A STUDY ON
STABILIZATION OF CLAYEY SOIL USING WASTE MATERIAL

A dissertation submitted in

Partial Fulfillment of the Requirement for the Award of the Degree of

MASTER OF TECHNOLOGY
In
CIVIL ENGINEERING
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DECLARATION

I hereby declare that the work presented in the dissertation **"A STUDY ON STABILIZATION OF CLAYEY SOIL USING WASTE MATERIAL"** in partial fulfillment of the requirement for the award of the degree of **"MASTER OF TECHNOLOGY"** in Civil Engineering (With specialization in Geotechnical Engineering), submitted in the Department of Civil Engineering, Assam Engineering College, Jalukbari, Guwahati-13 under Assam Science & Technology University, is a real record of the work carried out in the said college for twelve months under the supervision of Prof. Bhaskarjyoti Das, Associate Professor, Department of Civil Engineering , Assam Engineering College, Jalukbari, Guwahati-13.

Do hereby declare that this project report is solemnly done by me and is my effort and that no part of it has been plagiarized without citation.

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This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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ABSTRACT

The field of geotechnical engineering faces many problems related to construction on problematic soil. These problems often lead to settlement issues. So, improvement of the soil is much required for increasing its strength and bearing capacity. Soil Stabilization is the process of improving the engineering properties of soil and thus making it more stable by increasing its shear strength, bearing capacity and reduce the construction cost by making best use of the locally available materials. It is required when the soil available for construction is not suitable for the intent purpose. Soil stabilization being used for a variety of engineering works and the most common application in the construction of roads are air-field pavements, volume stability etc. Geotechnical properties of the problematic soils are improved by various methods such as controlled compaction or addition of admixtures like fly ash, lime, etc. But the cost of these admixtures has also increased in recent years which has been a great opening for the development of other additives which are locally available and most importantly are the waste products. Some of the stabilizing agents are sugarcane bagasse, rice husk, stone dust, natural fibers like coconut coir, egg-shells etc. This new technique of soil stabilization has attained much progress as it leads to reduce the quantities of industrial waste and its disposal problem. This study put emphasis on sugarcane bagasse and fly ash as soil stabilizer in varying percentages to examine in what way the properties of the soil changes in comparison with the untreated soil and a comparative analysis of static UCS and dynamic UCS of both the waste has been done.

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LIST OF ABBREVIATIONS & SYMBOLS

Abbreviation & Symbols	Definition
FA	Fly ash
SB	Sugarcane bagasse
OMC	Optimum Moisture content
MDD	Maximum dry density
LL	Liquid limit
PL	Plastic limit
PI	Plasticity index
UCS	Unconfined Compressive strength
CBR	California Bearing Ratio
ε	Axial Strain
SBA	Sugarcane bagasse ash

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CHAPTER 1
INTRODUCTION

1.1 General

The basic construction material of the geotechnical engineering field is the soil. Soil is either a part of the foundation or one of the raw materials used in the construction process which is also an unconsolidated matter which has great variability in its chemical composition. Therefore, it is expected that soil properties are also bound to the chemical variability of its constituents. Soil contains almost all types of elements including the most important ones are oxygen, silicon, hydrogen, aluminum, calcium, sodium, potassium, magnesium and carbon. Atoms of these elements form different crystalline arrangement to yield common minerals with which the soil is made up of. Soil in general is made up of minerals(solids), liquid(water), organic compounds and gases. But in many of the cases like road construction, foundation layers etc., soil of poor quality cannot be used directly because there are many engineering properties associated with the soils like low bearing capacity, high settlement, high erodibility, soil deformations etc. Therefore, it is required to improve the quality of the soil. The stabilization of geotechnical properties of soil aims to increase the shear strength, decrease properties like permeability, deformability etc. because some soils show major volume changes due to change in the moisture content. These soils are capable of absorbing more water because of this property volume increase as well as compressibility increases which becomes very dangerous for the construction purposes.The most common application of the stabilization technique is to reduce the construction cost by making best use of the locally available materials and to improve the properties of the soil. Stabilization can increase the shear strength of a soil and to control the shrink -swell properties of a soil, thus improving the load bearing capacity of a sub-grade to support pavements and foundations. The most common improvement achieve through soil stabilization technique includes better soil gradation, reduction of swelling potential, increase in durability and strength. It is important either to remove the existing soil and replace it with a good quality soil or to improve the important properties of the existing soil by adding admixtures or cost-effective practices like most importantly use of waste materials. Due to industrial revolution waste materials like sugarcane baggage, rice husk, plastics, fly ash etc.

considered to be one of the biggest problems in the world. The construction and demolition, mining and industrial wastes presents 74% of whole waste which are a huge amount of waste. The waste materials in most of the cases are non-biodegradable, causes environmental pollution, and a huge effect to human life and biodiversity. So, if these wastes can be a good adding agent for the soil for improving its properties, then it is definitely a good cost-effective approach. As good soil becomes scarer and their location becomes more difficult and costly, the need to improve quality of the soil using soil stabilization is becoming more important. The first experiment on soil stabilization was conducted in USA with sand or clay mixtures in 1906. The soil stabilization for road construction was done in thirties in Europe. Many researchers attempt to use the industrially available wastes like rice husk (RHA), fly ash etc. are used to revamp the geotechnical properties of a soil. However, the inclination of using the waste material is being used by all over the world nowadays. In this study the waste materials that are beings used are sugarcane bagasse and fly ash. **Nirali Bhaskar Hasilkar (2017)** et.al., studied about the use of sugarcane bagasse as a waste material for the stabilization of the soil having CL classification with increase in waste percentage as 1 to 3% for the strength testing of the soil includes Unconfined Compressive Strength Test, California Bearing ratio test along with the geotechnical properties of the soil. **B.A. Mir (2013)** et.al., performed laboratory tests involved determination of physical properties, compaction characteristics and swell potential. The test results show that the consistency limits, compaction characteristics and swelling potential of expansive soil–fly ash mixtures are significantly modified and improved. Again, **SPK Kodicherla(2019)** et.al., dealt with the stabilization of subgrade with the use of coir fibers along with fly ash. Whereas now a days fly ash has opened doors for its utilization in agriculture, rising as a tremendous potential in improving crop productivity and soil health as well. Besides its nutrient efficiency, fly ash treatment showed a significant result in agricultural insect-pest control. The coal-based thermal plants in North-east India produce approximately 37,000 t of fly ash per year **(Pandey et al., 2011)**. In many of the soil stabilization techniques giving an example of fly ash and sugarcane bagasse, these two materials are used by many along with the mix of lime, cement or sugarcane bagasse can also be used as an ash. Soil stabilization techniques using various waste materials alone with the soil or with the addition of other additives are prevailing very much in today’s world.

1.2 Types of Soil Stabilization Techniques

Methods of soil stabilization may be grouped under two main types namely-

a) Modification or improvement of the properties of the existing soil without any admixture, and
b) 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 the soil whereas the examples of the second type are: mechanical stabilization, stabilization with cement, lime, bitumen, and chemicals etc.

1)Mechanical stabilization: It deals with changing the composition of soil by addition or removal of certain constituents and densification or compaction. The primary purpose is to have a soil resistant to deformation and displacement under loads, soil materials can be divided into two fractions: the granular fraction retained on a 75 micron IS sieve and the fine fraction passing a 75 micron IS sieve. The granular fraction imparts strength and hardness whereas the fine fraction provides cohesion or binding property as well as acts as a filler for the voids of the coarse fraction. Mechanical stabilization has been largely used in the construction of cheap roads. For bases the liquid limit should not exceed 25% and plasticity index not exceeding 6. For surfacing the liquid limit should not exceeding 35% and plasticity index should be between 4-9.

2) Cement stabilization: The soil stabilized with cement (Portland) is known as soil cement. The cementing action is believed to be the result of chemical reaction of cement with silicious soil during hydration. The binding action of individual particles through cement may be possible only in coarse-grained soils. In fine-grained, cohesive soils, only some of the particles can be expected to have cement bonds, and the rest will be bonded through natural cohesion. The important factors affecting soil cement are nature of the soil, cement content, conditions of mixing, curing and admixtures.

3) Lime stabilization: Hydrated(slaked) lime is very effective in treating heavy, plastic clayey soils. Lime may be used alone or in combination with cement, bitumen, or fly ash. Lime has been used mainly for stabilizing the roads bases and subgrades. Lime reduces the plasticity index of highly plastic soils making them easier to handle.

The amount of lime required may be used on the unconfined compressive strength or the CBR test criteria. Normally 2 or 8% of lime may be required for coarse grained soils and 5 to 10% for plastic soils.

4) Bitumen stabilization: The bituminous materials like asphalts and tars are mostly used for stabilization of soils, generally for construction of pavements. These materials are normally too viscous to be incorporated directly with the soil. The fluidity of asphalts is increased by either heating, emulsifying or by cut-back process. The bituminous materials when added to soil impart cohesion and reduced water absorption. Bitumen stabilization are of many types likewise soil-bitumen, water-proofed mechanical stabilization, soil-bitumen etc.

5) Chemical stabilization: There are many chemicals used for stabilization. Calcium chloride, Sodium chloride, Sodium silicate are one of the major ones. Calcium chloride acts as soil flocculent. It facilitates compaction and usually causes a slight increase in the compacted density. Sodium chloride is somewhat similar to that of calcium chloride. The sodium silicate in combination with other chemicals such as calcium chloride is used as an injection for stabilizing deep deposits of soil. These injections are found to be most successful in fine and medium sands.

6) Stabilization by heating: Heating a fine-grained soil to temperatures of the order of 400-600°C causes irreversible changes in clay minerals. The soil becomes non-plastic, less water sensitive and non-expansive. The method consisted in burning a mixture of liquid fuel and air injected into the ground through a network of pipes.

7) Electrical stabilization: The stability or shear strength of fine-grained soils can be increased by draining them with the passage of direct current through them. Electrical drainage is accompanied by electro-chemical composition of the electrodes and deposition of the metal salts in the soil pores. There may also be some changes in the structure of the soil. The resulting cementing of soil due to all these reactions is also known as electro-chemical hardening and for this purpose the use of aluminum anodes is recommended.

1.3 Materials used in soil stabilization

- Cement
- Lime
- Bitumen
- Polymer
- Geotextile
- Different grades of soil
- Fibrous materials
- Waste materials – industrial waste, solid municipal waste etc.
- Aggregates of various grades.
- Emulsions.
- Naturally available materials – sugarcane bagasse, coconut coir, areca nut fiber etc.

1.4 Application of Soil Stabilization

The process of soil stabilization is useful in the following applications:

- Increasing the bearing capacity of the foundation soils.
- Reducing the permeability of soils.
- Increasing the shear strength of soils.
- Improving the durability under adverse moisture and stress conditions.
- Substituting poor quality soils with good quality soil or adding certain admixtures or locally available fibers, wastes etc.
- To enhance unfavorable soil properties such as excessive swelling or shrinkage, high plasticity and so on.
- Controlling the grading of soils and aggregates in the construction of bases and sub bases of the highway and air fields.

1.5 Sugarcane bagasse and its potentiality

Wastes have become the integral part of our day-to-day life. But then the disposal and dumping of the used and unwanted wastes has become a major problem for the society. One of the waste materials which I used as a stabilizing agent while performing the laboratory experiments is sugarcane bagasse. Sugarcane bagasse is the dry pulpy fibrous material that remains after crushing sugarcane. Bagasse is the solid by- product when the liquid components are extracted from the plants. There is about 30% bagasse produced from the crushed sugarcane. This bagasse is used by many researchers as a soil stabilizer due to fibrous material contained in it. Because of its fibrous material it helps to binds the particles of the soil together by reducing the void ratio and increasing its shear strength. Again, each tone of sugarcane yields approximately 26% bagasse fibers, which are most burnt as bioenergy for electricity in sugar mills as well. Sugarcane bagasse contains about 40-50% cellulose which helps in binding the soil particles together and improves its strength. It also contains other fibers i.e., lignin, hemicellulose. The use of sugarcane bagasse is also a cheap approach as it is available locally or near the industries as compared to other expensive admixes or additives. The interest in using bagasse like any other fibers or waste materials is the global initiative of minimizing waste material to the environment and the use of cheaply and abundantly available materials.

Fig 1.1 Sugarcane bagasse

1.6 Fly Ash and its brief summary

At present, nearly 100 million tons of fly ash is being generated annually in India posing serious health and environmental problems. Fly ash is a coal combustion product that is composed of the particulates (fine particles of burned fuel) that are driven out of coal-fired boilers together with the flue gases. Ash that falls to the bottom of the boiler's combustion chamber (commonly called a firebox) is called bottom ash. In modern coal-fired power plants, fly ash is generally captured by electrostatic precipitators or other particle filtration equipment before the flue gases reach the chimneys. Together with bottom ash removed from the bottom of the boiler, it is known as **coal ash**. There are basically four types/ranks of coal: anthracite, bituminous, sub-bituminous, and lignite. The principal components of bituminous coal fly ash are silica, alumina, iron oxide, and calcium, with varying amounts of carbon. Lignite and sub-bituminous coal fly ash is characterized by higher concentrations of calcium and magnesium oxide and reduced percentages of silica and iron oxide, as well as lower carbon content, compared with bituminous coal fly ash. Depending upon the source and composition 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), aluminum oxide (Al_2O_3) and calcium oxide (CaO), the main mineral compounds in coal-bearing rock strata. Fly ash is fine glass powder, the particles of which are generally spherical in shape and range in size from 0.5 to 100 μm.

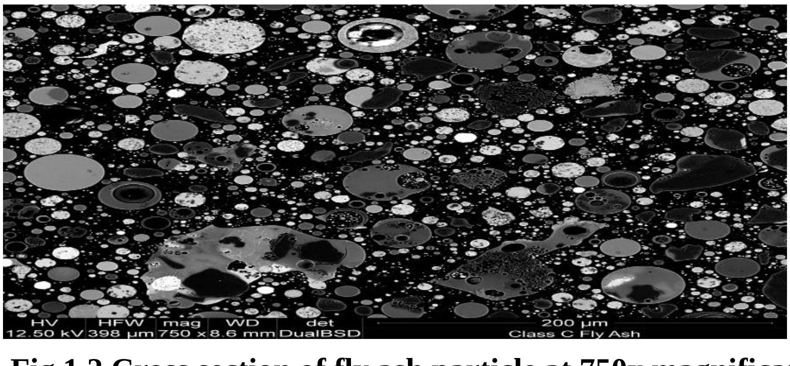

Fig 1.2 Cross section of fly ash particle at 750x magnification

(Source:https://en.wikipedia.org/wiki/Fly_ash#/media/File:Back-Scattered_Electron_Micrograph_of_Coal_Fly_Ash_small.tif)

Fly ash is classified into two types according to the type of coal used.

1. **Class C Fly Ash:** Two classes of fly ash are defined by American Society for Testing and Materials (ASTM) C618: Class F fly ash and Class C fly ash. The chief difference between these classes is the amount of calcium, silica, alumina, and iron content in the ash. The chemical properties of the fly ash are largely influenced by the chemical content of the coal burned (i.e., anthracite, bituminous, and lignite). Fly ash produced from the burning of younger lignite or sub-bituminous coal, in addition to having pozzolanic properties, also has some self-cementing properties. In the presence of water, Class C fly ash hardens and gets stronger 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.
2. **Class F Fly Ash:** The burning of harder, older anthracite and bituminous coal typically produces Class F fly ash. This fly ash is pozzolanic in nature, and contains less than 7% 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 mixed with water to react and produce cementitious compounds. Alternatively, adding a chemical activator such as sodium silicate (water glass) to a Class F ash can form a geopolymer. But the disposal problem of fly ash becomes a major concern so the most commonly used method is addition of fly ash as a stabilizing agent usually used in combination with soils. Coal continues to be one of the primary sources of energy in India and the present generation of fly ash is more than 150 million tons per year posing serious disposal and environmental problems. Thus, the coal-based thermal power plants not only in India, but also all over the world face a serious problem of handling and disposal of ash generated. In India, this problem is particularly sensitive and complex due to the high ash content (30–45 %) of coal. The safe disposal of these ashes without affecting the environment and the large area involved are of major concern. Therefore, it is important to find alternative uses for fly ash so that their bulk disposal without adverse environmental effects becomes possible.

Not only in the case of stabilization of soil, fly ash has many other uses as well. It is in the concrete production, as a substitute material for Portland cement, sand, corrosion control in RC structures, cement clinker production, subbase material for road construction, for brick production, agricultural uses etc. Other applications include cosmetics, kitchen counter tops, floor and ceiling tiles, utensils, tool handles, picture frames, auto bodies and boat hulls, cellular concrete, geopolymers, roofing tiles, roofing granules, decking, fireplace mantles, cinder block, PVC pipe, structural insulated panels, running tracks, utility poles and crossarms, railway sleepers, highway noise barriers, marine pilings, doors, window frames, scaffolding, sign posts, columns, railroad ties, vinyl flooring, paving stones, garage doors, park benches etc.

Fig 1.3 Picture of fly ash used in this study

1.7 Objective of the study

- To reduce the settlement of the structure on the soil.
- Improve the shear strength of the soil and thus increase the bearing capacity.
- Increase durability and strength of the soil.
- Reduce the water absorption capacity of the soil.
- Reduces plasticity index, lower permeability and reduction of pavement thickness by increasing the bearing capacity by addition of sugarcane-bagasse and fly ash.
- Comparative study of both sugarcane bagasse and fly ash waste material which shows better results on addition to the soil.

- Comparative study of the dynamic compaction and static compaction of Unconfined Compressive Strength Test using both the waste.

1.8 Advantages of soil stabilization

- It improves the strength of the soil results in increasing the soil bearing capacity.
- It is also used to provide more stability to the soil in slopes or other such places.
- Sometimes soil stabilization is also used to prevent soil erosion or formation of dust, which is very useful especially in dry and arid weather.
- Soil stabilization is also done for soil water proofing. This prevents water from entering into the soil and helps the soil from losing its strength.
- It is more economical in terms of costs and energy.
- Minimizes and decreases volume instability, swelling and shrinkage control.
- Reduces soil permeability, plasticity index (PI), soil compressibility, deformation and settlement.
- Soil stabilization improves the workability and soil durability of the soil.
- It saves money by making use of the locally available materials compared to the cost-effective additives.
- It reduces risk by incorporating stabilizers. Specially, lime can be used to dry moist and makes it safer to work with it.
- It also helps in the conservation of energy.

1.9 Disadvantages of soil stabilization

- It is not suitable for some types of soil.
- Addition of some types of stabilizers may not always respond in a positive way.
- If cement is used as a stabilizer, then addition of too high cement leads to brittleness.
- The use of toxic wastes should be avoided as much as possible.

CHAPTER 2
REVIEW OF LITERATURE

2.1 General

The studies of soft soil stabilization by using industrial waste were done by different researchers at different times. Some of the literatures are discussed briefly in this chapter.

2.2 Review of literature

Bidula Bose (2012) this paper studies about stabilization of expansive soil using fly ash as a stabilizer. Tests that are carried out includes specific gravity, liquid limit, plastic limit, free swell index, Standard proctor test, Modified Proctor test, Unconfined compressive strength test, California bearing ratio (soaked and unsoaked). Fly ash were added as 20%,40%,60%,80%&90% and it has been found out that the liquid limit, plastic limit and plasticity index of the soil decreased. Addition of 20% fly ash decreased the liquid limit of the untreated soil by 43%. Again, the plastic limit on addition of 20% fly ash decreased the plastic limit by 52%. The value of free swell index of the soil also decreased with the increase in addition of fly ash. The OMC of the soil found to be decreased on waste addition whereas the MDD of the soil found out to be maximum at 20% waste addition after that it decreased. The reduction of OMC of the soil was explained as the cation exchange between the additive and soil decreases the diffuse double layer results in flocculation of the particles which results in decreased in OMC of the soil. The CBR value was found to be maximum at 20% waste addition and after that it decreased. The unsoaked CBR value shows higher value compared to soaked CBR. In this paper, the unsoaked CBR peaks were found out to be on 20% and 80% addition of fly ash.

B. Naga Niranjan Kumar and Dr. M. Ashok Kumar (2016) this paper deals with soil stabilization with fly ash-industrial waste which was used as crushed marble powder. The aim of this paper is to stabilize the soft soil as a construction material using industrial waste-fly ash mixture. Fly ash has some cementitious properties due to which when it reacts with water forms cementitious compounds which leads to the improvement of the soil. Industrial waste-fly ash soil mixtures were prepared at several percentages (0%, 5%, 10%, 15%) by weight of the soil and tests are done including compaction properties, UCS, and permeability. Stress-strain behavior of UCS showed that failure stress and strains increased by 106% and 50% respectively when the fly ash content was increased from 0-25%. When the industrial waste content was increased from 0-12%, UCS increased by 97% while CBR improved by 47%. It has been concluded that industrial waste content of 12% and a fly ash content of 25% recommended for strengthening the expansive soil.

Aparna Roy (2014) this paper deals with the soil stabilization using rice husk ash (RHA) and cement. Soil sample taken for this study is clay with high plasticity (CH) which was strengthened with different percentages of RHA like 10%, 15%, 20% and a small amount of 6% cement is mixed with the soil. Experiments are done for evaluating the properties of soil like MDD, OMC, CBR and UCS. The best results are obtained that the increase in RHA content increases the OMC but decreases the MDD. Also, the CBR value and UCS of soil are considerably improved with RHA content. The CBR value is increased by 106% for RHA content of 10% but after that the CBR value is slightly decreased for RHA content of 15%. The reason for increment in CBR may be because of the gradual formation of cementitious compounds in the soil by the reaction between RHA and cement. Variation of UCS with increase in RHA from 10% to 20% were investigated (90.6% improved).

