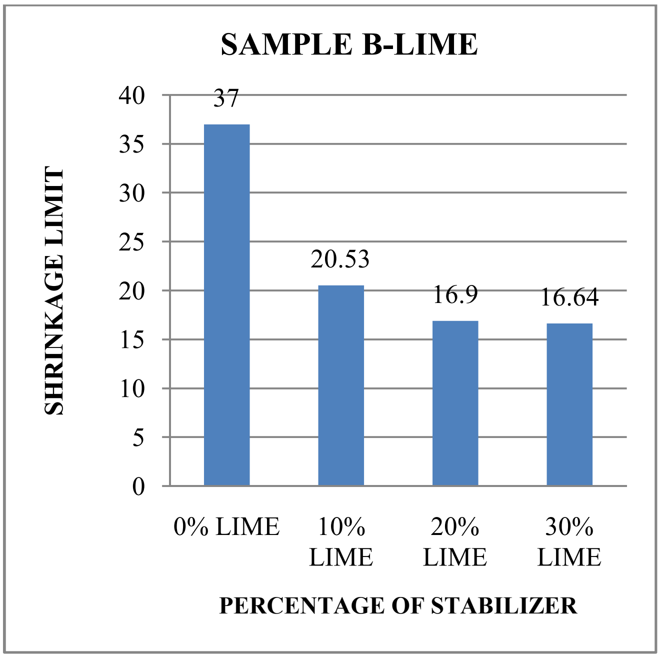

4.7 DIFFERENTIAL FREE SWELL INDEX FOR SOIL SAMPLE A WITH STABILIZERS IN VARIOUS PROPORTIONS

The clayey soil were tested for DFS for different proportions of soil, flyash the results are given in table 4.11. The clayey soil having maximum differential free swell index due to its mineral constitutes. Fig.4.12 shows that DFS of treated soil with flyash only is increase as compared to untreated soil. The DFS value increase considerably due to addition of flyash.

Table 4.11 Differential free swell index for sample A with flyash in various proportions

S. no	Observation	0% flyash	10% flyash	20% flyash	30% flyash
1	Volume of the soil in kerosene after swelling, V1 ml	21	21	21	21
2	Volume of soil in water after swelling,V2 ml	32	32	33	34
3	The free swell index of the soil (%) [(V1-V2)/V1] X 100 %	52.38 %	52.38 %	57.14 %	61.9 %
Degree of expansiveness of soil: Since, The Free swell index is greater than 50.		very high	very high	very high	very high

The clayey soil were tested for DFS for different proportions of soil, gypsum the results are given in table 4.12. The clayey soil having maximum differential free swell index due to its mineral constitutes. Fig.4.13 shows that DFS of treated soil with gypsum only is increase as compared to untreated soil. The DFS value increase considerably due to addition of gypsum.

Table 4.12 Differential free swell index for sample A with gypsum in various proportions

S. no	Observation	0% gypsum	10% gypsum	20% gypsum	30% gypsum
1	Volume of the soil in kerosene after swelling, V1 ml	21	21	21	21
2	Volume of soil in water after swelling,V2 ml	32	32	33	34.5
3	The free swell index of the soil (%)	52.38 %	52.38 %	57.14 %	64.28 %
Degree of expansiveness of soil: Since, The Free swell index is greater than 50.		very high	very high	very high	very high

The clayey soil were tested for DFS for different proportions of soil, lime the results are given in table 4.13. The clayey soil having maximum differential free swell index due to its mineral constitutes. Fig.4.14 shows that DFS of treated soil with lime only is increase as compared to untreated soil. The DFS value increase considerably due to addition of lime.

Table 4.13 Differential free swell index for sample A with lime in various proportions

S. no	Observation	0% lime	10% lime	20% lime	30% lime
1	Volume of the soil in kerosene after swelling, V1 ml	21	21	21	21
2	Volume of soil in water after swelling,V2 ml	32	33	34	35
3	The free swell index of the soil (%) [(V1-V2)/V1] X 100 %	52.38 %	57.14 %	61.9 %	66.67%
Degree of expansiveness of soil: Since, The Free swell index is greater than 50.		very high	very high	very high	very high