Amu,O.O. et.al (2011) studied about the geotechnical properties of lateritic soil modified with sugarcane straw ash with a view to obtain a cheaper replacement in place of those costly stabilizers. Lateritic soils are most commonly found in a leached soil of humid tropics. Preliminary tests were performed on three samples A, B & C for identification and classification of consistency limit tests, strength tests were performed on the samples (adding 2%, 4%, 6% and 8% sugarcane straw ash). After results were obtained sugarcane straw ash comes out to be a good stabilizing agent. OMC increased from 19-20.3%, 13.3-15.7%, and 11.7-17%, CBR increased from 6.31-23.3%, 6.24-14.88%, 6.24-24.88% and UCS increased from 79.64-284.66 KN/m^2, 204.86-350.10 KN/m^2 and 240.4-564.6% KN/m^2 in samples A, B & C respectively.

Ajay Upadhyay and Suneet Kaur (2016) studied the topic on soil stabilization using ceramic waste. Clayey soil was considered which was mixed with ceramic waste and experimental results are noted. With the addition of ceramic waste LL, PL, PI of the soil decreases. OMC decreases as the percentages of ceramic waste increases and MDD obtained at certain optimum content of ceramic waste and decrease beyond this optimum content of ceramic waste. CBR as by studies increases as percentages of ceramic waste dust increases. Swelling of clayey soil decreases as the percentages of ceramic waste increases.

Mir et.al (2016) this paper studied the potential use of sugarcane bagasse ash (SBA) as an additive for soil stabilization. Soil samples were taken from Chenab River to study the behavior of SBA with cement stabilized soil. Varying percentages of SBA (0%, 7.7%, 15%, 22.5%) and cement (0%, 3%, 6% by weight of the soil) were mixed with the soil. It indicates that there was improvement in CBR value with increase in SBA and cement content at 6%. It has been concluded that the cement treatment changes the behavior of natural soil to a good extent.

B. Sharma et.al (2016) had studied static method to determine compaction characteristics of fine-grained soils. The static compaction test was performed in 3 different soil thickness at a particular moisture content. A known weight of soil at a particular moisture content was placed into the mould up to a certain height and it was statically compacted in a standard proctor mould of capacity 1000cc with the help of cylindrical plunger. The height of the penetration of the metal plunger is measured from the top of the mould at a different load level where the load is applied at the rate of 1.25mm/min with proving ring constant of 0.05KN/div. It is observed that at lower static pressure significant variation in dry density is observed whereas an insignificant variation is observed at high static pressure. Moreover, no significant variation in dry unit weight is observed corresponding to different static pressure when the height of soil sample is altered. But the research was limited to soil sample with a maximum height of 100mm. The researcher also found that the different water content non-linear curve is observed when the relationship between static pressure and dry density is examined. A parabolic relationship is observed between moisture content and dry density at different static pressure where MDD obtained from static compaction test are found to be higher as compared to that of standard proctor test.

Baishakhi Debnath et.al. (2017) studied how the behavior of the soft soil having organic content of about 36.7% changes with the addition of fly ash. Standard Proctor compaction tests were carried out to determine the optimum moisture content and maximum dry density of all fly ash-soil mixtures. Cylindrical samples having a diameter of 38 mm and height of 76 mm, used in the UCS test, were prepared at their corresponding optimum moisture content and maximum dry density by static compaction. It is seen that both the liquid limit and the plastic limit increase with the increase of both types of fly ash content (Class C & Class F fly ash) but the plasticity index found to be decreased. The maximum dry density of the soil found to be decreased and it is said that the decrease in the maximum dry density is attributed to the agglomeration and flocculation of clay particles through cation exchange reaction, leading to the occupation of a larger space. Whereas, the optimum moisture content found to be increased with the increase in addition of fly ash. The unconfined compressive strength increases with the increasing percentages of fly ash content for both types of fly ash. The unconfined compressive strength also increases with the increase of curing period of 3,7 & 28days. Improvement with the addition of waste depends on the characteristics of organic soil as well as the properties and the amount of fly ash.

Dr. Afaf Ghais Abadi Ahmed (2014) studied in this paper about the effect of fly ash on stabilization of the soil. The soil type was found to be clay with intermediate plasticity and the percentages of fly ash used are 5%, 10%, 15%, 20%. The effect of the fly ash indicates that the liquid limit of the soil found to be decreased whereas the same thing observed in the case of plastic limit as well. Whereas the maximum dry density of the soil found to be increased up to 15% fly ash addition but in 20% it shows sharp declination. The optimum moisture content of the soil found to be decreased with the increase in fly ash content. Soaked CBR value considerably increased with the increase of ash content but slightly reduction of CBR value appears when soil ash mixture contains more than 15% ash.

R.K.Sharma (2020) this experimental study was performed to assess the efficiency of using fly ash and waste ceramic along with poor sand for clayey soil stabilization by evaluating compaction, strength and drainage properties to be used as road sub-grade material. The addition of sand to clayey soil decreased the optimum moisture content (OMC), increased the maximum dry density (MDD) whereas the California bearing ratio

(CBR) improved. Further, adding fly ash to clayey soil increased OMC value, decreased MDD value but improved the CBR value. The results indicate that adding ceramic tile waste reduced MDD value and increased OMC value and the CBR value. The drainage characteristics of composite material were better than those of clayey soil or fly ash. The stabilization of sub-grade resulted in significant savings in terms of the material required for subgrade of road pavement compared to those using un-stabilized soil.

S.Bhuvaneshwari et.al.(2005) this paper describes a study carried out to check the improvements in the properties of expansive soil with fly ash in varying percentages. Both laboratory trials and field tests have been carried out and results are reported in this paper. Laboratory tests includes Grain size analysis, Liquid limit (LL), Standard proctor test, Unconfined compressive strength test and permeability test. On addition of fly ash, LL found out to be decreased up to 20% after that it increases to 30% which is the same value as the untreated soil but the plastic limit was found out to be increased. In Standard Proctor test, the OMC of the soil found out to be decreased as well as the MDD of the soil. The UCS value was found out to be maximum in 10% fly ash addition is 353KN/m^2 whereas after that it decreases and the least value comes out to be at 50% fly ash addition. In the field test, an embankment measuring 3-4m wide, 30m long and 600m high has been constructed for carrying out the test. Each layer of 200mm thickness were placed with varying fly ash content in such a way that the layer of the fly ash gets sandwiched between two layers of soil to prevent it from flying off. After this, a disc harrow equipment was used for uniform mixing of soil and fly ash. This equipment is a circular disc, which penetrates through the loosely placed layers and pulled horizontally by a tractor. The discs rotate in such a way that the soil is shuffled and mixed thoroughly. After this, required quantity of water was manually sprayed over the layer to achieve the required moisture content of 15%, six passes of disc harrow were made for uniform mixing of additional water with the material already mixed. After 6 passes, the mixing of moisture was found to be uniform. Though a smooth wheel roller compaction is done and after mixing with fly ash, there was considerable improvement in the workability. Each layer of mix prepared was compacted with 8 passes of the roller. The material after compaction was found to be quite hard and no significant penetration of the roller wheel was noticed during the last 2 passes. After compaction the thickness of the layer (initial loose thickness of 200mm) was found to be 120 to 130mm. To check the adequacy of compaction, In-site density by core cutter, Natural moisture content, Light cone penetration tests were carried out by compaction in each layer. The tests carried out with different proportion of FA indicated that the workability is maximum with 25% FA also the dry density observed is maximum for 25% FA. It was concluded in the paper that the method used for compaction and FA sandwiched between two layers was found to be workable.

Hafez et.al. (2011) this paper studied about the determination of MDD and OMC by dynamic and static method of compaction. A new method has been invented to determine the MDD and shear strength of the soil by static and dynamic compaction methods. The static compaction onto soil specimen A has given the value of MDD = 1.84Mg/m^3, the amount of energy input was less and the shear strength value also increases to 366.5KPa. Whereas in dynamic compaction, MDD = 1.79 Mg/m^3, amount of energy input was high and shear strength value is 327KPa.The static compaction method reached higher degree of compaction at lower energy. Hafez (2007) defines the dynamic compaction energy contributing to different amount of energy to each layer. Whereas, in static compaction test the soil is given equal amount of energy keeping the homogeneity of the soil equal. Here in all the seven samples the MDD obtained by static compaction are higher compared to dynamic compaction. The UCS values are also found out to be higher compared to dynamic compaction values. In X rays image also, in dynamic compaction, the bottom layer seems to be denser compared to middle and upper layer. Whereas, in static compaction, X-rays image look smoother for the whole specimen.

R.K. Yadav and Satyendra Singh Rajput (2015) studied about black cotton soil and its change in behavior after addition of fly ash. As fly ash now a days used by many for the purpose of stabilization of soil for its easily availability and at the same time because of its disposal problem as well. Tests conducted includes liquid limit, plastic limit, free swell index, compaction test, California bearing ratio. Liquid limit of the untreated soil was found to be 55.2%, plastic limit was 28.1%. Fly ash percentage varies as 10%, 20%,30%,40% & 50%. It can be seen that the LL of the soil decreases from 55.2%-36.3% as the fly ash content increases. But the OMC of the soil increases from 19.3%- 24.1% and MDD of the soil decreases from 1.63%-1.52%. Free swell index of the soil also decreases on the addition of fly ash further. The MDD of the soil found out to be the least on 50% addition of waste. Maximum CBR of the soil found to be at 20% addition of fly ash. It has been found out from this paper is that on very expansive soil fly ash plays a very important role on CBR value and on free swell index as well.

Surender Roy (2021) studied about the use of additives in soil stabilization. Soil stabilization is a technique through which soil is made strong enough to bear the structures' load. The swell-shrink properties of expansive soils can be reduced and made useful for construction purposes using various stabilizing materials like cement, lime, fly ash, chemicals, etc. If the cement, lime, fly ash, and various chemicals are used in a suitable proportion, the geotechnical properties can be increased during stabilization of soils. In this paper, expansive soil, collapsible soil, and the influence of cement, lime, fly ash, and chemicals on the geotechnical properties of stabilized soils have been discussed. Expansive soils are those soils which have high swelling and shrinkage characteristics so these types of soil need to be treated well with stabilizing agents to make it good for construction purposes. Radhakrishnan et al., have reported that numerous techniques have been used to improve the properties of expansive soils, however, all these methods fall into two main categories namely, mechanical stabilization and chemical stabilization. The mechanical stabilization involves one of the following processes: replacement of expansive layer with an inert material, pre-wetting of soil, introduction of barrier systems to control moisture influx into expansive soil, etc. Chemical stabilization, on the other hand, involves the addition of chemicals such as cement, lime, bituminous emulsions, pozzolans, etc. to the soil. Chemicals such as lime, cement, fly ash are used in this study for better knowledge of soil stabilization. While treating the soil with lime they have found out that the swelling and shrinkage properties of the soil decreases with its addition. They carried out studies to find out the effect of fly ash-lime mix stabilization on unconfined compressive strength (UCS), and the California bearing ratio (CBR) of UCS increased as lime content increased. With the use of fly ash, it has been found out that it has hollow spheres of silicon, aluminum, and iron oxides, and unoxidized carbon. It reduces the plasticity index, activity, and swell potential as an expansive soil. Due to the cation exchange process, it agglomerates the fine clay parties into coarse particles. The stable exchangeable cations such as Ca2+, Al3+, and Fe3+, released by fly ash helps in the flocculation of the clay particles. Furthermore, the time-dependent cementation process (pozzolanic reaction) results in the formation of cemented compounds characterized by their high shear strength and low volume change. On the other hand, chemicals such as potassium chloride, calcium chloride, etc. are also used a stabilizing material and it was noted that it can be used as a subgrade material as it shows reduction in plasticity index.

Kumar. N. Darga et.al. (2010) studied about the behavior of fly ash and stone dust mixed as an admixture to the expansive soil in an increment percentage of 10%, 15%, 20%, 25%, 30% respectively and 10%, 20%, 30%, 40% and 50%. It has been found out in this paper that 30% stone dust and 25% fly ash are found to be optimum for strength improvement of the soil. Tests includes Liquid limit, plastic limit, heavy compaction, Free swell test, Unconfined compressive strength test, and California bearing ratio. It has been found out that on increasing the percentage of fly ash and stone dust the liquid limit of the soil decreases as well as the plastic limit of the soil also decreases. Whereas, it was also mentioned about the reduction of plasticity is more in terms of stone dust rather than fly ash. In terms of stone dust, the maximum dry density attained at 30% of its addition whereas the maximum dry density on addition of fly ash attained at 25%. Increase percentages of stone dust and fly ash in terms of maximum dry density are 15% and 7%. It is also concluded that the UCS value attained maximum strength on 50% addition of fly ash and stone dust and it was mentioned about its gain of strength and control over plasticity of the soil is very prominent. At the same time, the increase in CBR values were maximum i.e., 100% and 235% w.r.t. the untreated soil. It can be clearly seen from the paper that both stone dust and fly ash shows very prominent results on addition to soil.

CHAPTER 3
METHODOLOGY

3.1 General

This chapter represents different laboratory tests which were done on untreated soil and by using waste materials, apparatus and test procedures used in carrying out the project.

3.2 Test performed:

The experimental study is done to understand how the behavior of the fine-grained soil changes with the addition of both the waste. For comparison of the behavior of the soil tests of the untreated soil were also done. Along with the index properties of the soil strength tests were also done. For this purpose, tests include liquid limit, plastic limit, wet sieve analysis, specific gravity, Standard Proctor test, Unconfined compressive strength test (static and dynamic), California Bearing ratio (CBR). This experimental study is done to understand how the behavior of the soil changes w.r.t the above tests done.

3.3 Collection of soil sample:

In this experimental study one soil sample has been collected where two different wastes were mixed to identify how its behavior changes with the addition of waste. For collecting the fine-grained soil samples, the top (40-60) cm of the soil is removed so that there remains no organic matter attached with the soil specimen. About 80-90kg of soil sample is collected from Deepor Beel area, Guwahati. After the collection of the soil sample, it is allowed to dry at room temperature for few weeks. Oven dried samples are not used in this experimental study as during oven drying of the soil specimen the intermolecular attraction of the soil particles get destroyed easily in comparison with the air-dried soil samples. The soil used here are air-dried soil samples as oven dried samples results in disturbance of the soil structure.

Fig 3.1 Collection of soil sample from Deepor Beel area

Fig 3.2 Air dried soil sample after collection

3.4 Collection of waste materials:

The wastes that are used in this study are sugarcane bagasse and fly ash. Sugarcane bagasse has been collected from a local sugarcane selling vendor and after collecting it has been air-dried so that it can be cut into small pieces. The sugarcane bagasse was cut into small pieces of length ranging from **2.5cm-3cm** and breadth **0.3cm-0.5cm.** Secondly, fly ash has been collected from STD bricks Standard Products from Airport Road, Azara, Guwahati which is a manufacturing quality bricks industry. The fly ash was itself a fine powder of grey color during its collection so it has been used directly with the soil for testing.

Fig 3.3 Sugarcane bagasse cut into small pieces **Fig 3.4 Fly ash**

3.5 Description of tests performed:

Tests performed for the determination of the physical as well as the strength of the soil are according to the IS codes and are discussed below-

3.5.1 Sieve Analysis:

This test was performed according to the IS Code- Determination of gradation of the soil samples by wet sieve analysis according to **IS 2720 (Part 4)-1985.**

Sieve Analysis is generally done by two methods- Dry method and Wet method. Dry sieve method is performed when the soil retains on 4.75mm IS sieve after sieving. Whereas wet sieve analysis is performed on soil passing through 4.75mm IS sieve and retained on 75-micron IS sieve. Here only wet sieve analysis of the untreated soil is done because of the removal of the clay particles intact to it as the soil taken for testing is clayey soil. Sieves used in this methods are- 4.75mm, 2.36mm, 1.18mm, 600µ, 300µ, 150µ, 75µ. 200g oven dried soil sample passing through 4.75mm IS sieve is taken for this experiment. Sieve analysis is carried out to determine the Particle-Size Distribution of a material. Graph is plotted between Sieve size (mm) and % Finer which is obtained from the fine sieve analysis.

3.5.2 Liquid limit test by Static Cone Penetration

This test was performed according to the IS specification **IS:2720(Part 5)-1985**. Liquid limit (W_L) is the water content corresponding to the arbitrary limit between liquid limit and plastic limit. Liquid limit is defined as the minimum water content at which the soil is still in the liquid state, but has a small shearing strength against flowing. Graph between Cone penetration(x) and water content(y) should be plotted to determine the liquid limit of the soil. The water content corresponding to a cone penetration of 20mm is then taken as the liquid limit. The set of values used for the graphs are such that the penetration should be in between 14-28mm. The IS sieve used for performing this experiment is 425μ passing.

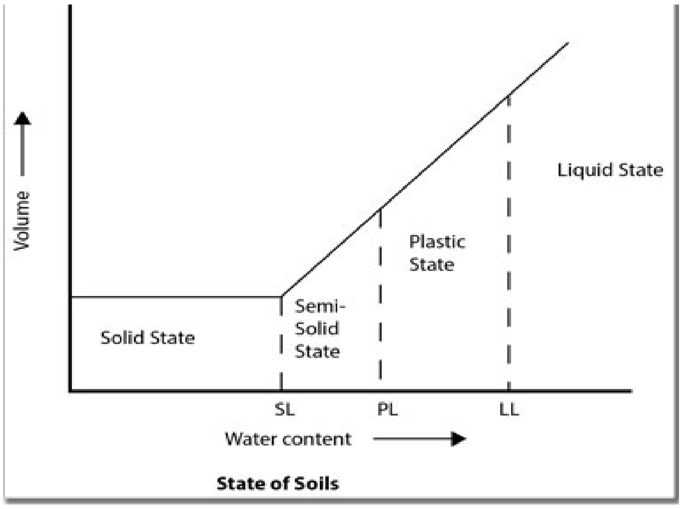

Fig 3.5 Consistency limits of soil **Fig 3.6 Liquid limit test**

(Source:https://www.globalgilson.com)

3.5.3 Determination of Plastic limit

This test was performed according to the IS specification **IS:2720(Part 5)-1985**. Plasticity is defined as the property of soil which allows it to be deformed rapidly, without rupture, without elastic rebound and without volume change. Plastic limit (W_P) is the water content between the plastic and the semi-solid states of consistency of soil. It is defined as the minimum water content at which the soil will just begin to crumble when rolled into a thread approximately 3mm in diameter. IS sieve in performing this experiment is 425μ passing. The plasticity index, $I_P = W_L - W_P$.

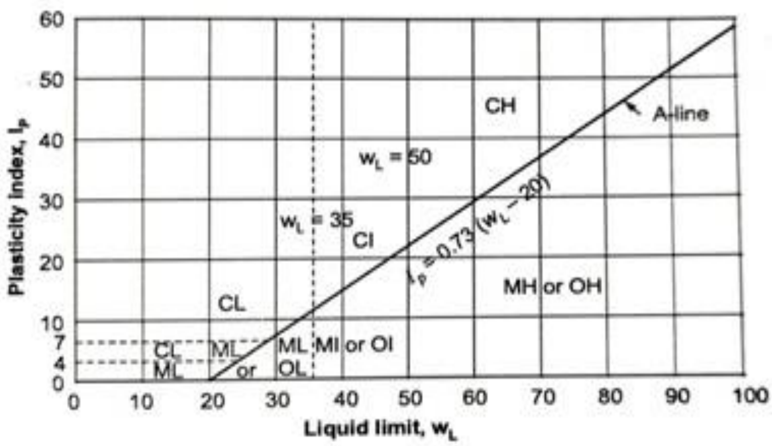

Fig 3.7 Plasticity Chart (Source: https://www.engineeringcivil.com/classification-of-soil.html)

Fig 3.8 Plastic Limit thread

3.5.4 Determination of Specific Gravity by Density Bottle

The specific gravity is performed as per **IS-2720 (Part 3/ Section 1)-1980**: Method of test of soil. Part-8 Determination of specific gravity, section-1 Fine grained soil. The specific gravity of soil particles is the mass density of soil to that of distilled water at the standard temperature of 27°C. It is the ratio between mass of the given volume of soil to that of equivalent volume of water. It is denoted by the symbol G. The apparatus required for performing this test includes density bottle of 50ml capacity, digital balance, vacuum desiccator, oven. The procedure includes the following steps:

- Firstly, the density bottle was cleaned and dried properly before conducting the test.
- The density bottle along with the stopper been weighed and demoted as

M_1.

- 5-10g of soil sample was taken in the density bottle and weigh the bottle along with the stopper as M_2.
- Now add distilled water to the soil in the density bottle upto the soil level and shake gently to mix soil and water.
- Now the stopper of density bottle was removed and placed in the vacuum desiccator and connect the vacuum pump.
- Take out the bottle after attaining constant temperature and dry the outer surface using cloth and weighed the bottle as a total of mass of bottle, soil and water as M_3.
- In the last step, bottle was emptied and filled solely with distilled water along with stopper and weighed as M_4.

The specific gravity is determined by the following equation-

$\mathbf{G} = M_2 - M_1/(M_4-M_1) - (M_3-M_2)$,

3.5.5 Compaction test:

3.5.5.1 Dynamic compaction method:

This test is performed to determine the relationship between the moisture content and the dry density of a soil for a specified compactive effort as per **IS: 2720 (Part 7)**. Compaction is the process of expelling the air from the soil sample by applying any mechanical energy. The expulsion of air from the soil reduces the porosity of the soil and thereby increases the density of the soil. This can be achieved by repetitive application of loads either in dynamic manner or static loading. Several methods are used for compaction like tamping, vibration, etc. Generally, two types of compaction test are performed as developed by R.R. Proctor are The Standard Proctor test and The Modified Proctor Test. In the Standard Proctor test, the soil is compacted by a 2.6kg rammer at a free fall of 310mm. The mould is filled with three layers and each layer is given 25 number of blows. Whereas in Modified Proctor test, a 4.89kg rammer is used at a free fall of 450mm along with the mould filled with five layers of soil. Proctor compaction tests are most commonly used to determine the compaction characteristics for proper control over the field compaction. These dynamic compaction tests are laborious and time consuming and also limitations are there in determination of maximum dry density and optimum moisture content. Thus, to improve the properties of the soil, compaction technique is adopted for the strength improvement of the soil. In this experimental study, standard proctor test has been carried. The soil samples were prepared at different water content of about 2kg each and kept it for 24 hours maturation.

Fig 3.9 Mixing of sample for proctor test

Fig 3.10 Hammering of the sample

Fig 3.11 Compaction of the soil with mould

3.5.5.2 Static compaction method:

In this method of compaction, all the specifications described for standard proctor test is followed but the difference is observed in the mode of compaction and amount to soil taken for the test.

The apparatus required for the test includes Standard proctor mould of 1000cc capacity, tamping rod, two metal plates of diameter 98mm and thickness 5mm and 16mm,4.75mm IS sieve, sampler and a measuring scale.

The air-dried soil sample taken for this laboratory test of about 1500g passing through 4.75mm IS sieve and is mixed with water content and kept for 24hr for maturation. The soil is then poured into the mould with along with two metal plates one above the other and slowly the soil has been tamped to a maximum height of 100mm or less.

 

Fig 3.12 Proctor mould, two metal plates, tamping rod

Fig 3.13 Proctor mould, collar& plate

Fig 3.14 Final image of height of compacted specimen

3.5.6 Unconfined Compressive Strength Test

This test is performed to determine the unconfined compressive strength (UCS) of the soil in the laboratory. The specimens used while doing the test are undisturbed, remolded or compacted specimen. The UCS (q_u) is the load per unit area at which a cylindrical specimen fails in compression without any confining pressure.

$$\mathbf{q_u = \frac{P}{A}}$$

where P = axial load at failure,

A = corrected area = $A_o/(1-\varepsilon)$

where A_o = initial cross-sectional area of the specimen.

ε = axial strain.

Table 3.1 Unconfined compressive strength of cohesive soil in terms of their consistency

Sl.no	Consistency of clay	Unconfined Compressive strength (KPa)
1	Very soft	≤ 25
2	Soft	25-50
3	Medium	50-100
4	Stiff	100-200
5	Very Stiff	200-400
6	Hard	≥ 400

Fig 3.15 Sample for UCS test

Fig 3.16 Unconfined Compression machine

Fig 3.17 The above pictures shows the sample after failure at different waste percentages

3.5.7 Determination of California Bearing Ratio (CBR):

In this experimental study, unsoaked CBR test are performed on the collected soil sample according to the IS specification **IS:2720 (Part-16)-1980**. The soil samples are prepared at optimum moisture content and kept it for 24hours maturation. The test was performed on remolded specimen by means of dynamic compaction.

3.5.7.1 Preparation of soil sample by dynamic compaction:

In this dynamic method of compaction, air-dried soil specimen of about 4.5kg is taken and mixed with thoroughly at OMC and kept it for maturation of 24hours. If the soil is to be compacted to the maximum dry density at the optimum moisture content determined from standard proctor test and the necessary quantity of water added so that the water content of the soil sample is equal to the determined optimum water content. The material used in the remolded specimen shall pass through 19mm IS sieve.

The mould with extension collar attached shall be clamped to the base plate. The spacer disc shall be inserted over the base plate and a disc of coarse filter paper placed on the top of the spacer disc. The soil-water mixture shall be compacted into the mould in accordance with the method applicable to the 150mm diameter mould specified by IS:2720 (Part 7)-1980 which means the test specimen is compacted in 3layers using 2.6kg rammer with a free fall of 31cm by giving 56 number of blows in each layer. The extension collar is removed and the compacted soil is trimmed carefully by means of a straightedge, any hole that develop on the surface of the compacted soil by the removal of coarse material, is patched with smaller size material. Then the mould is turned upside down and the base plate as well as the spacer disc is removed. The mass of the mould and the compacted soil specimen is recorded so that the bulk density and dry density can be determined. A disc of coarse filter paper shall be placed on the perforated base plate, the mould and the compacted soil shall be inverted and the perforated base plate clamped to the mould with the compacted soil in contact with the filter paper.

Fig 3.18 Compacted CBR mould with soil, spacer disc, surcharge

Fig 3.19 Compacted soil with CBR mould after load penetration

Fig 3.20 CBR mould, base plate, collar, spacer disc, surcharge, rammer

3.5.7.2 Penetration test:

The mould containing the specimen with the base plate in position but the top face exposed shall be placed on the lower plate of the testing machine. Surcharge weights, sufficient to produce an intensity of loading equal to the weight of the base material and pavement shall be placed on the specimen. If the specimen has been soaked previously, the surcharge shall be equal to that used during the soaking period. To prevent upheaval of soil into the hole of the surcharge weights, 2.5kg annular weight shall be placed on the soil surface prior to seating the penetration plunger after which the remainder of the surcharge weighs shall be placed. The plunger shall be placed under a load of 4kg so that full contact is established between the surface of the specimen and the plunger. The load and deformation gauge shall be set to zero. Load shall be applied to the plunger at the rate of 1.25mm/min. Readings of the load shall be taken as penetrations of 0.5, 1, 1.5, 2, 2.5, 4, 5, 7.5, 10 and 12.5mm. Corresponding to the penetration value of 2.5mm and 5mm the percentage CBR values of the soil specimen are recorded as well.

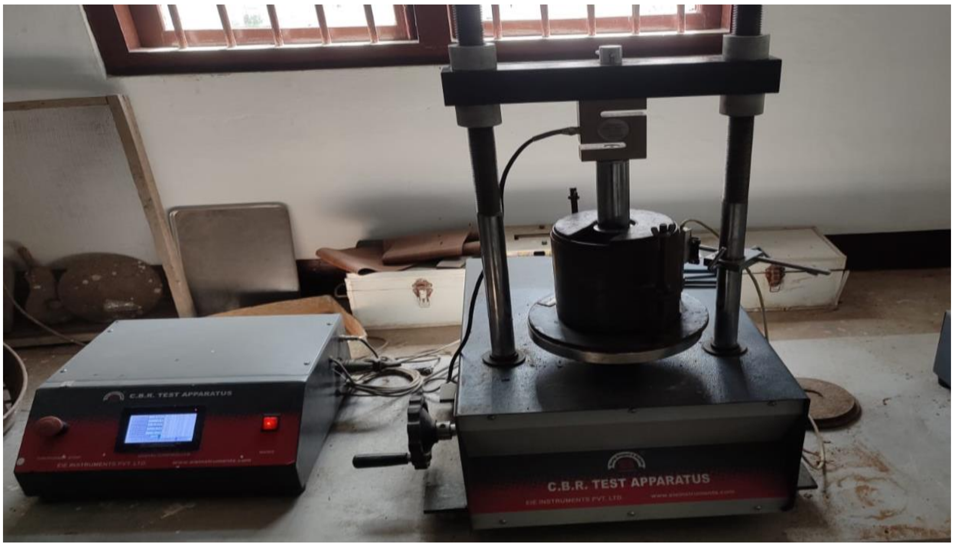

Fig 3.21 CBR test apparatus (digital set up)

At the end the plunger was raised and the mould was detached from the loading equipment. About 20-50g of soil sample is collected from the top 30mm layer of the specimen and the water content is determined as per **IS:2720 (Part 4)-1973**.

3.5.7.3 Load Penetration Curve:

This curve is usually convex upwards although the initial portion of the curve may be convex downwards due to surface initial portion of the curve may be convex downwards due to surface irregularities. A correction shall be applied by drawing a tangent to the point of greatest slope and then transposing the axis of the load so that zero penetration is taken as the point where the tangent cuts the axis of penetration.

3.5.7.4 California Bearing ratio (CBR):

The CBR values are usually calculated for penetration of 2.5mm and 5mm. Corresponding to the penetration values at which the CBR values is desired, corrected load value is taken from the load penetration curve and the CBR calculated as follows-

California Bearing Ratio = $P_T/P_S \times 100$

Where P_T = corrected unit (total) test load corresponding to the chosen penetration from the load penetration curve.

P_S = unit (or total) standard load for the same depth of penetration as for P_T taken from the table 3.2.

Table 3.2 Standard load used in CBR test

Penetration depth (mm)	Unit standard load (kg/cm^3)	Total standard load (kgf)
2.5	70	1370
5	105	2055

Generally, the CBR value at 2.5mm penetration is greater than 5mm penetration. Whenever, the CBR for 5mm exceeds that for 2.5mm, the test is repeated. If identical results follow, the CBR corresponding to 5mm penetration is reported as CBR value of the specimen.

CHAPTER 4
EXPERIMENTAL TEST RESULTS

4.1 Introduction

This chapter include the laboratory test results and findings from different tests carried out from the experiments of the untreated soil.

4.2 Observations and Calculations

4.2.1 Sieve Analysis

Total mass of oven dried sample = 200g

Table 4.1 Particle size distribution

Sieve size (mm)	Retained	% Retained	Cumulative % retained	% Finer
4.75	0	0	0	100
2	0.55	0.27	0.27	99.73
1.18	0.50	0.25	0.52	99.48
0.6	0.94	0.47	0.99	99.01
0.425	0.84	0.42	1.41	98.59
0.3	1.34	0.67	2.08	97.92
0.15	4.80	2.40	4.48	95.52
0.075	7.57	3.78	8.26	91.74

Sand = 100-91.74

= 8.26%

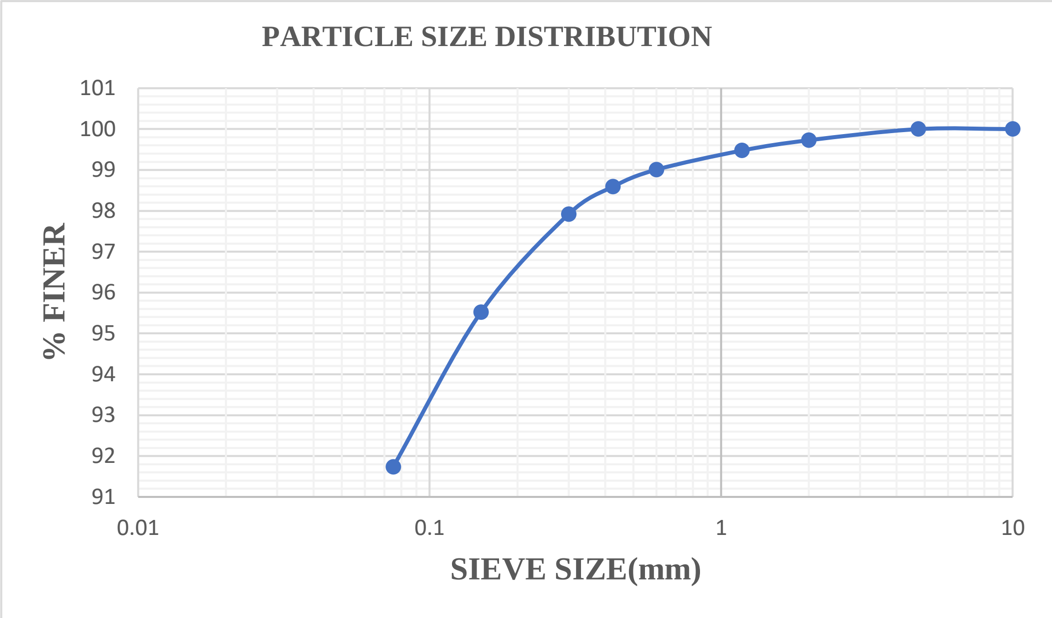

Fig 4.1 Particle size distribution curve

4.2.2 Liquid Limit Test

Total mass of the sample = 450g

Table 4.2 Values for water content determination for liquid limit

Cone penetration(mm)	Mass of empty container(g)	Mass of container with wet soil(g)	Mass of container with dry soil(g)	Mass of water(g)	Mass of dry soil(g)	Water content (%)
15	8.60	26.70	22.08	4.62	13.48	34.31
17	10.12	23.10	19.70	3.40	9.57	35.51
22	10.10	25.10	20.72	4.38	10.63	41.18
24	8.93	17.86	15.13	2.72	6.20	43.92
25	8.46	24.43	19.46	4.97	10.99	45.23

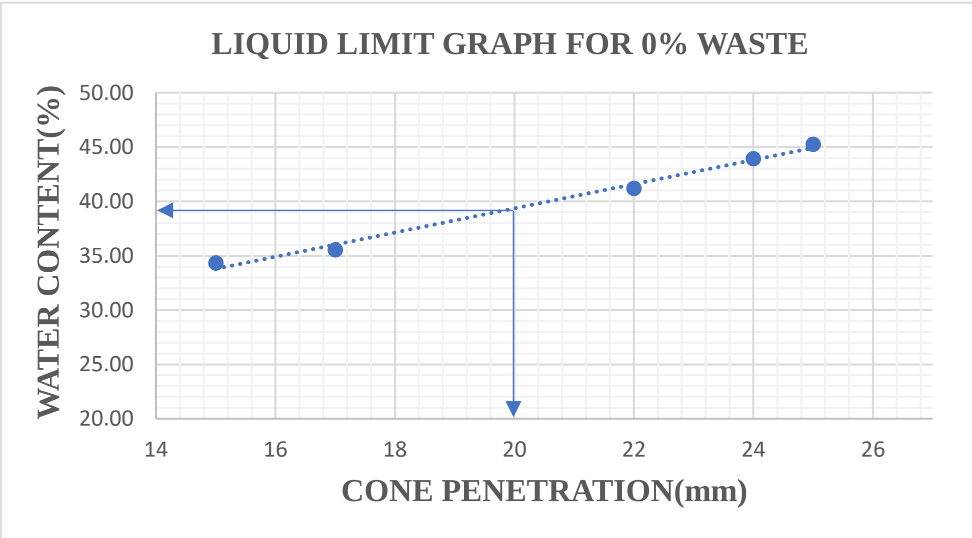

Fig 4.2 Liquid limit graph for 0% waste

Liquid Limit (LL) =39.1% (Fig 4.2)

4.2.3 Plastic limit test

Total mass of the sample taken = 450g

Table 4.3 Values for water content determination for plastic limit

Sl no.	Mass of empty container(g)	Mass of container with wet soil(g)	Mass of container with dry soil(g)	Mass of water(g)	Mass of dry soil(g)	Water content (%)
1	9.929	10.559	10.442	0.117	0.513	22.807
2	8.544	9.466	9.298	0.168	0.754	22.281
3	9.886	11.366	11.109	0.257	1.223	21.013

Plastic limit =22.8, from **Table 4.3**

Plasticity index =16.3.

A-line (PI)= 13.94.

Soil type = CI soil (Clay with Intermediate Plasticity)

4.2.4 Determination of specific gravity

Total mass of the sample taken = 5-10g.

Table 4.4 Specific gravity values

Density bottle no.	Mass of density bottle and stopper, M_1	Mass of density bottle, stopper and soil, M_2	Mass of density bottle, stopper, soil and water, M_3	Mass of density bottle, stopper and water, M_4	Specific gravity(G)
1	27.137	32.278	81.020	77.813	2.658
2	31.789	37.672	86.900	83.211	2.673
3	21.281	28.027	75.019	70.808	2.661

Specific gravity = 2.664

4.2.5 Standard proctor test

Diameter of the mould = 100mm.

Volume of the mould = 10000cc.

Height of the mould = 127.5mm.

Weight of the sample taken = 2kg.

Empty mould + base plate = 4234g.

Table 4.5 Standard Proctor Test values for 0% waste

Mass of compacted soil+mould with base plate(g)	Mass of empty container (g)	Mass of container with wet soil(g)	Mass of container with dry soil(g)	Mass of water (g)	Mass of dry soil (g)	Mass of compacted soil (g)	Bulk density (g/cc)	Water content (%)	Dry density (g/cc)
5312	10.001	26.769	23.931	2.838	13.93	1974	1.974	20.373	1.640
5362	9.857	28.274	25.051	3.223	15.194	2024	2.024	21.212	1.670
5342	8.714	27.732	24.257	3.475	15.543	2004	2.004	22.357	1.638
5292	9.552	30.88	26.912	3.968	17.36	1954	1.954	22.857	1.590
5288	8.292	25.883	22.562	3.321	14.27	1950	1.95	23.273	1.582

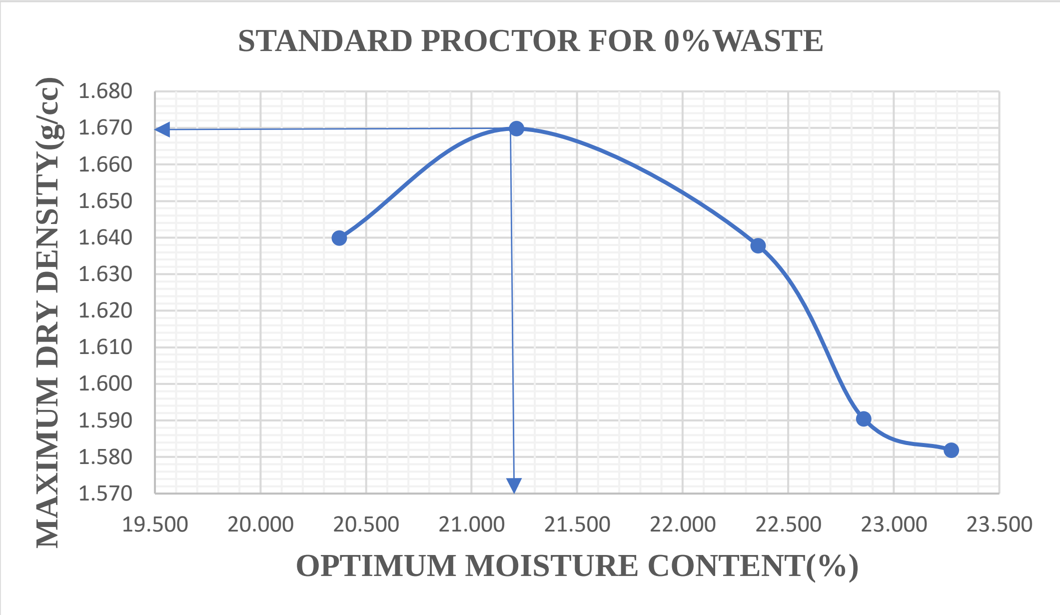

Fig 4.3 Compaction curve for 0% waste

Results – OMC (Optimum moisture content) = 21.2%. (**From Fig 4.3**)

MDD (Maximum dry density) = 1.67g/cc.

4.2.6 Unconfined compressive strength test

Initial diameter = 38mm.

Initial length =76mm.

Initial area = 11.341cm^2.

Soil specimen is mixed at OMC obtained from the Standard Proctor test.