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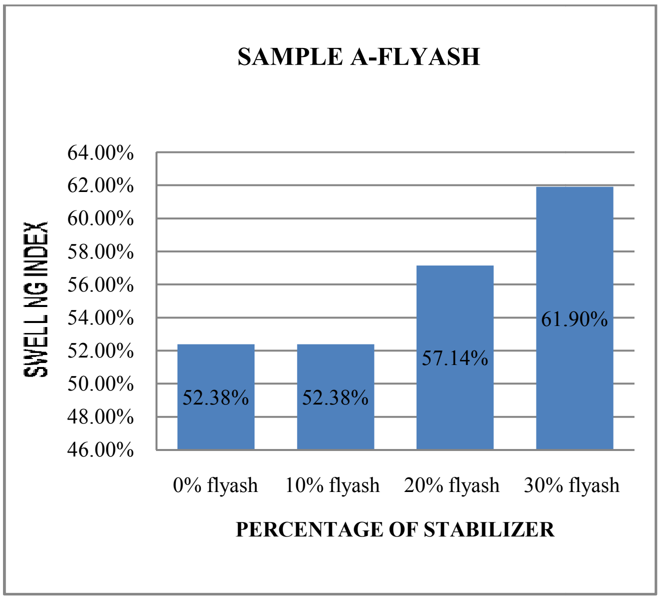

Figure 4.12 Bar chart for swelling index of sample A with flyash in various proportions

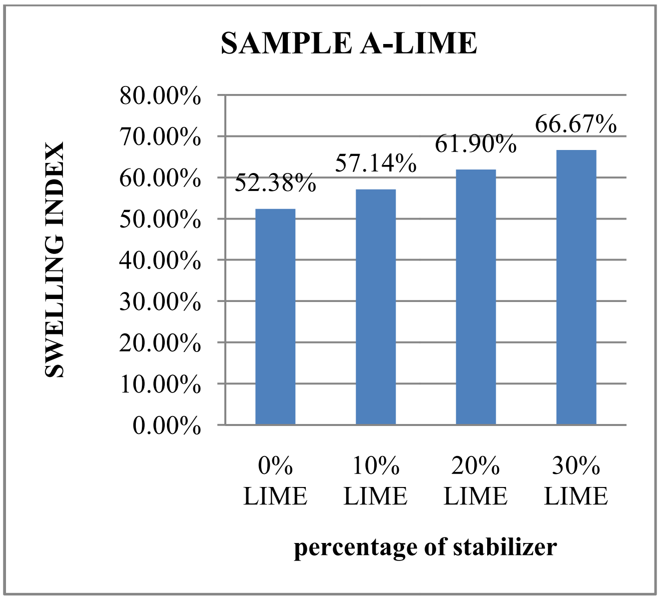

Figure 4.14 Bar chart for swelling index of soil sample A with lime in various proportions

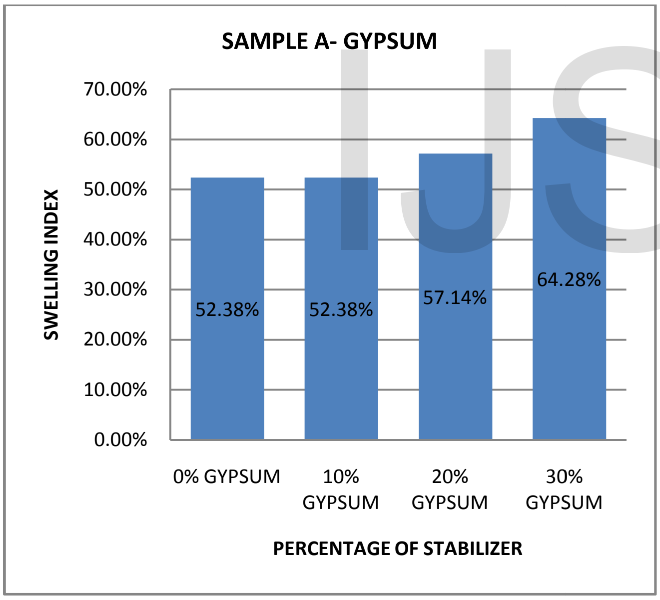

Figure 4.13 Bar chart for swelling index of soil sample A with lime in various proportions

4.8 STANDARD PROCTOR COMPACTION TEST RESULT FOR SOIL SAMPLE A WITH STABILIZERS IN VARIOUS PROPORTIONS

4.8.1 Effect of stabilizers on MDD and OMC of soil sample A

The results of the MDD and OMC for untreated soil and soil treated with different percentage of stabilizers are as given in table 4.14, figure 4.15 and figure 4.16 shows that, there is increase in MDD of treated soil with stabilizers than untreated soil. The stabilizers reacts chemically with soil particles and binds them together and reduces the pore spaces and help to increase the MDD of soil. The stabilizers contains fibres due to which the increase in MDD is less and if the percentage of stabilizers increases the MDD of soil reduces. The flyash

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contains fibres due to which the increase in MDD is less and if the percentage of stabilizers increases the MDD of soil reduces, then 10% to 30% of flyash added with soil sample is produce the MDD value is increased. The lime contains fibres due to which the increase in MDD is less and if the percentage of stabilizers increases the MDD of soil reduces, then 10% to 30% of lime added with soil sample is produce the MDD value is increased. The gypsum contains fibres due to which the increase in MDD is less and if the percentage of stabilizers increases the MDD of soil reduces, then 10% to 30% of gypsum added with soil sample is produce the MDD value is increased.