Table 4.6 Unconfined Compression Test values for 0% waste (dynamic compaction)

Deformation dial reading	Axial deformation (mm)	Axial strain, ε	Area (cm^2)	Proving ring dial reading	Axial force (kg)	Compressive stress (KPa)
50	0.05	0.000658	11.35	22	7.26	62.69
100	0.1	0.001316	11.36	32	10.56	91.13
150	0.15	0.001974	11.36	41	13.53	116.68
200	0.2	0.002632	11.37	49	16.17	139.36
250	0.25	0.003289	11.38	56	18.48	159.16
300	0.3	0.003947	11.39	61	20.13	173.26
350	0.35	0.004605	11.39	68	22.44	193.02
400	0.4	0.005263	11.40	74	24.42	209.91
450	0.45	0.005921	11.41	78	25.74	221.11
500	0.5	0.006579	11.42	82	27.06	232.29
550	0.55	0.007237	11.42	84	27.72	237.80
600	0.6	0.007895	11.43	86	28.38	243.30
650	0.65	0.008553	11.44	88	29.04	248.79
700	0.7	0.009211	11.45	90	29.70	254.28
750	0.75	0.009868	11.45	91	30.03	256.93
800	0.8	0.010526	11.46	85	28.05	239.83
850	0.85	0.011184	11.47	80	26.40	225.58
900	0.9	0.011842	11.48	68	22.44	191.61

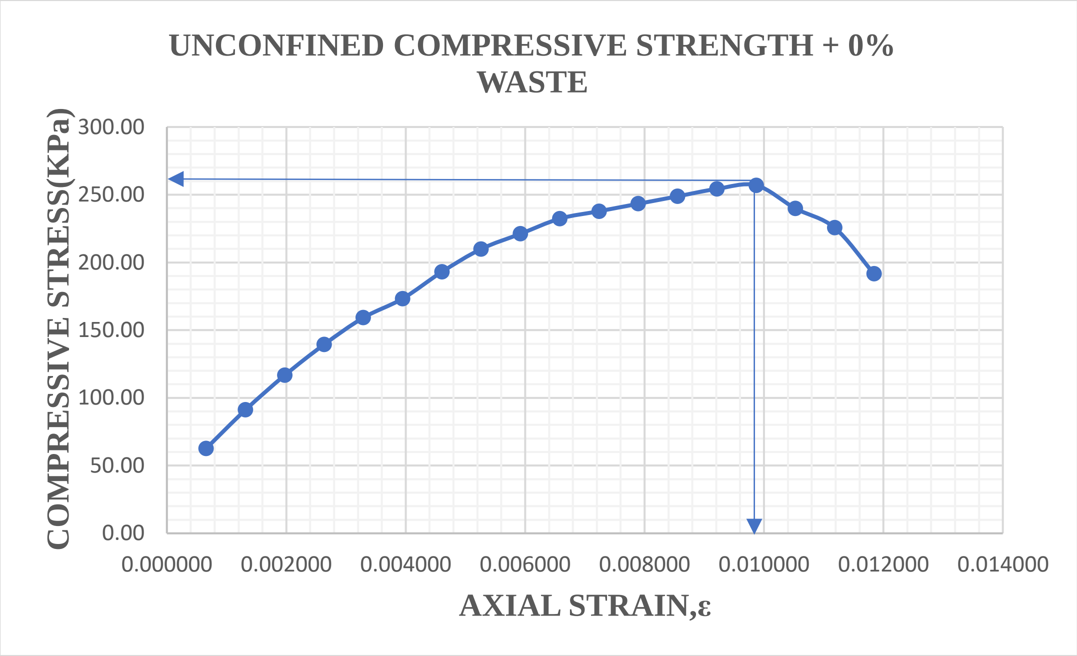

Fig 4.4 Unconfined compressive strength graph for 0% waste (Dynamic Compaction)

Compressive stress = 256.93 Kpa (From Fig 4.4)

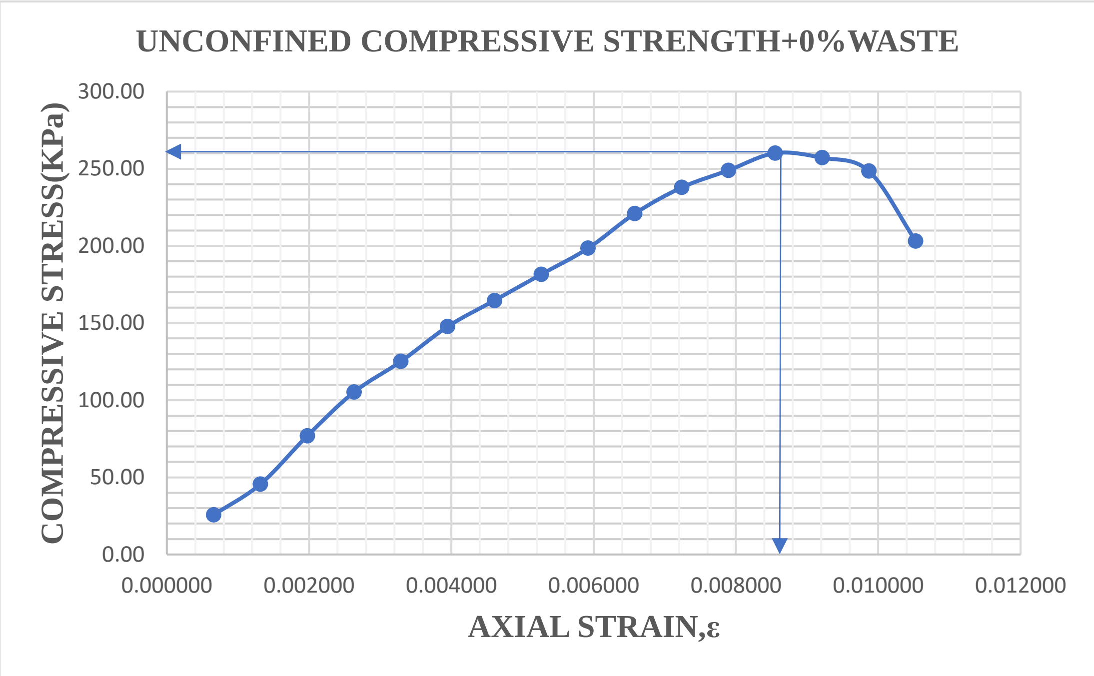

Fig 4.5 Unconfined compressive strength graph for 0% waste (Static Compaction)

Compressive stress = 260.10Kpa (From Fig 4.5).

4.2.7 CBR Test of untreated soil (0%waste)

Table 4.7 Load penetration data for CBR test of untreated soil

LOAD ON PISTON (kg)	PENETRATION (mm)
0	0
37.9	0.5
43.1	1
47.6	1.5
52.7	2
57.7	2.5
62.9	3
72.3	4
81.9	5
103.5	7.5
122.2	10
137.5	12.5

2.5mm CBR value = 4.21%

5mm CBR value = 3.99%

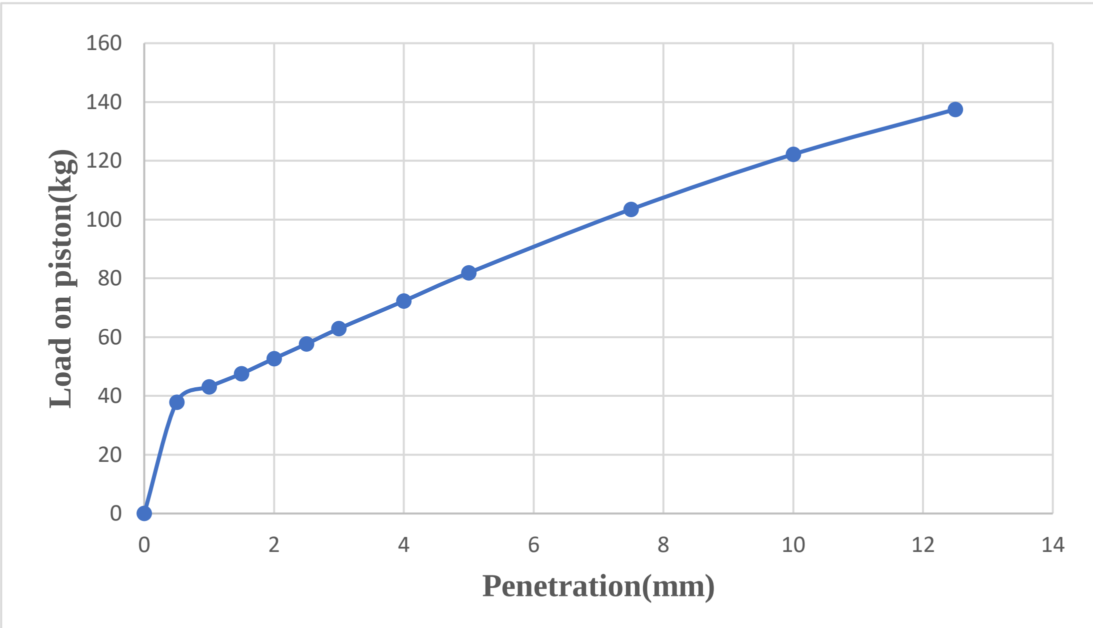

Fig 4.6 Load penetration curve for untreated soil

CHAPTER 5
TEST RESULTS WITH ADDITION OF WASTE

5.1 Introduction

This chapter represents different percentages of sugarcane bagasse added to the soil and fly ash corresponding results obtained are discussed below-

5.2 Tests results of soil mixed with sugarcane bagasse

Table 5.1 Standard Proctor Test for 2% SB

Empty mould + base plate = 4234g

Mass of compacted soil+mould with base plate(g)	Mass of empty container(g)	Mass of container with wet soil(g)	Mass of container with dry soil(g)	Mass of water (g)	Mass of dry soil (g)	Mass of compacted soil(g)	Bulk density (g/cc)	Water content (%)	Dry density (g/cc)
4890	9.123	28.919	26.919	2.000	17.796	656	0.656	11.24	0.590
5592	8.993	36.659	33.54	3.119	24.547	1358	1.358	12.71	1.205
6392	9.399	33.564	30.519	3.045	21.12	2158	2.158	14.42	1.886
6746	8.62	29.598	26.598	3.000	17.978	2512	2.512	16.69	2.153
6340	10.124	34.584	30.707	3.877	20.583	2106	2.106	18.84	1.772

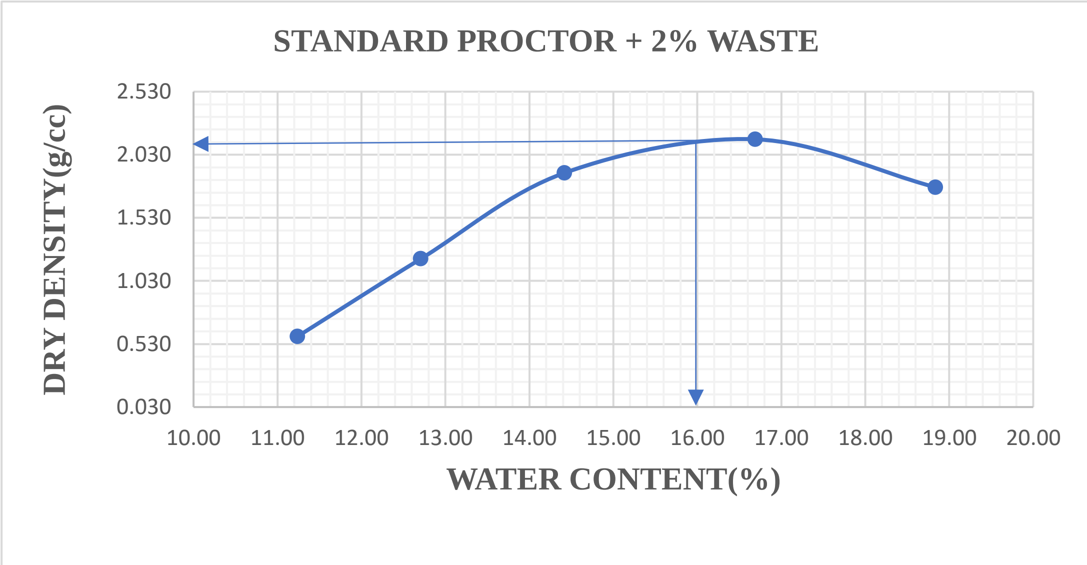

Fig 5.1 Compaction curve for 2% waste (SB)

Here from **Fig 5.1** the optimum moisture content of the soil while adding 2% waste decreases to 16% and the MDD increases to 2.153g/cc. This means that the strength of the soil increases compared to the natural soil with less water content than 0% waste.

Table 5.2 Standard Proctor Test for 4% SB

Empty mould + base plate = 3336g

Mass of compacted soil+mould with base plate(g)	Mass of empty container (g)	Mass of container with wet soil(g)	Mass of container with dry soil(g)	Mass of water (g)	Mass of dry soil (g)	Mass of compacted soil(g)	Bulk density (g/cc)	Water content (%)	Dry density (g/cc)
3960	9.247	32.912	30.121	2.791	20.874	624	0.62	13.37	0.55
4764	10.176	34.427	31.215	3.212	21.039	1428	1.43	15.27	1.24
5660	11.213	36.142	32.423	3.719	21.210	2324	2.32	17.53	0.13
5212	11.012	38.108	33.037	5.071	22.025	1876	1.88	23.02	0.08
4892	10.128	37.152	31.456	5.696	21.328	1556	1.56	26.71	0.06

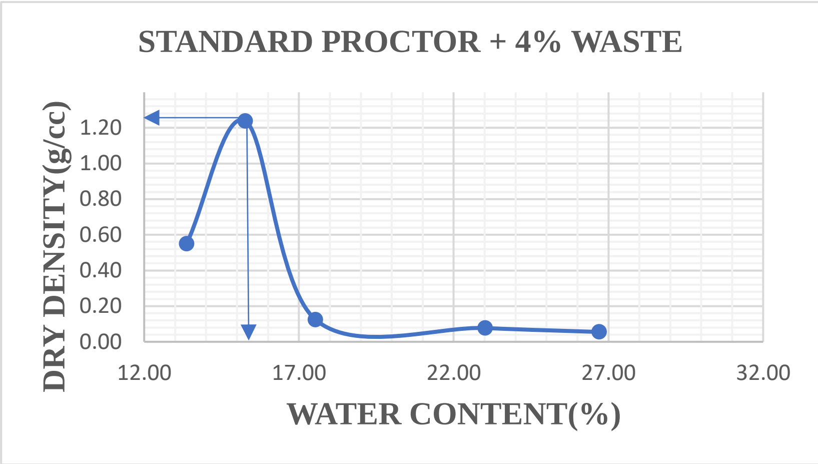

Fig 5.2 Compaction curve for 4% waste (SB)

From **Fig 5.2** The MDD attained on 4% waste is 1.21g/cc which is less compared to the 2% waste but the OMC of the soil decreases to 15.3% compared to 2% waste which is a positive conclusion. This means that the soil needs less water content to attain maximum strength but the strength is not more which we got earlier in 2% waste.

Table 5.3 Standard Proctor Test for 6% waste

Empty mould + base plate = 3336g

Mass of compacted soil+mould with base plate(g)	Mass of empty container(g)	Mass of container with wet soil(g)	Mass of container with dry soil(g)	Mass of water (g)	Mass of dry soil (g)	Mass of compacted soil(g)	Bulk density (g/cc)	Water content (%)	Dry density (g/cc)
4360	9.905	22.728	21.035	1.693	11.13	1028	1.028	15.21	0.89
5234	10.128	32.775	29.483	3.292	19.355	1902	1.902	17.01	1.63
5168	9.393	35.637	31.152	4.485	21.759	1836	1.836	20.61	1.52
5160	10.097	32.57	28.484	4.086	18.387	1828	1.828	22.22	1.50
5154	7.588	25.591	22.047	3.544	14.459	1822	1.822	24.51	1.46

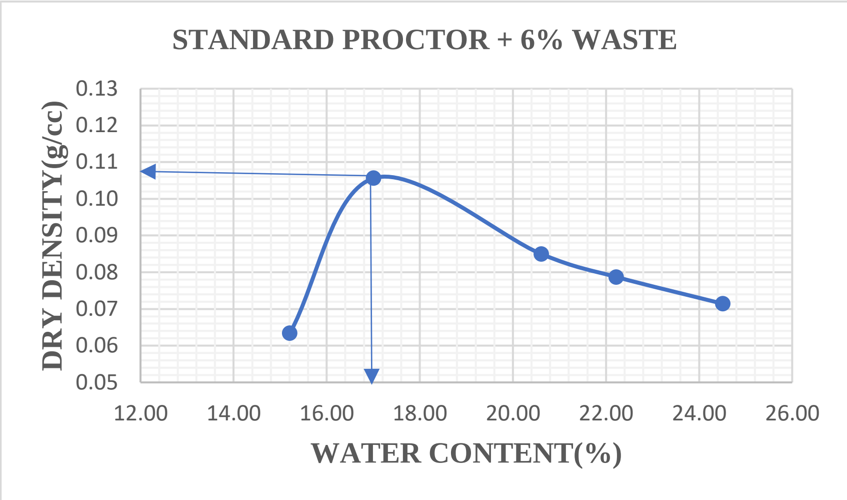

Fig 5.3 Compaction curve for 6% waste (SB)

Here From **Fig 5.3** the MDD attained is 1.66 g/cc which is more than 4% waste addition that means the strength is maximum attained but the water content needed for the maximum dry density has increased to 17%.

Table 5.4 Standard Proctor Test for 8% SB

Empty mould + base plate = 3336g

Mass of compacted soil+mould with base plate(g)	Mass of empty container (g)	Mass of container with wet soil(g)	Mass of container with dry soil(g)	Mass of water (g)	Mass of dry soil (g)	Mass of compacted soil(g)	Bulk density (g/cc)	Water content (%)	Dry density (g/cc)
4342	8.821	30.121	27.42	2.701	18.599	1010	1.01	14.52	0.882
5120	10.21	33.512	30.121	3.391	19.911	1788	1.788	17.03	1.528
4952	9.843	31.672	27.858	3.814	18.015	1620	1.62	21.17	1.337
4580	7.298	31.003	26.578	4.425	19.28	1248	1.248	22.95	1.015
4232	10.121	32.462	27.761	4.701	17.64	900	0.900	26.65	0.033

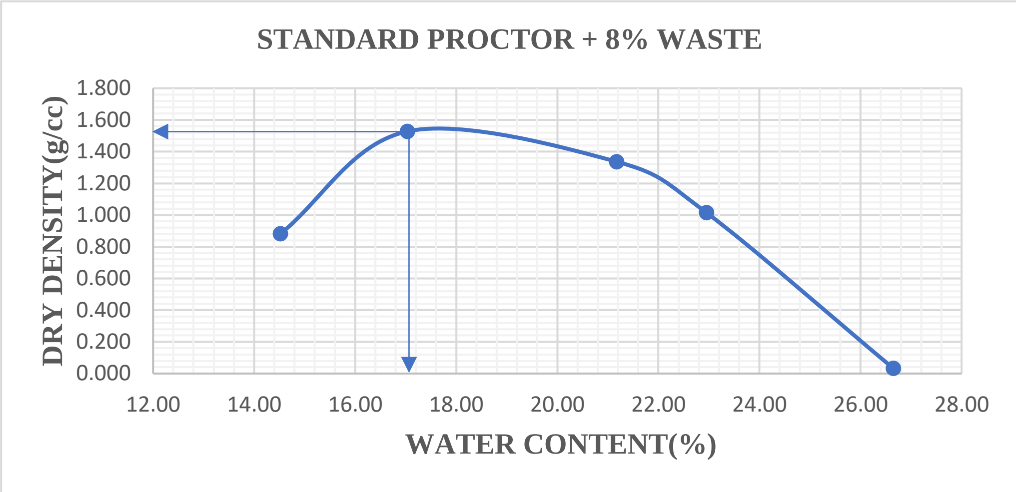

Fig 5.4 Compaction curve for 8% waste (SB)

Here From **Fig 5.4** the maximum dry density comes out to be 1.590g/cc which is less than 6% waste addition and OMC has also increased to 17.1%. That means that maximum strength of the soil decreases whereas the water content increases.

Table 5.5 Maximum dry density V/s percentage of sugarcane bagasse

Percentage of waste increment (%)	Maximum dry density(g/cc)
0%	1.67
2%	2.153
4%	1.210
6%	1.660
8%	1.590

Results: Maximum dry density attained at 2% waste.

Whereas after that the maximum dry density decreases. This makes a statement that minimum waste addition should be 2% because after that the dry density decreases. To attain maximum strength of the soil for stabilization, minimum of 2% waste addition is necessary.

Table 5.6 Unconfined Compression Test for 2% SB (dynamic compaction)

Deformation dial reading	Axial deformation (cm)	Axial strain, ε	Area(cm^2)	Proving ring dial reading	Axial force (kg)	Compressive stress (KPa)
50	0.05	0.000658	11.35	16	5.28	45.60
100	0.1	0.001316	11.36	24	7.92	68.35
150	0.15	0.001974	11.36	28	9.24	79.69
200	0.2	0.002632	11.37	32	10.56	91.01
250	0.25	0.003289	11.38	37	12.21	105.16
300	0.3	0.003947	11.39	42	13.86	119.29
350	0.35	0.004605	11.39	46	15.18	130.57
400	0.4	0.005263	11.40	50	16.50	141.83
450	0.45	0.005921	11.41	54	17.82	153.07
500	0.5	0.006579	11.42	58	19.14	164.30
550	0.55	0.007237	11.42	62	20.46	175.52
600	0.6	0.007895	11.43	66	21.78	186.72
650	0.65	0.008553	11.44	70	23.10	197.90
700	0.7	0.009211	11.45	74	24.42	209.07
750	0.75	0.009868	11.45	80	26.40	225.88
800	0.8	0.010526	11.46	86	28.38	242.66
850	0.85	0.011184	11.47	92	30.36	259.41
900	0.9	0.011842	11.48	88	29.04	247.97
950	0.95	0.012500	11.48	78	25.74	219.64
1000	1	0.013158	11.49	74	24.42	208.24

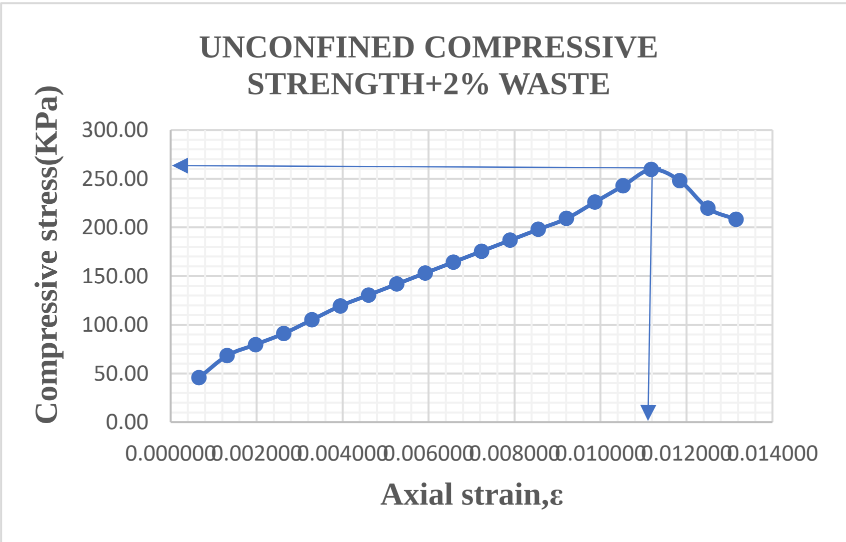

Fig 5.5 Unconfined compressive strength graph for 2% SB

From **Fig 5.5** The compressive stress for 2% waste mix was found to be 259.41KPa which has increased from the untreated soil. The increase in percentage for this mix was found to be 2.48% of the untreated soil and this is the maximum compressive stress attained compared to untreated soil as well as from the waste mixes.