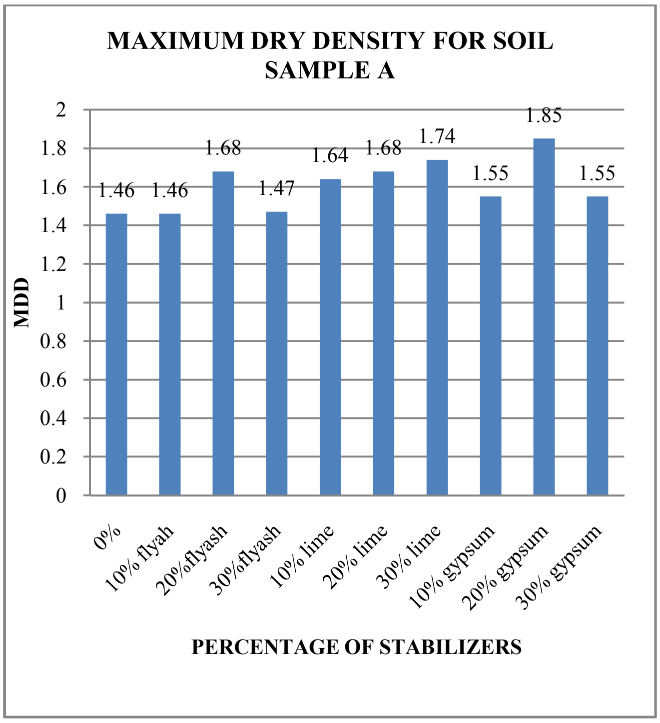

Figure 4.15 Maximum dry density for soil sample B with stabilizers in various proportions

Table 4.14 Maximum dry density and OMC value for sample A with stabilizers in various proportions

Stabilizer	Percentage of stabilizer	MDD (g/cm³)	OMC %
Soil A	0%	1.464	16
Flyash	10%	1.455	15
	20%	1.68	16
	30%	1.47	16
Lime	10%	1.635858	20
	20%	1.68027	20
	30%	1.742733	14
Gypsum	10%	1.548266	18
	20%	1.850518	16
	30%	1.552344	18

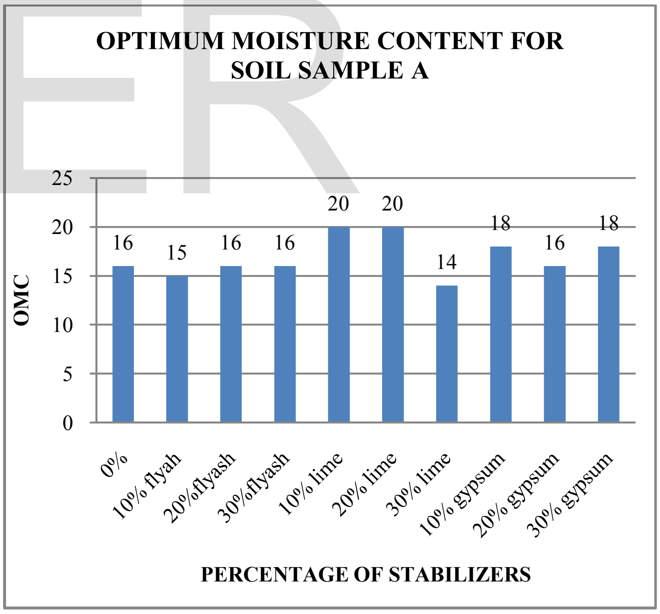

Figure 4.16 Optimum moisture content for sample B with stabilizers in various proportions

4.9 STANDARD PROCTOR COMPACTION TEST RESULT FOR SOIL SAMPLE B WITH STABILIZERS IN VARIOUS PROPORTIONS

4.9.1 Effect of stabilizers on MDD and OMC of soil sample B

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The results of the MDD and OMC for untreated soil and soil treated with different percentage of stabilizers are as given in table 4.15, figure 4.17 and figure 4.18 shows that, there is increase in MDD of treated soil with stabilizers than untreated soil. The stabilizers reacts chemically with soil particles and binds them together and reduces the pore spaces and help to increase the MDD of soil. The stabilizers contains fibres due to which the increase in MDD is less and if the percentage of stabilizers increases the MDD of soil reduces. The flyash contains fibres due to which the increase in MDD is less and if the percentage of stabilizers increases the MDD of soil reduces, then 10% to 30% of flyash added with soil sample is produce the MDD value is increased. The lime contains fibres due to which the increase in MDD is less and if the percentage of stabilizers increases the MDD of soil reduces, then 10% to 30% of lime added with soil sample is produce the MDD value is increased. The gypsum contains fibres due to which the increase in MDD is less and if the percentage of stabilizers increases the MDD of soil reduces, then 10% to 30% of gypsum added with soil sample is produce the MDD value is increased.

Table 4.15 Maximum dry density and OMC value for sample B with flyash in various proportions

Stabilizer	Percentage of stabilizer	MDD (g/cm³)	OMC %
Soil B	0%	1.839943	12
Flyash	10%	1.736509	16
	20%	1.736509	16
	30%	1.940688	10
Lime	10%	1.648875	16
	20%	1.736509	16
	30%	1.965811	10
Gypsum	10%	1.654831	16
	20%	1.732255	16
	30%	1.926333	10

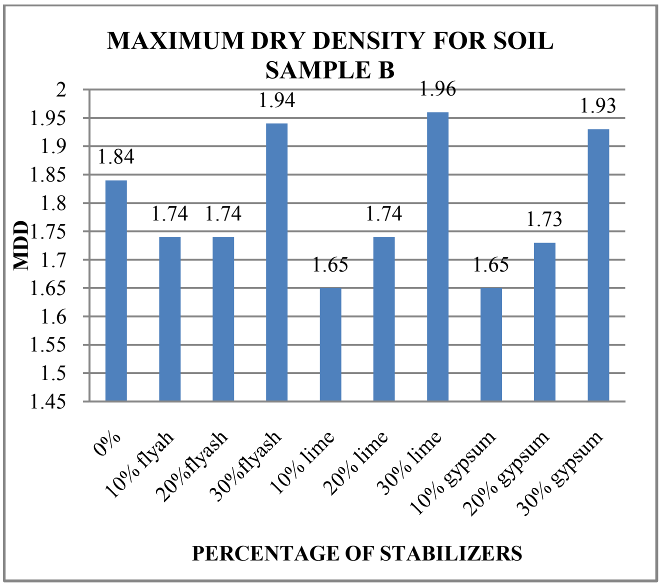

Figure 4.17 Maximum dry density for soil sample B with stabilizers in various proportions

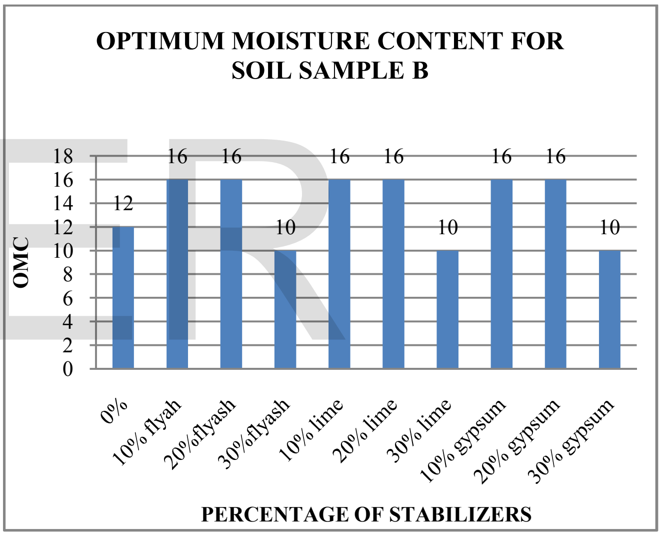

Figure 4.18 Optimum moisture content for soil sample B with stabilizers in various proportions

4.10 CBR TEST

It is observed that with the addition of stabilizers the CBR of sub grade soil increased. The CBR value is increased in both soaked and un soaked condition.