Table 5.7 Unconfined Compression Test for 4% SB (dynamic compaction)

Deformation dial reading	Axial deformation(cm)	Axial strain, ε	Area (cm^2)	Proving ring dial reading	Axial force (Kg)	Compressive stress (KPa)
50	0.05	0.00066	11.35	18	5.94	51.30
100	0.1	0.00132	11.36	26	8.58	74.04
150	0.15	0.00197	11.36	32	10.56	91.07
200	0.2	0.00263	11.37	38	12.54	108.08
250	0.25	0.00329	11.38	44	14.52	125.06
300	0.3	0.00395	11.39	50	16.5	142.02
350	0.35	0.00461	11.39	56	18.48	158.95
400	0.4	0.00526	11.40	60	19.8	170.20
450	0.45	0.00592	11.41	65	21.45	184.26
500	0.5	0.00658	11.42	68	22.44	192.63
550	0.55	0.00724	11.42	72	23.76	203.83
600	0.6	0.00789	11.43	76	25.08	215.01
650	0.65	0.00855	11.44	82	27.06	231.83
700	0.7	0.00921	11.45	84	27.72	237.33
750	0.75	0.00987	11.45	86	28.38	242.82
800	0.8	0.01053	11.46	88	29.04	248.30
850	0.85	0.01118	11.47	90	29.7	253.77
900	0.9	0.01184	11.48	86	28.38	242.33
950	0.95	0.01250	11.48	78	25.74	219.64
1000	1	0.01316	11.49	72	23.76	202.61

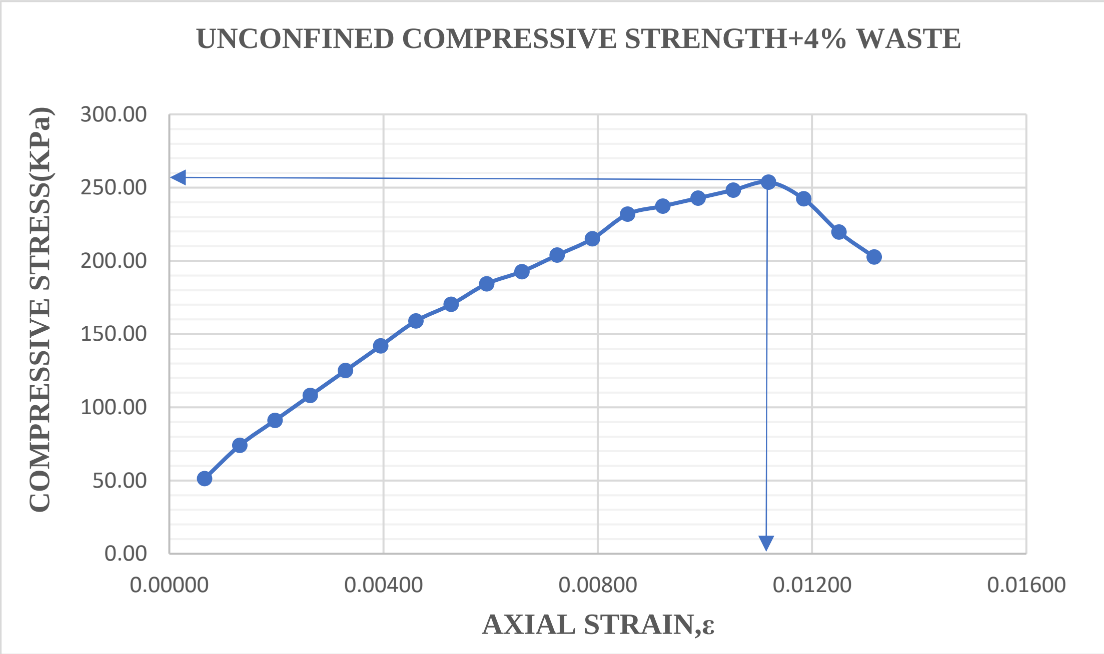

Fig 5.6 Unconfined compressive strength graph for 4% SB

From **Fig 5.6** The maximum compressive stress was found to be 253.77KPa for this waste mix. This has decreased from the 2% waste mix.

Table 5.8 Unconfined Compression Test for 6% SB (dynamic compaction)

Deformation dial reading	Axial deformation (cm)	Axial strain, ε	Area(cm^2)	Proving ring dial reading	Axial force (kg)	Compressive stress (KPa)
50	0.05	0.00066	11.35	13	4.29	37.05
100	0.1	0.00132	11.36	22	7.26	62.65
150	0.15	0.00197	11.36	28	9.24	79.69
200	0.2	0.00263	11.37	34	11.22	96.70
250	0.25	0.00329	11.38	38	12.54	108.00
300	0.3	0.00395	11.39	42	13.86	119.29
350	0.35	0.00461	11.39	46	15.18	130.57
400	0.4	0.00526	11.40	52	17.16	147.50
450	0.45	0.00592	11.41	58	19.14	164.41
500	0.5	0.00658	11.42	64	21.12	181.30
550	0.55	0.00724	11.42	68	22.44	192.51
600	0.6	0.00789	11.43	74	24.42	209.35
650	0.65	0.00855	11.44	78	25.74	220.52
700	0.7	0.00921	11.45	80	26.4	226.03
750	0.75	0.00987	11.45	81	26.73	228.70
800	0.8	0.01053	11.46	82	27.06	231.37
850	0.85	0.01118	11.47	83	27.39	234.04
900	0.9	0.01184	11.48	78	25.74	219.79
950	0.95	0.01250	11.48	72	23.76	202.75
1000	1	0.01316	11.49	68	22.44	191.36

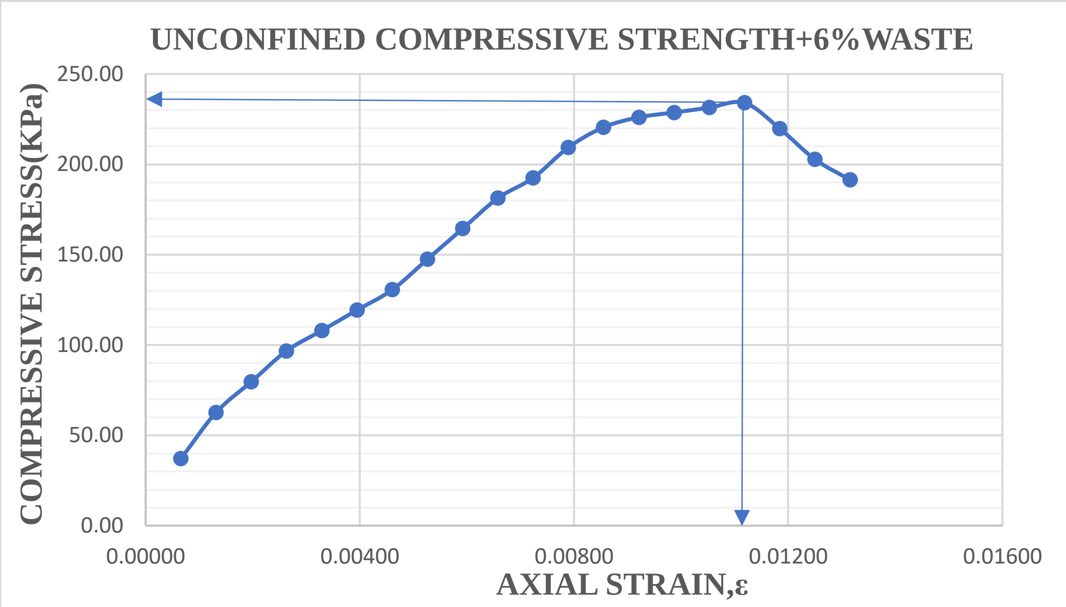

Fig 5.7 Unconfined compressive strength graph for 6% SB

From **Fig 5.7** the maximum compressive stress was found to be 234.04KPa for this waste mix. The decrease in percentage was found to be 22.89% of the untreated soil which has decreased from 2% as well from 4% waste mix which means that the strength of the soil found out to be decreasing.

Table 5.9 Unconfined Compression Test for 8% SB (dynamic compaction)

Deformation dial reading	Axial deformation (cm)	Axial strain, ε	Area (cm^2)	Proving ring dial reading	Axial force(kg)	Compressive stress (KPa)
50	0.05	0.00066	11.35	2	0.66	5.70
100	0.1	0.00132	11.36	5	1.65	14.24
150	0.15	0.00197	11.36	7	2.31	19.92
200	0.2	0.00263	11.37	10	3.3	28.44
250	0.25	0.00329	11.38	12	3.96	34.11
300	0.3	0.00395	11.39	14	4.62	39.76
350	0.35	0.00461	11.39	15	4.95	42.58
400	0.4	0.00526	11.40	17	5.61	48.22
450	0.45	0.00592	11.41	19	6.27	53.86
500	0.5	0.00658	11.42	22	7.26	62.32
550	0.55	0.00724	11.42	23	7.59	65.11
600	0.6	0.00789	11.43	25	8.25	70.73
650	0.65	0.00855	11.44	27	8.91	76.33
700	0.7	0.00921	11.45	30	9.9	84.76
750	0.75	0.00987	11.45	34	11.22	96.00
800	0.8	0.01053	11.46	38	12.54	107.22
850	0.85	0.01118	11.47	44	14.52	124.07
900	0.9	0.01184	11.48	38	12.54	107.08
950	0.95	0.01250	11.48	32	10.56	90.11

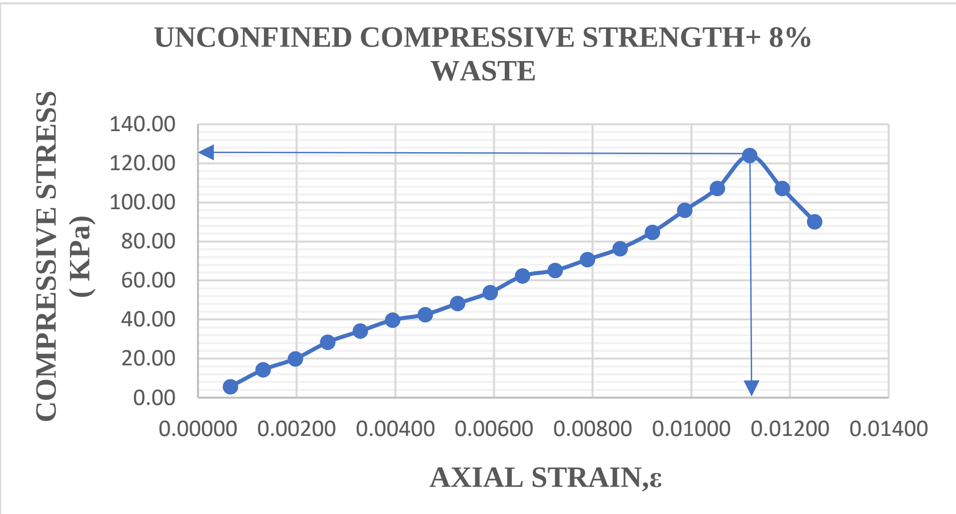

Fig 5.8 Unconfined compressive strength graph for 8% SB

Here it can be seen from **Fig 5.8** that the maximum stress that the soil can take is 124.07KPa which is the least among the other mixes and the strength has decreased 132.86% of the 0% waste mix.

Table 5.10 Unconfined compressive stress of the soil V/s Percentage of sugarcane bagasse(waste)

Percentages of waste increment (%)	Unconfined compressive stress (KPa)
0%	256.93
2%	259.41
4%	253.77
6%	234.04
8%	124.07

Results- Maximum strength attained at 2% waste mix. This means that minimum 2% waste should be added to the soil to attain maximum compressive strength because after that the strength decreases.

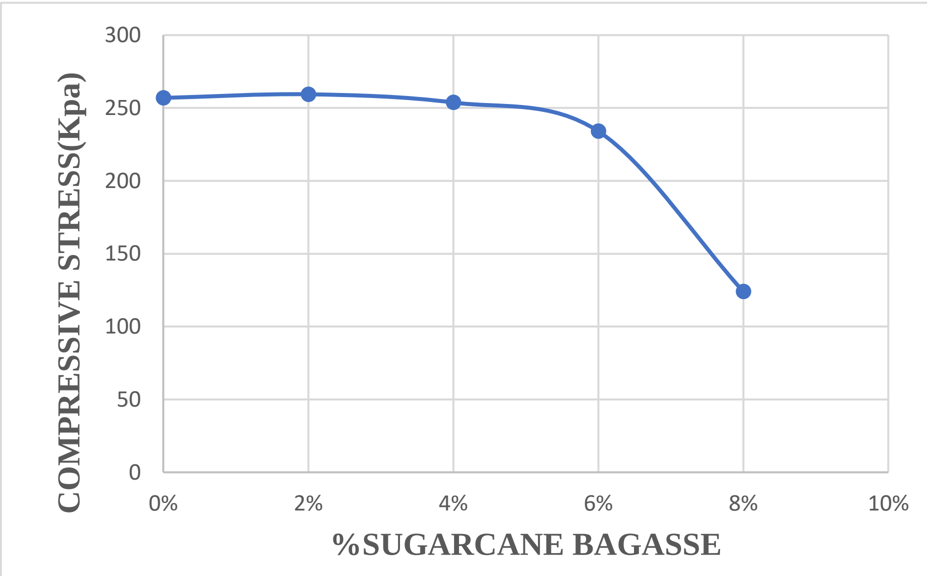

Fig 5.9 Compressive stress v/s % sugarcane bagasse

As it can be seen from the **Fig 5.9** that the compressive stress value was found to be maximum at 2% waste addition after that the value declines and at 8% waste value was found to be the least.

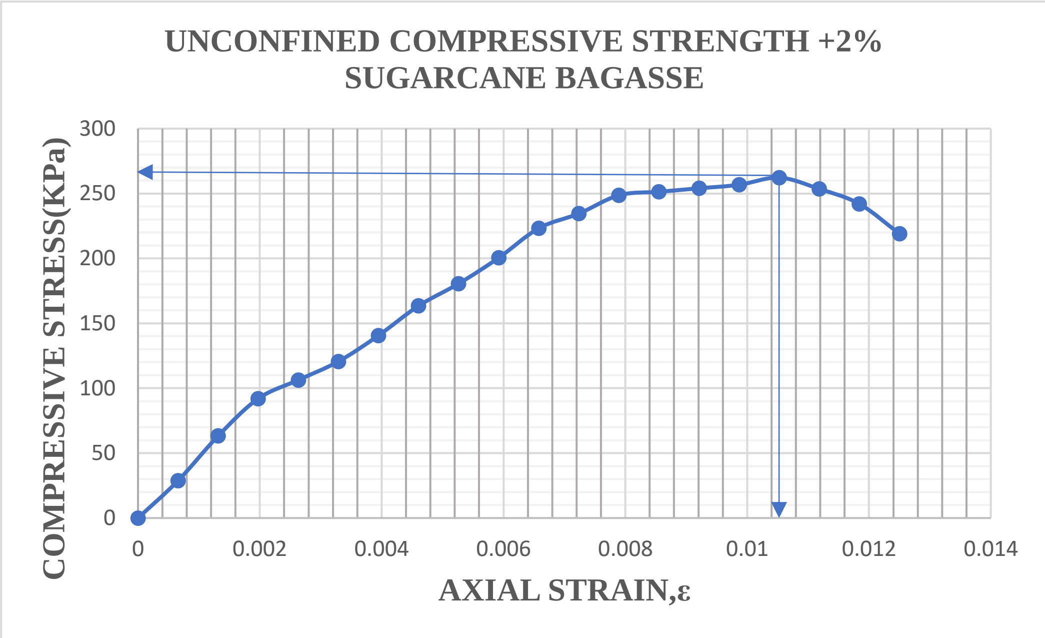

Fig 5.10 Unconfined compressive strength graph for 2% SB (static compaction)

lmkFrom the above **Fig 5.10** it can be seen that on statically compacted sample of UCS, the compressive stress was found out to be 262.23KPa. Compared to the dynamic compaction, static compaction value for UCS was found to be more.

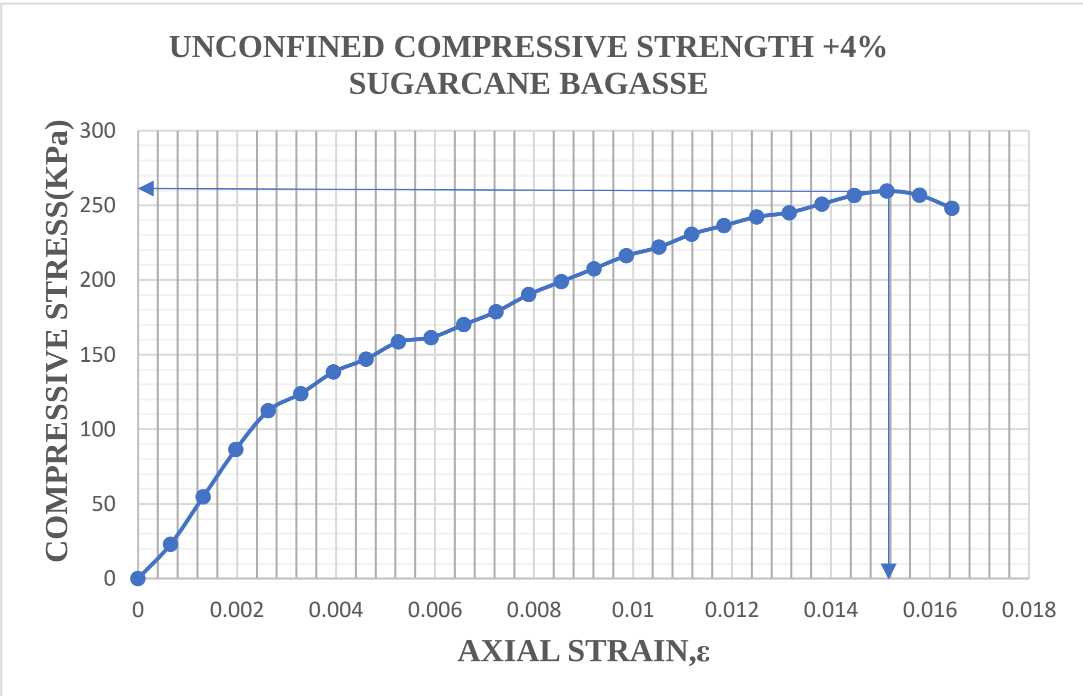

Fig 5.11 Unconfined compressive strength graph for 4% SB (static compaction)

As it can be seen from the above **Fig 5.11** for statically compacted sample the compressive stress of the sample was found out to be 259.60Kpa which gave a nearest value compared to the dynamic compaction and the change rate of the graph was also found out to be very smooth.

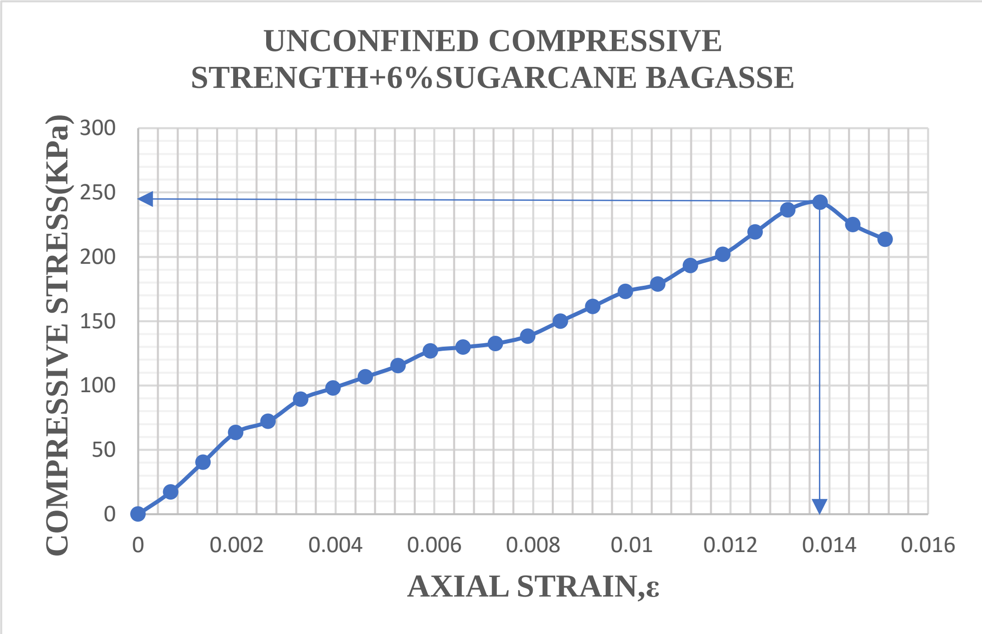

Fig 5.12 Unconfined compressive strength graph for 6% SB (static compaction)

As it can be seen from the above **Fig 5.12** the compressive stress was found out to be 242.27Kpa which decreases from the 4% waste addition and from the untreated soil as well.