Table 4.16 Effect of flyash on Strength (CBR) Characteristics of Soil

Flyash Proportions	Sample B (CBR %)	
	Soaked	Un soaked
0% flyash	10.15	4.09
10% flyash	12.68	5.06
20% flyash	14.27	8.02

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30% flyash	16.22	10.58

The addition of flyash improved the CBR significantly. The CBR increased from 4.09% for only sample B to 10.58% for sample B + flyash. These variations are cited in Figure 4.19 and 4.20. It is shown in Table 4.16

Table 4.17 Effect of lime on Strength (CBR) Characteristics of Soil

Lime Proportions	Sample B (CBR %)	
	Soaked	Un soaked
0% lime	8.22	3.16
10% lime	10.16	5.11
20% lime	12.21	7.15
30% lime	14.26	9.19

The addition of flyash improved the CBR significantly. The CBR increased from 3.16% for only sample B to 9.19% for sample B +lime. These variations are cited in Figure 4.19 and 4.20. It is shown in Table 4.17

Table 4.18 Effect of gypsum on Strength (CBR) Characteristics of Soil

Gypsum Proportions	Sample B (CBR %)	
	Soaked	Un soaked
0% gypsum	10.38	4.04
10% gypsum	12.58	6.08
20% gypsum	14.90	8.42
30% gypsum	16.34	10.68

The addition of gypsum improved the CBR significantly. The CBR increased from 4.08% for only sample B to 10.68% for sample B +gypsum. These variations are cited in Figure 4.19 and 4.21. It is shown in Table 4.18

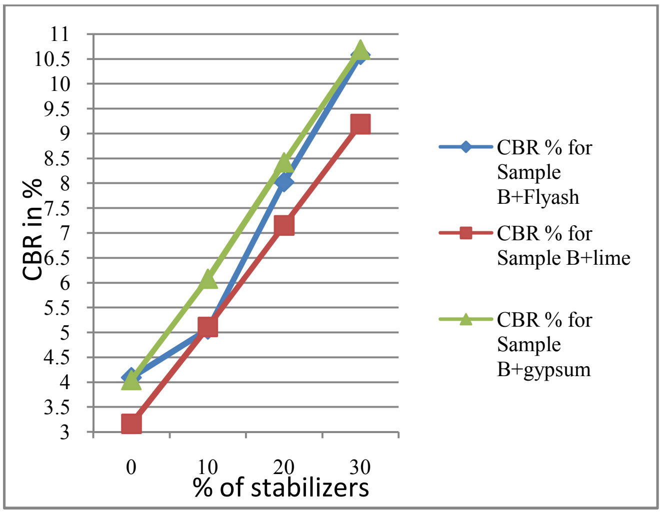

Figure 4.19 Effect of stabilizers on Strength (CBR) Characteristics of Soil

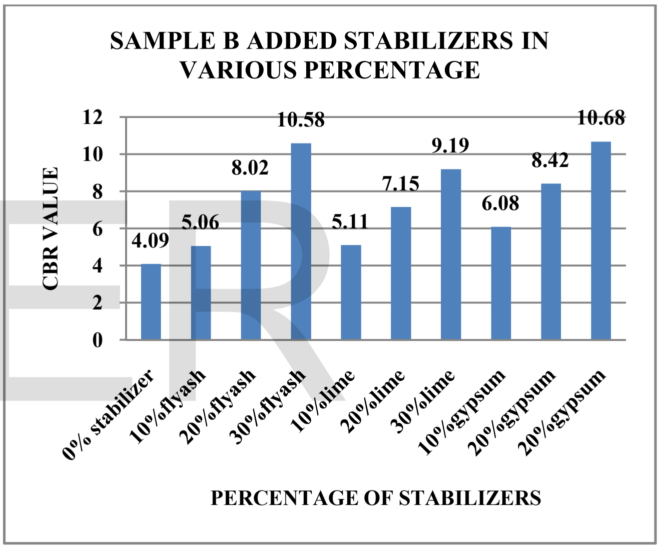

Figure 4.20 Bar chart for Effect of stabilizers on Strength (CBR) Characteristics of Soil

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CHAPTER 5

SUMMARY AND CONCLUSION

From the result and discussion of the project, the following points were concluded. Based on the laboratory experimental investigations in this study following conclusions can be drawn.

1. The geotechnical properties of clayey soil improve significantly due to addition of flyash, lime and gypsum.
2. The lime work as a good clayey soil stabilizer flyash and gypsum.
3. The dry density of clayey soil decreased by adding lime, gypsum and flyash.
4. The soaked CBR value of clayey soil improved by addition of lime and therefore it is possible to reduce the thickness of road. The optimum mixed obtained for subgrade soil is soil+30% lime of its weight.
5. The use of lime for road construction work reduces environmental pollution up to certain extent.
6. As the disposal of flyash is a big problem in thermal industries, flyash stabilization is one of the best methods for the effective and economical disposal of flyash.

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