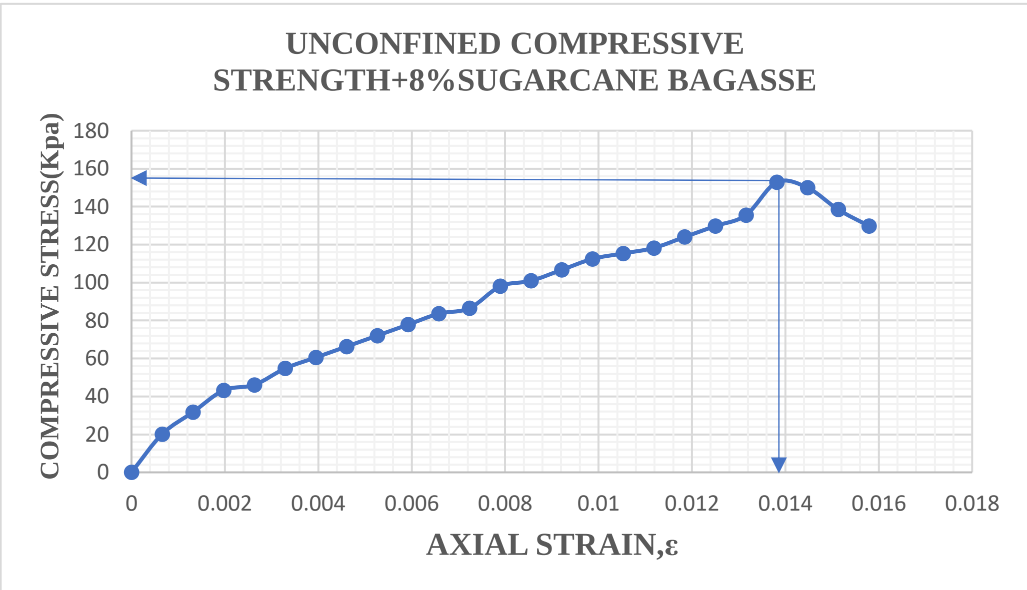

Fig 5.13 Unconfined compressive strength graph for 8% SB (static compaction)

As it can be seen from the **Fig 5.13**, for statically compacted sample of UCS, the compressive stress was found out to be 152.68Kpa which is the lowest amongst all the percentages.

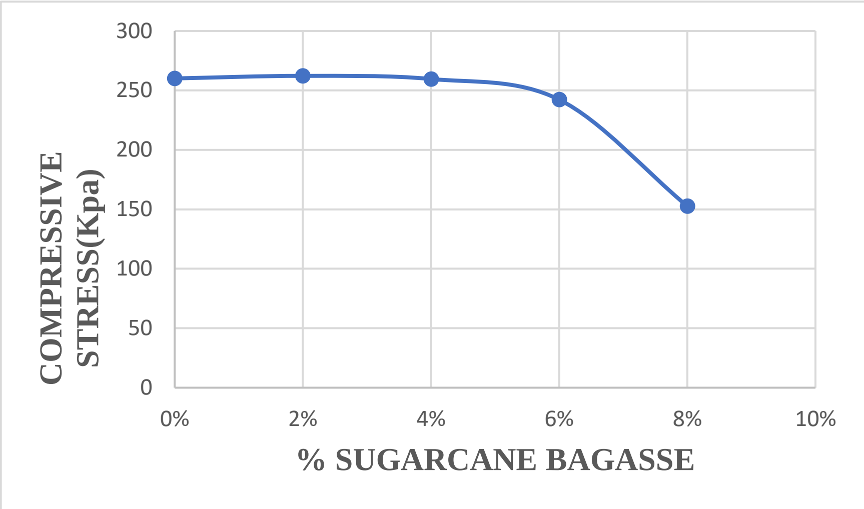

Fig 5.14 Compressive stress v/s % sugarcane bagasse

From the above **Fig 5.14** the maximum compressive stress was found out to be at 2% waste addition. So, optimum of 2% waste has to be added to the specimen to attain maximum strength as on increasing waste percentage after 2%, the strength of the soil found out to be decreased.

Table 5.11 Load penetration data for CBR test for 0.5% sugarcane bagasse

PENETRATION(mm)	LOAD ON PISTON (kg)
0	0
0.5	76.1
1	90.1
1.5	102.4
2	115.1
2.5	127.7
3	139
4	159.4
5	180.7
7.5	220.4
10	252.9
12.5	281.9

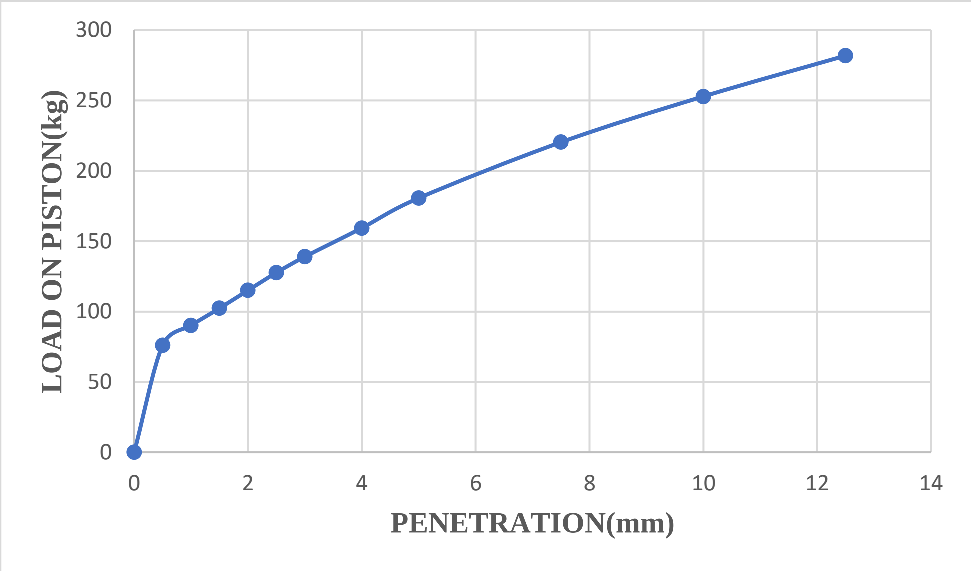

Fig 5.15 Load penetration graph for 0.5% addition of sugarcane bagasse

2.5=9.32%
5=8.79%

It can be seen from the Fig **5.15**, the CBR value at 2.5mm was found to be more compared to the 5mm CBR value.

Table 5.12 Load penetration data for CBR test for 1% sugarcane bagasse

PENETRATION(mm)	LOAD ON PISTON (kg)
0	0
0.5	84.1
1	97.1
1.5	109.2
2	121.2
2.5	134
3	146.2
4	169.1
5	194.3
7.5	255.2
10	306.8
12.5	348.5

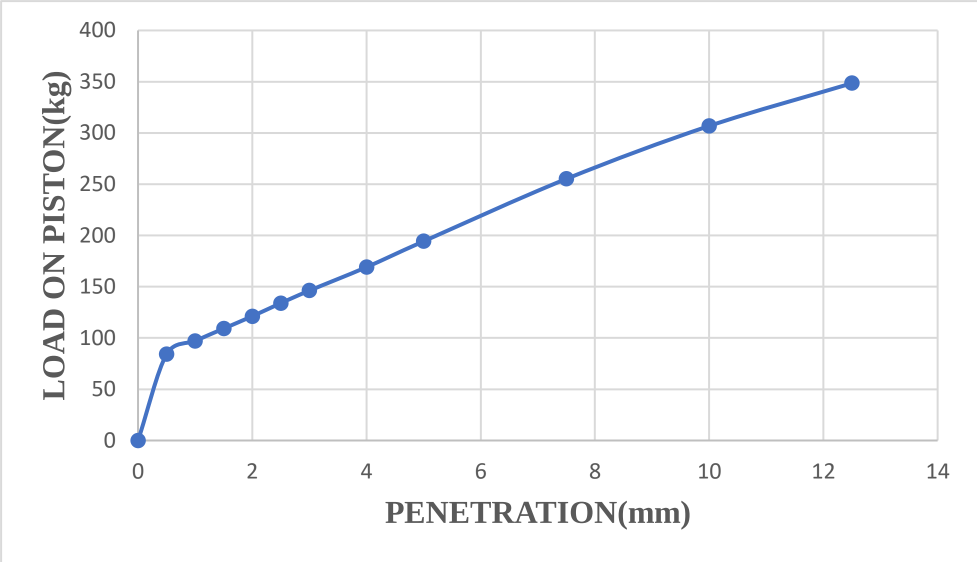

Fig 5.16 Load penetration graph for 1% addition of sugarcane bagasse

2.5=9.78%

5=9.45%

From the above **Fig 5.16**, the CBR of the soil found out to be more in case of 2.5mm penetration and it increases from 0.5% of addition of waste.

Table 5.13 Load penetration data for CBR test for 1.5% sugarcane bagasse

PENETRATION(mm)	LOAD ON PISTON (kg)
0	0
0.5	48.1
1	75.8
1.5	96.1
2	112.2
2.5	128
3	142.8
4	167.2
5	190.2
7.5	236.7
10	277.8
12.5	314.7

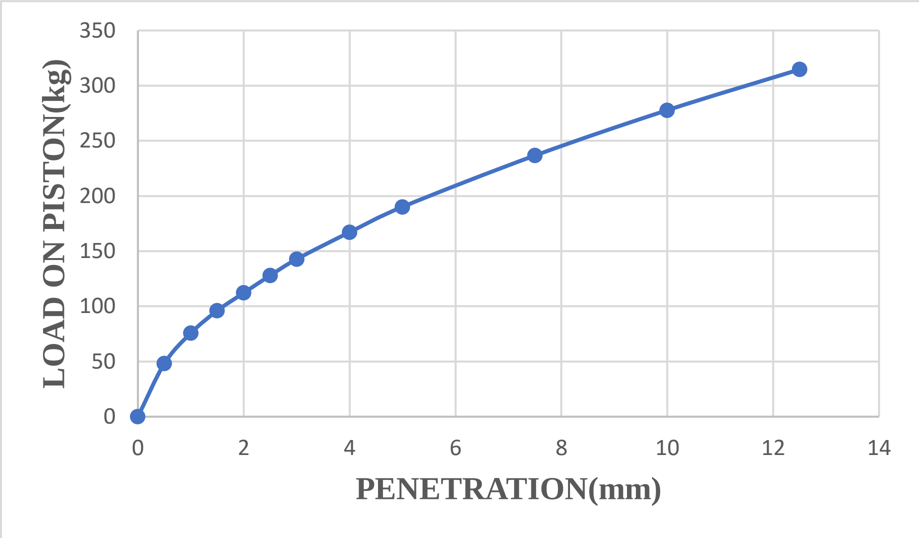

Fig 5.17 Load penetration graph for 1.5% addition of sugarcane bagasse

2.5=9.34%

5=9.26%

From the above **Fig 5.17**, it can be seen that the CBR for 2mm penetration found out to be the highest and it increases from the untreated soil but it decreases from the 1% addition of waste.

Table 5.14 Load penetration data for CBR test for 2% sugarcane bagasse

PENETRATION(mm)	LOAD ON PISTON (kg)
0	0
0.5	70.3
1	81.4
1.5	91.3
2	100.9
2.5	110.8
3	121.2
4	141.9
5	163
7.5	212.3
10	255.8
12.5	292.4

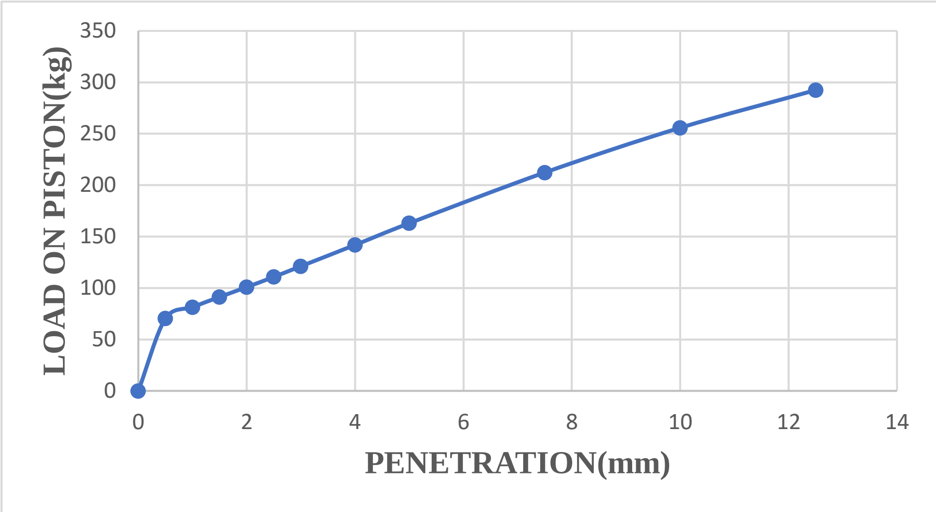

Fig 5.18 Load penetration graph for 2% addition of sugarcane bagasse

2.5=8.09

5=7.93

From **Fig 5.18** it can be seen that on 2% addition of sugarcane bagasse the CBR value was found out to be decreased compared to the 1.5% addition of waste.

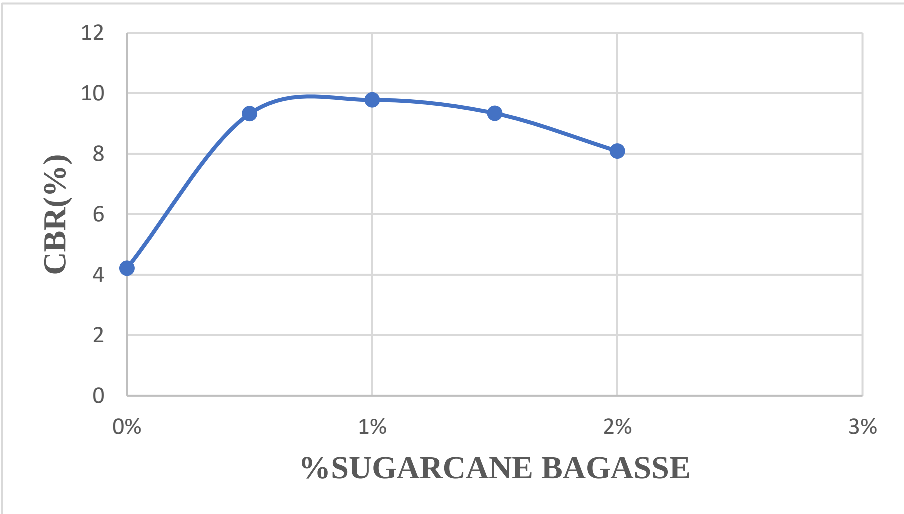

Fig 5.19 % sugarcane bagasse v/s CBR (%)

It can be seen from the above graph **Fig 5.19** that at 1% addition of sugarcane bagasse the CBR value was found to be the maximum and after that it decreases.

Table 5.15 Test values of soil mixed with sugarcane bagasse(waste)

%waste	Maximum dry density(g/cc)	Optimum moisture content (%)	UCS value (Dynamically compacted,KPa)	UCS value (Statically compacted, (KPa)	*CBR value
0	1.67	21.2	256.93	260.10	4.21%
2	2.153	16	259.41	262.23	9.32(at 0.5%waste)
4	1.210	15.3	253.77	259.60	9.78(at 1% waste)
6	1.660	17	234.04	242.27	9.34% (at 1.5%waste)
8	1.590	17.1	124.07	152.68	8.09% (at 2% waste)

*While doing the CBR test, the percentage of waste used are 0.5%,1%,1.5%&2% as beyond 2% it has been found out that the CBR values were very less.

5.3 Tests results of soil mixed with fly ash

Table 5.16 Liquid limit values at 2% fly ash

Cone penetration (mm)	Mass of empty container (g)	Mass of container with wet soil (g)	Mass of container with dry soil(g)	Mass of water (g)	Mass of dry soil(g)	Water content (%)
16	10.001	17.895	16.206	1.689	6.205	27.22
19	9.953	20.295	17.984	2.311	8.031	28.77
22	9.823	20.925	18.351	2.574	8.528	30.18
24	8.662	19.319	16.729	2.59	8.067	32.10
27	9.22	19.298	16.653	2.645	7.433	35.58

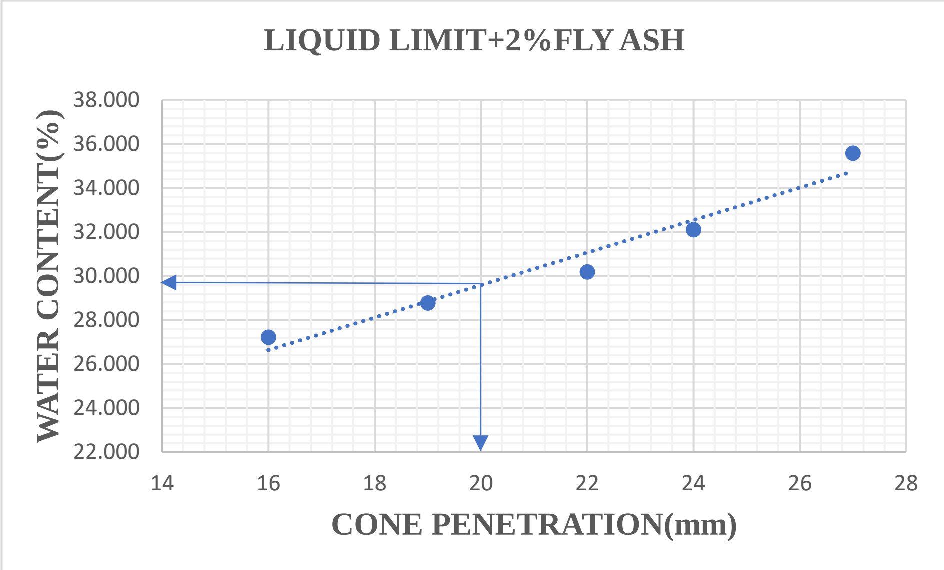

Fig 5.20 Liquid limit at 2% fly ash addition

LL = 29.8%

It can be seen from the above graph **Fig 5.20** that at 2% fly ash the liquid limit was found out to be 29.8% which means the liquid limit of the soil found to be decreased compared to the untreated soil which was found out to be 39.1%. This happens because of the non-plastic nature of the fly ash reduces the flowing capacity of the soil results in decreased in liquid limit.

Table 5.17 Plastic limit values at 2% fly ash

Mass of empty container (g)	Mass of container with wet soil(g)	Mass of container with dry soil(g)	Mass of water(g)	Mass of dry soil (g)	Water content (%)
9.912	10.559	10.442	0.117	0.53	22.07
8.544	9.466	9.298	0.168	0.754	22.28
9.834	11.366	11.109	0.257	1.275	20.16

PL = 21.50

From **Table 5.17**, At 2% addition of fly ash to the soil, the plastic limit of the soil found out to be decreased from the untreated soil. This happens because fly ash is of non-plastic nature on addition of it decreases the plasticity of the soil which as well decreases the thickness of the diffuse double layer and causes flocculation of clay particles result in changes of plasticity characteristics.

Table 5.18 Liquid limit values at 4% fly ash

Cone penetration (mm)	Mass of empty container (g)	Mass of container with wet soil(g)	Mass of container with dry soil(g)	Mass of water (g)	Mass of dry soil(g)	Water content (%)
19	9.928	24.619	21.392	3.227	11.464	28.15
20	9.953	18.861	16.844	2.017	6.891	29.27
23	9.823	18.592	16.478	2.114	6.655	31.77
24	8.662	16.704	14.731	1.973	6.069	32.51
27	9.22	20.322	17.435	2.887	8.215	35.14

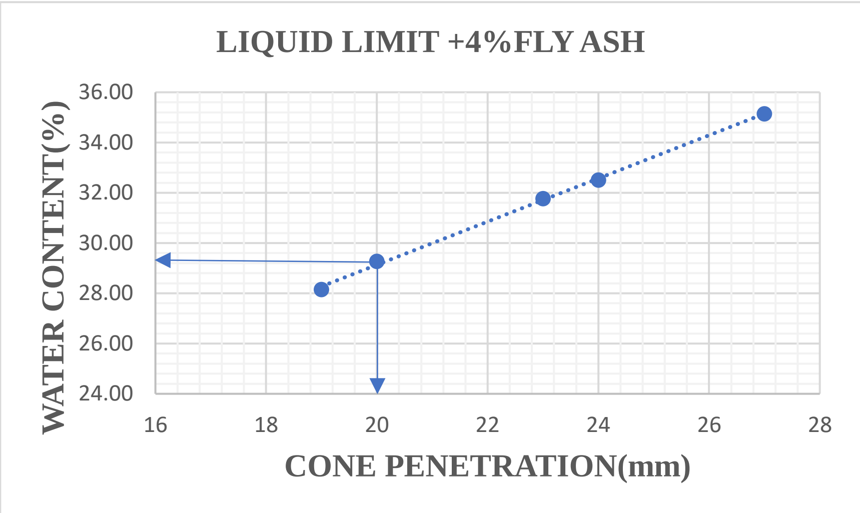

Fig 5.21 Liquid limit at 4% fly ash addition

LL = 28.8%

From **Fig 5.21**, on 4% waste addition, the liquid limit of the soil found out to be decreased compared to the untreated soil. The change in percentage was found out to be 10.3%. On increasing the amount of fly ash, the liquid limit of the soil found out to be decreased

Table 5.19 Plastic limit values at 4% fly ash

Mass of empty container (g)	Mass of container with wet soil	Mass of container with dry soil	Mass of water	Mass of dry soil	Water content (%)
8.142	10.724	10.293	0.431	2.151	20.04
9.504	12.761	12.218	0.543	2.714	20.007
9.644	11.9	11.518	0.382	1.874	20.38

PL = 20.6%

As it can be seen from **Table 5.19** that on 4% addition of fly ash, the plastic limit of the soil found out to be decreased. This happens because of the non-plastic nature of the fly ash which possess non-plastic nature to the soil results in decrease in plastic limit of the soil.

Table 5.20 Liquid limit values at 6% fly ash

Cone penetration (mm)	Mass of empty container (g)	Mass of container with wet soil (g)	Mass of container with dry soil(g)	Mass of water (g)	Mass of dry soil (g)	Water content (%)
17	9.928	16.693	15.37	1.323	5.442	24.31
20	9.953	20.039	17.847	2.192	7.894	27.77
25	9.823	16.63	15.083	1.547	5.26	29.41
26	8.662	17.402	15.358	2.044	6.696	30.52
28	9.22	16.929	15.087	1.842	5.867	31.39

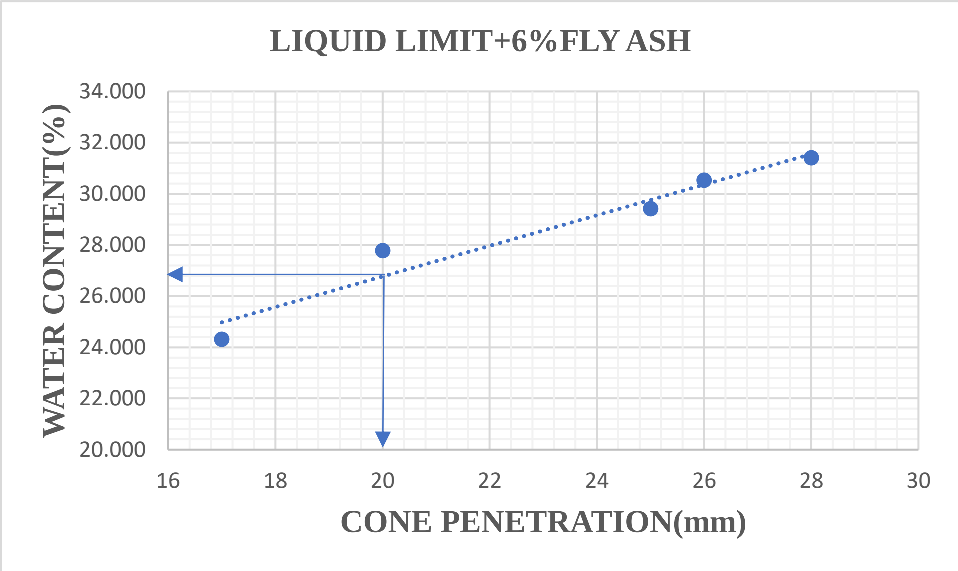

Fig 5.22 Liquid limit at 6% fly ash addition

LL = 26.2%

From **Fig 5.22**, on 6% addition of fly ash, the decrease in liquid limit found out to be 12.9% compared to the untreated soil. So, as we can see from the previous liquid limit graphs of fly ash percentages on increasing fly ash, the liquid limit of the soil found out to be decreased.

Table 5.21 Plastic limit values at 6% fly ash

Mass of empty container(g)	Mass of container with wet soil (g)	Mass of container with dry soil(g)	Mass of water (g)	Mass of dry soil(g)	Water content (%)
8.544	10.943	10.577	0.366	2.033	18.003
9.927	12.322	11.946	0.376	2.019	18.62
9.887	11.462	11.224	0.238	1.337	17.80

PL = 18.1%

From **table 5.21**, on increasing the percentages of fly ash, the plastic nature of the soil found out to be decreased compared to the untreated soil. On 6% addition of fly ash as well the plastic limit of the soil found out to be decreased.

Table 5.22 Liquid limit values at 8% fly ash

Cone penetration (mm)	Mass of empty container (g)	Mass of container with wet soil(g)	Mass of container with dry soil (g)	Mass of water (g)	Mass of dry soil (g)	Water content (%)
19	9.928	18.188	16.586	1.602	6.658	24.06
21	9.49	16.918	15.386	1.532	5.896	25.98
24	8.683	17.508	15.492	2.016	6.809	29.608
25	9.247	17.447	15.497	1.95	6.25	31.20
28	9.919	17.293	15.424	1.869	5.505	33.95

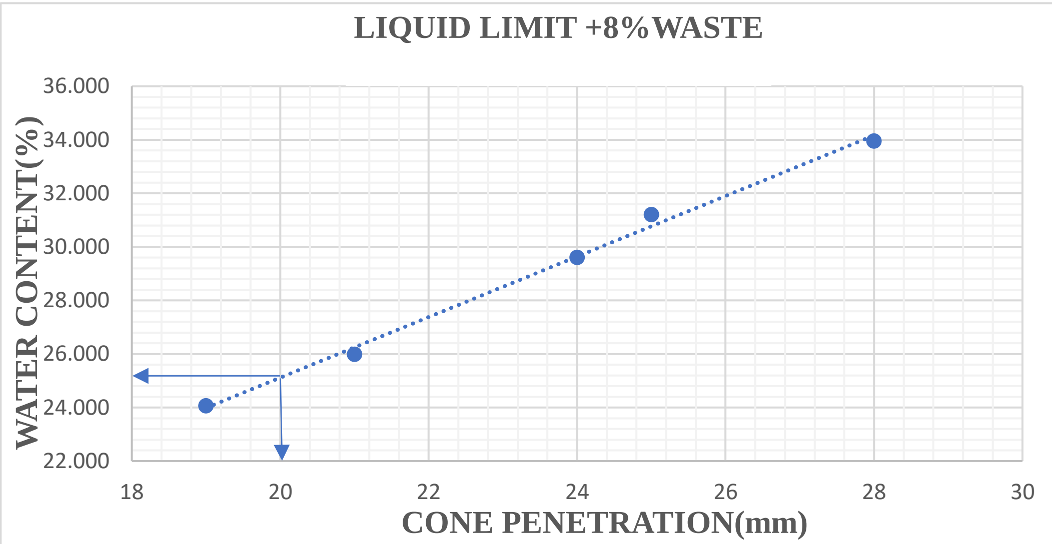

Fig 5.23 Liquid limit at 8% fly ash addition

LL = 25%

As it can be seen from the **Fig 5.23**, liquid limit of the on 8% addition of fly ash found out to be decreased the most in compared to the other percentages. The decreased in percentage is 14.1% which is the optimum decreased in percentage of fly ash addition.

Table 5.23 Plastic limit values at 8% fly ash

Mass of empty container(g)	Mass of container with wet soil	Mass of container with dry soil	Mass of water	Mass of dry soil	Water content (%)
9.826	11.381	11.163	0.218	1.337	16.305
8.921	10.192	9.995	0.197	1.074	18.34
9.459	11.546	11.23	0.316	1.771	17.84

PL = 17.4%

From **table 5.23**, on 8% addition of fly ash, the plastic limit of the soil found out to be decreased the most compared to the other percentages. So, the optimum decreased in plastic limit was found out to be on 8% waste addition.

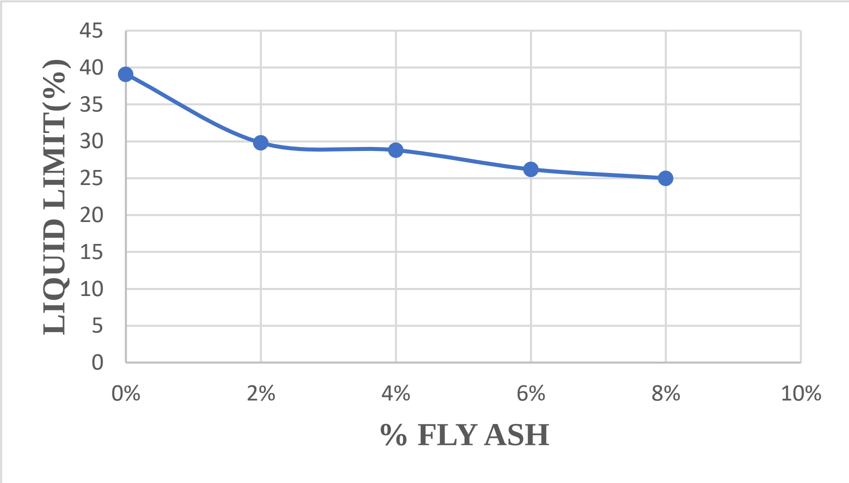

Fig 5.24 Liquid limit v/s % fly ash

The above graph **Fig 5.24** shows how the liquid limit of the decreased on increasing the fly ash percentages. This happened because of the non-plastic nature of the fly ash which results in decreased in water content of the soil. The maximum decrease in liquid limit was found out to be on 8% waste addition.

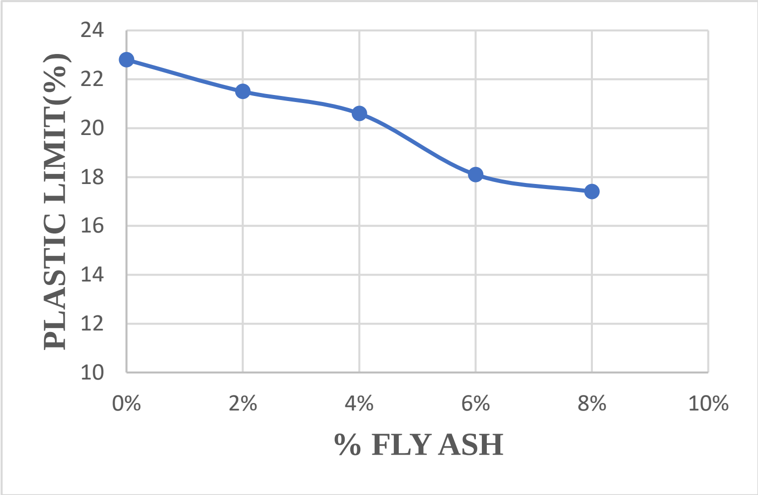

Fig 5.25 Plastic limit v/s % fly ash

From **Fig 5.25**, it can be seen that the plastic limit of the soil found out to be decreased on increasing the fly ash addition. Since fly ash is of non- plastic nature possessing no plasticity which decreases the plastic limit of the soil to utmost 17.4% compared to the untreated soil.

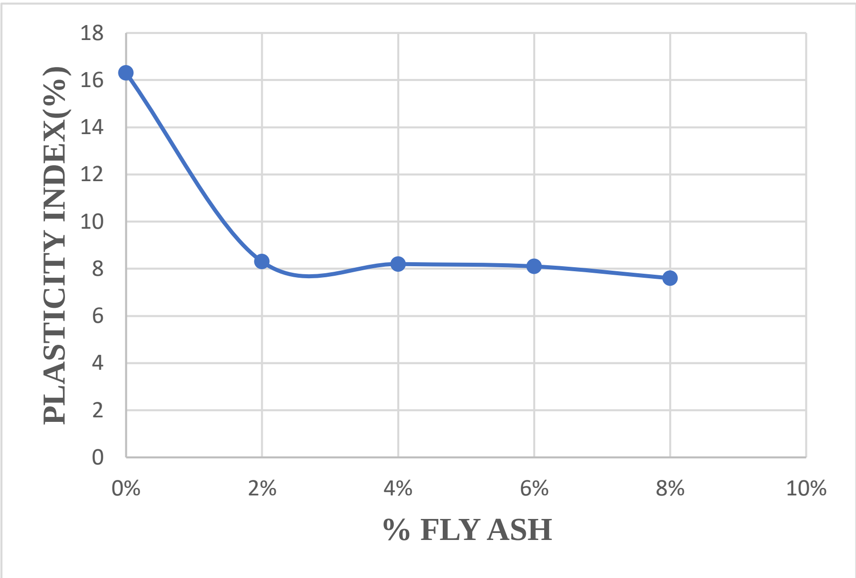

Fig 5.26 Plasticity index v/s % fly ash

As it can be seen from the above **Fig 5.26** plasticity index of the soil decreased significantly on addition of 2% fly ash and the decreased in percentage is about 8% compared to the untreated soil. Whereas, on 8% addition of fly ash the plasticity index is found out to be maximum decreased.

Table 5.24 Standard Proctor test results on 2% addition of fly ash

Empty mould+base plate = 3336g

Mass of compacted soil+mould with base plate(g)	Mass of empty container(g)	Mass of container with wet soil(g)	Mass of container with dry soil(g)	Mass of water (g)	Mass of dry soil (g)	Mass of compacted soil(g)	Bulk density (g/cc)	Water content (%)	Dry density (g/cc)
5180	9.896	22.85	20.924	1.926	11.028	1844	1.844	17.465	1.57
5268	9.854	25.237	22.782	2.455	12.928	1932	1.932	18.990	1.62
5290	8.768	25.891	23.08	2.811	14.312	1954	1.954	19.641	1.63
5286	9.251	24.181	21.622	2.559	12.371	1950	1.95	20.685	1.61

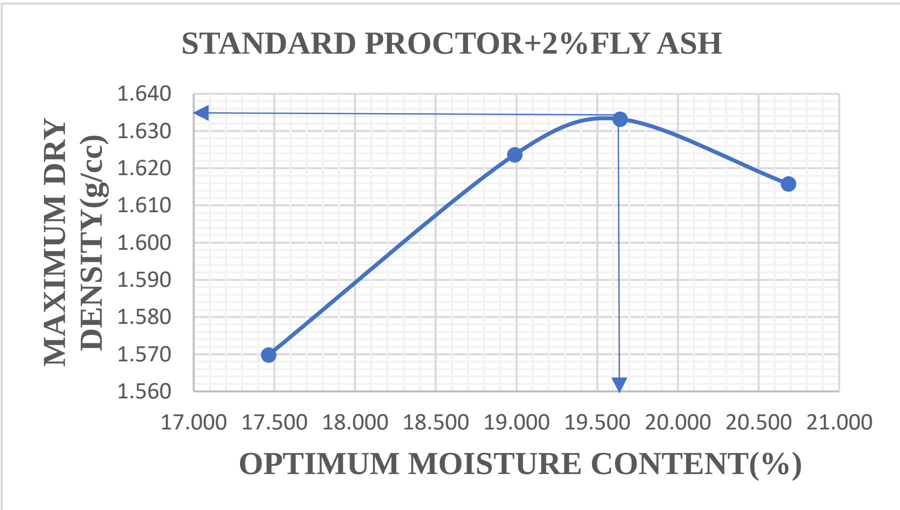

Fig 5.27 Compaction curve for 2% fly ash

From the above **Fig 5.27**, it can be seen that the OMC of the soil was found out to be 19.5% which decreases from the untreated soil and MDD of the soil was found out to be increased which is 1.63g/cc i.e., fly ash reduces the water content of the soil and makes it attain maximum density.

Table 5.25 Standard Proctor test results on 4% addition of fly ash

Mass of compacted soil+mould with base plate(g)	Mass of empty container(g)	Mass of container with wet soil(g)	Mass of container with dry soil(g)	Mass of water (g)	Mass of dry soil (g)	Mass of compacted soil (g)	Bulk density (g/cc)	Water content (%)	Dry density (g/cc)
4634	9.896	27.506	25.093	2.413	15.197	1298	1.298	15.87	1.12
4700	9.854	25.886	23.672	2.214	13.818	1364	1.364	16.02	1.17
5130	8.768	25.95	23.435	2.515	14.667	1794	1.794	17.14	1.53
5366	9.251	23.167	21.024	2.143	11.773	2030	2.03	18.20	1.71
5124	8.297	24.549	21.96	2.589	13.663	1788	1.788	18.94	1.50

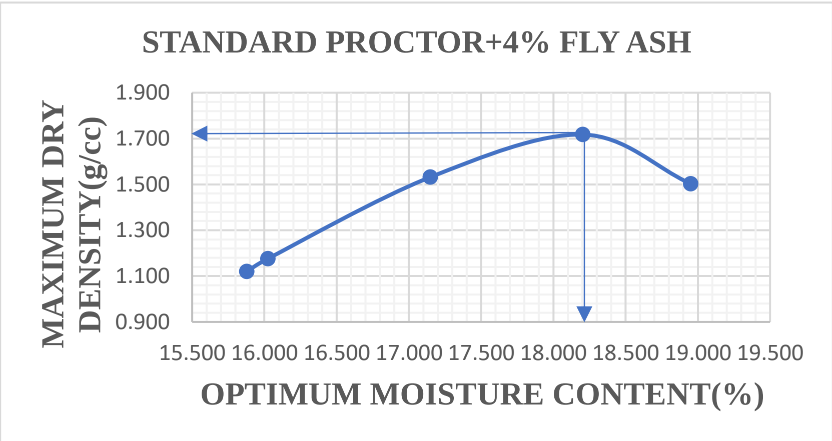

Fig 5.28 Compaction curve for 4% fly ash

From the above **Fig 5.28**, it can be seen that the OMC of the soil found out to be decreased to 18.2% whereas at the same time MDD of the soil found out to be increased to 1.7g/cc.

Table 5.26 Standard Proctor test results on 6% addition of fly ash

Mass of compacted soil+mould with base plate(g)	Mass of empty container(g)	Mass of container with wet soil(g)	Mass of container with dry soil(g)	Mass of water (g)	Mass of dry soil (g)	Mass of compacted soil(g)	Bulk density (g/cc)	Water content (%)	Dry density (g/cc)
5260	9.896	25.824	23.845	1.979	13.949	1924	1.924	14.18	1.68
5344	9.726	24.14	22.071	2.069	12.345	2008	2.008	16.76	1.72
5366	8.592	26.3	23.491	2.809	14.899	2030	2.03	18.85	1.70
5350	9.442	30.699	27.206	3.493	17.764	2014	2.014	19.66	1.68
5316	8.297	22.833	20.361	2.472	12.064	1980	1.98	20.49	1.64

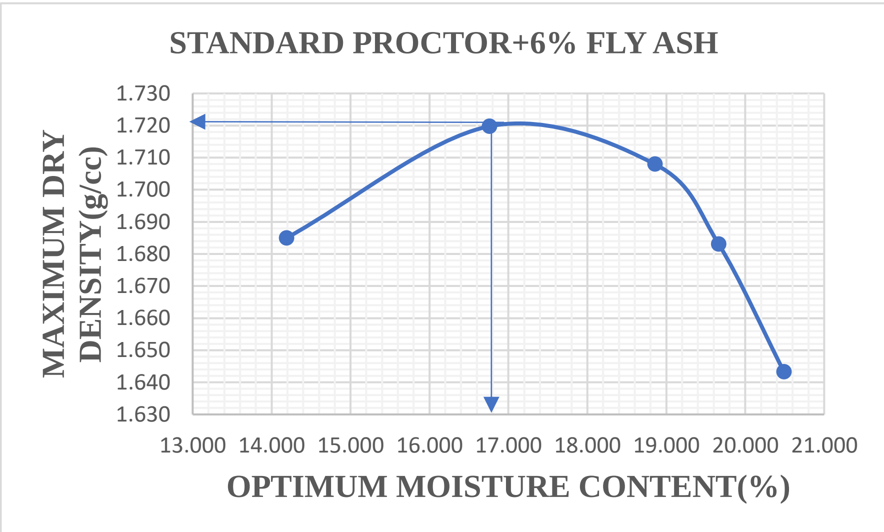

Fig 5.29 Compaction curve for 6% fly ash

As it can be seen from **Fig 5.29**, the OMC of the soil found out to be decreased to 17% whereas the MDD of the soil was found out to be increased compared to the untreated soil to 1.72g/cc which means that at increasing percentage of fly ash, soil has attained maximum strength compared to the untreated soil.

Table 5.27 Standard Proctor test results on 8% addition of fly ash

Mass of compacted soil+mould with base plate(g)	Mass of empty container(g)	Mass of container with wet soil(g)	Mass of container with dry soil(g)	Mass of water (g)	Mass of dry soil (g)	Mass of compacted soil (g)	Bulk density (g/cc)	Water content (%)	Dry density (g/cc)
4994	9.896	32.131	29.24	2.891	22.235	1658	1.658	13.00	1.46
5066	9.854	27.588	25.104	2.484	17.734	1730	1.73	14.00	1.52
5336	8.768	27.329	24.526	2.803	18.561	2000	2	15.10	1.74
5346	9.251	25.722	23.12	2.602	16.471	2010	2.01	15.79	1.73
5280	8.297	28.312	25.035	3.277	20.015	1944	1.944	16.37	1.67

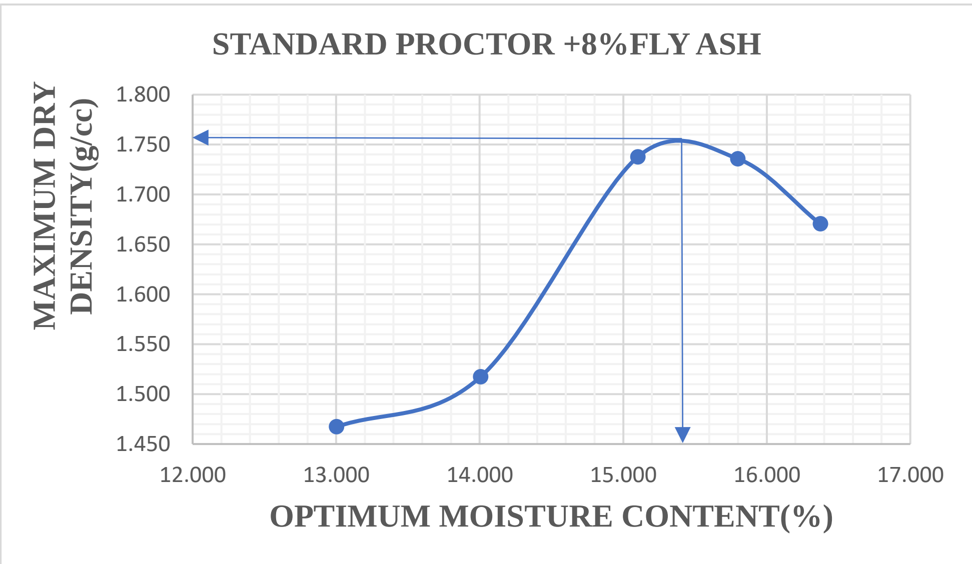

Fig 5.30 Compaction curve for 8% fly ash

From the above **Fig 5.30**, it can be seen that the OMC of the soil on 8% addition of fly found out to be decreased to 15.35% compared to the untreated soil whereas the MDD of the soil found out to be increased to 1.75g/cc. This means on reducing water content the soil has attained maximum strength on increasing percentage of fly ash.

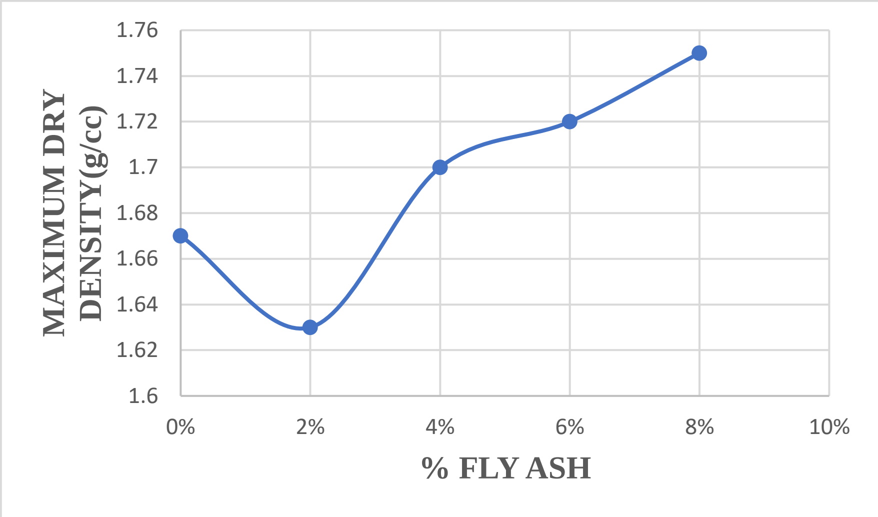

Fig 5.31 Maximum dry density v/s % fly ash

From the **Fig 5.31**, it can be seen that the MDD of the soil found to be increased except on 2% addition of fly ash. The maximum MDD found out to be on 8% fly ash addition. On 2% addition of fly ash, the MDD of the soil found out to be decreased.

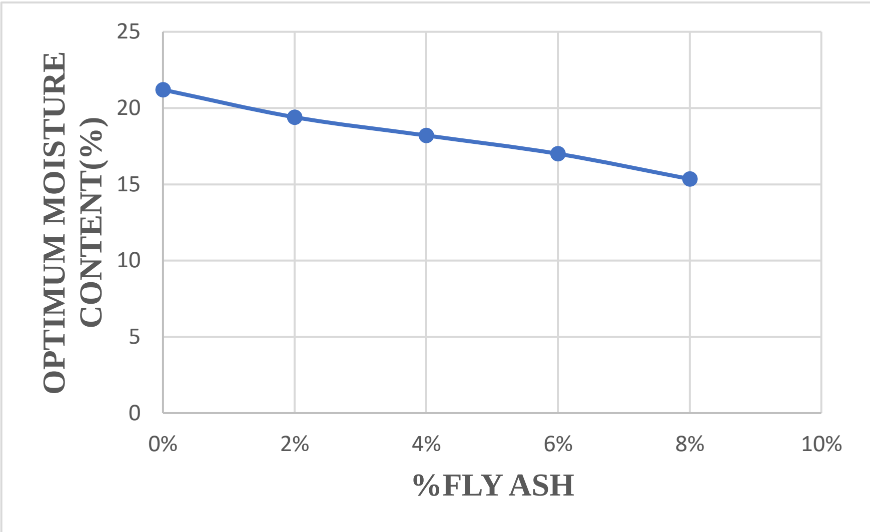

Fig 5.32 Optimum moisture content v/s % fly ash

From **Fig 5.32**, the OMC of the soil found out to be decreased on increasing percentage of fly ash which happens because of the non-plastic nature of the fly ash it decreases the water content on increasing fly ash addition.

Table 5.28 Unconfined Compressive Strength test for 2% fly ash (dynamic compaction)

Deformation dial reading	Axial deformation (cm)	Axial strain, ε	Area (cm^2)	Proving ring dial reading	Axial force (kg)	Compressive stress (KPa)
0	0	0	11.35	0	0	0
450	0.45	0.0059	11.40	22	7.26	63.00
500	0.5	0.0065	11.42	35	11.55	100.16
550	0.55	0.0072	11.42	45	14.85	128.69
600	0.6	0.0078	11.44	53	17.49	151.47
650	0.65	0.0085	11.44	61	20.13	174.21
700	0.7	0.0092	11.44	67	22.11	191.22
750	0.75	0.0098	11.45	73	24.09	208.21
800	0.8	0.0105	11.46	79	26.07	225.18
850	0.85	0.0111	11.46	84	27.72	239.27
900	0.9	0.0118	11.47	88	29.04	250.49
950	0.95	0.0125	11.48	95	31.35	270.24
1000	1	0.0131	11.49	99	32.67	281.43
1050	1.05	0.0138	11.49	103	33.99	292.61
1100	1.1	0.0144	11.51	106	34.98	300.93
1150	1.15	0.0151	11.52	110	36.3	312.08
1200	1.2	0.0157	11.52	113	37.29	320.38
1250	1.25	0.0164	11.54	116	38.28	328.66
1300	1.3	0.0171	11.53	119	39.27	336.93
1350	1.35	0.0177	11.54	116	38.28	328.22

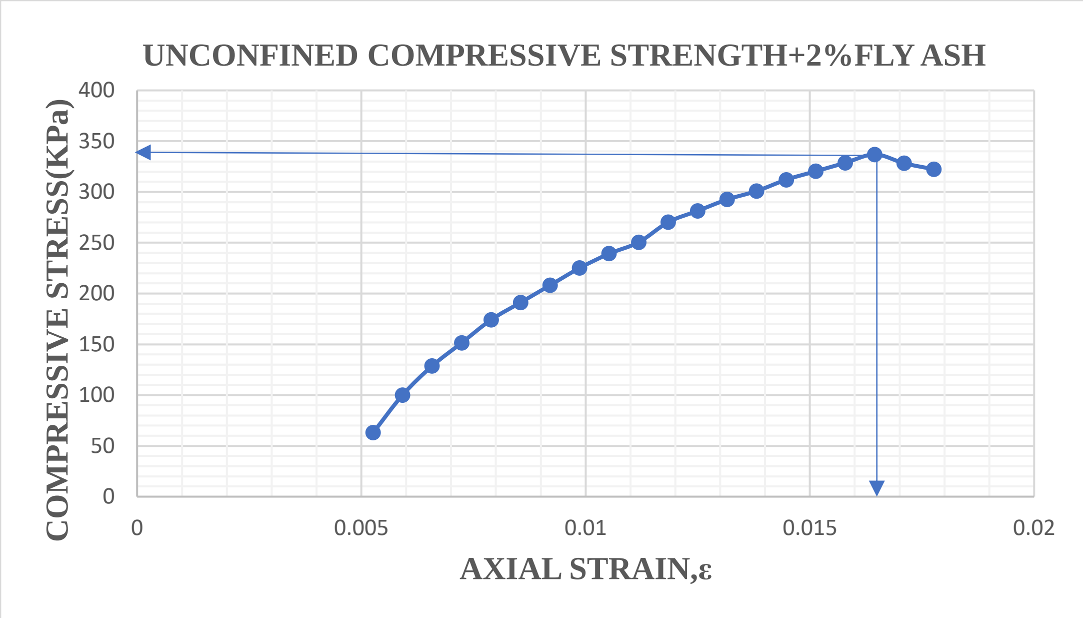

Fig 5.33 Unconfined Compressive Strength graph for 2% fly ash

From the **Fig 5.33** the compressive stress of the soil was found out to be 336.93Kpa which increases from the untreated soil which indicates that the quantity of fly ash induces pozzolanic reaction and cemented materials contributing to increase in strength of the soil.

Table 5.29 Unconfined Compressive Strength test for 4% fly ash (dynamic compaction)

Deformation dial reading	Axial deformation (cm)	Axial strain, ε	Area (cm^2)	Proving ring dial reading	Axial force (kg)	Compressive stress (KPa)
0	0	0	11.35	0	0	0
50	0.05	0.0006	11.34	18	5.94	51.81
100	0.1	0.0013	11.36	25	8.25	71.92
150	0.15	0.0019	11.37	35	11.55	100.63
200	0.2	0.0026	11.38	43	14.19	123.54
250	0.25	0.0032	11.38	53	17.49	152.17
300	0.3	0.0039	11.39	62	20.46	177.89
350	0.35	0.0046	11.40	70	23.1	200.72
400	0.4	0.0052	11.40	74	24.42	212.05
450	0.45	0.0059	11.40	77	25.41	220.50
500	0.5	0.0065	11.41	82	27.06	234.66
550	0.55	0.0072	11.42	84	27.72	240.23
600	0.6	0.0078	11.44	88	29.04	251.50
650	0.65	0.0085	11.44	93	30.69	265.61
700	0.7	0.0092	11.44	95	31.35	271.15
750	0.75	0.0098	11.45	100	33	285.23
800	0.8	0.0105	11.46	105	34.65	299.29
850	0.85	0.0111	11.46	109	35.97	310.48
900	0.9	0.0118	11.48	115	37.95	327.36
950	0.95	0.0125	11.48	120	39.6	341.36
1000	1	0.0131	11.49	122	40.26	346.82
1050	1.05	0.0138	11.49	118	38.94	335.22

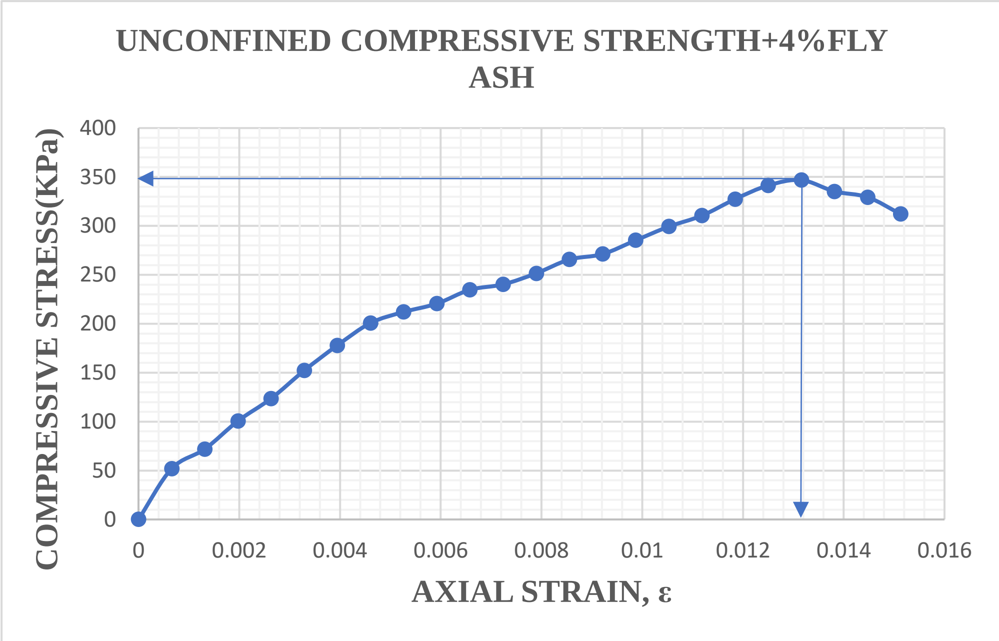

Fig 5.34 Unconfined Compressive Strength graph for 4% fly ash

As it can be seen from the above **Fig 5.34**, the compressive stress of the soil at 4% addition of fly ash was found out to be 346.82Kpa which itself increased from the 2% addition of fly ash as well as from the untreated soil.

Table 5.30 Unconfined Compressive Strength test for 6% fly ash (dynamic compaction)

Deformation dial reading	Axial deformation(cm)	Axial strain,ε	Area (cm^2)	Proving ring dial reading	Axial force (kg)	Compressive stress (KPa)
0	0	0	11.35	0	0	0
50	0.05	0.0006	11.34	10	3.3	28.80
100	0.1	0.0013	11.33	16	5.28	46.09
150	0.15	0.0019	11.33	22	7.26	63.38
200	0.2	0.0026	11.33	24	7.92	69.15
250	0.25	0.0032	11.34	27	8.91	77.80
300	0.3	0.0039	11.34	30	9.9	86.45
350	0.35	0.0046	11.33	33	10.89	95.10
400	0.4	0.0052	11.34	36	11.88	103.75
450	0.45	0.0059	11.33	39	12.87	112.40
500	0.5	0.0065	11.33	42	13.86	121.06
550	0.55	0.0072	11.33	44	14.52	126.83
600	0.6	0.0078	11.33	47	15.51	135.49
650	0.65	0.0085	11.33	52	17.16	149.90
700	0.7	0.0092	11.34	58	19.14	167.21
750	0.75	0.0098	11.34	68	22.44	196.05
800	0.8	0.0105	11.33	74	24.42	213.36
850	0.85	0.0111	11.32	78	25.74	224.91
900	0.9	0.0118	11.33	82	27.06	236.46
950	0.95	0.0125	11.32	88	29.04	253.78
1000	1	0.0131	11.33	80	26.4	230.73
1050	1.05	0.0138	11.33	74	24.42	213.43
1100	1.1	0.0144	11.32	70	23.1	201.90

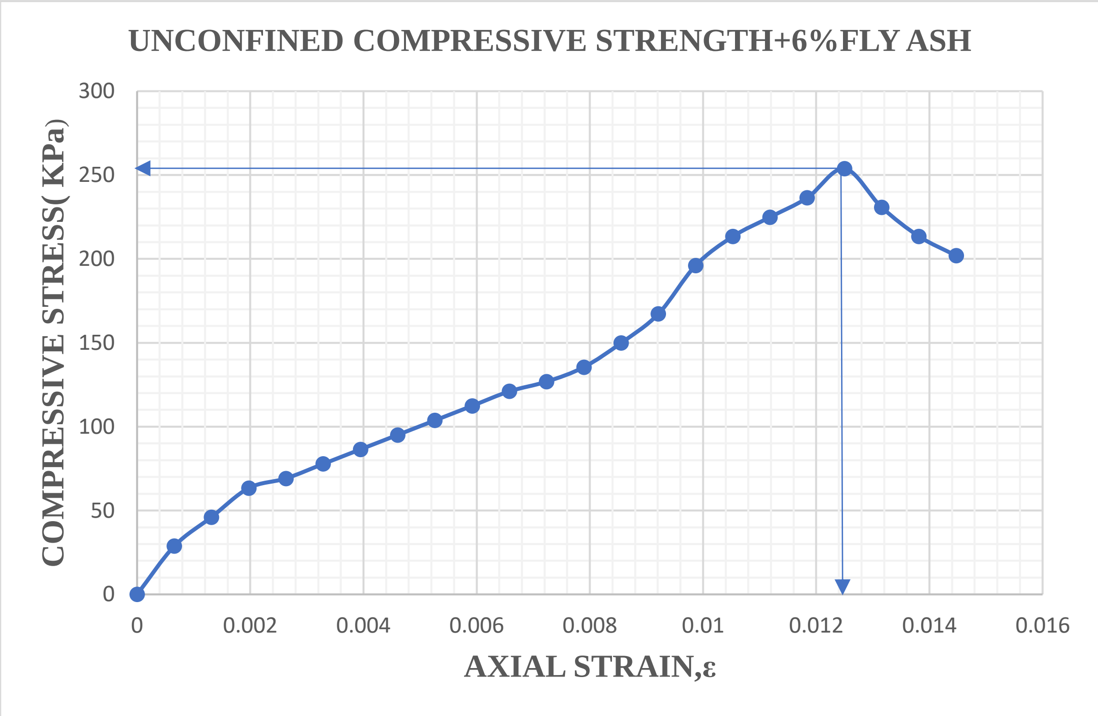

Fig 5.35 Unconfined Compressive Strength graph for 6% fly ash

As it can be seen from the **Fig 5.35**, the compressive stress for 6% waste addition was found to be decreased to 253.78KPa from 4% waste addition.

Table 5.31 Unconfined Compressive Strength test for 8% fly ash (dynamic compaction)

Deformation dial reading	Axial deformation (cm)	Axial strain,ε	Area (cm^2)	Proving ring dial reading	Axial force (kg)	Compressive stress (KPa)
0	0	0	11.35	0	0	0
50	0.05	0.0006	11.34	4	1.32	11.51521262
100	0.1	0.0013	11.36	10	3.3	28.76907959
150	0.15	0.0019	11.37	16	5.28	46.0002042
200	0.2	0.0026	11.37	21	6.93	60.33546889
250	0.25	0.0038	11.37	24	7.92	68.90933687
300	0.3	0.0039	11.38	30	9.9	86.0798152
350	0.35	0.0046	11.39	35	11.55	100.3601192
400	0.4	0.0052	11.40	37	12.21	106.0248609
450	0.45	0.0059	11.40	40	13.2	114.5456634
500	0.5	0.0065	11.41	46	15.18	131.6403339
550	0.55	0.0072	11.43	48	15.84	137.2728572
600	0.6	0.0078	11.44	52	17.16	148.6137118
650	0.65	0.0085	11.43	58	19.14	165.6515264
700	0.7	0.0092	11.44	64	21.12	182.6665986
750	0.75	0.0098	11.46	69	22.77	196.8066581
800	0.8	0.0105	11.47	74	24.42	210.9277656
850	0.85	0.0111	11.48	78	25.74	222.1814411
900	0.9	0.0118	11.47	82	27.06	233.4199551
950	0.95	0.0125	11.48	85	28.05	241.7986178
1000	1	0.0131	11.49	81	26.73	230.2663485
1050	1.05	0.0138	11.50	78	25.74	221.5901399

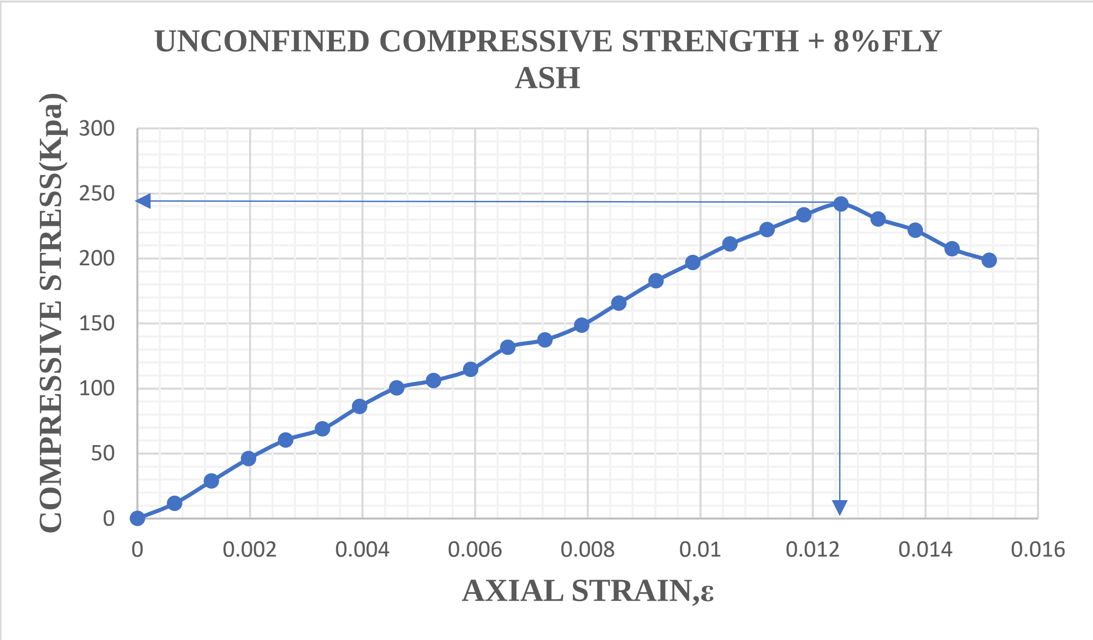

Fig 5.36 Unconfined Compressive Strength graph for 8% fly ash

As it can be seen from the above **Fig 5.36**, the compressive stress of the soil for 8% addition of fly ash was found out to be 241.79Kpa which decreases from the 6% addition of fly.

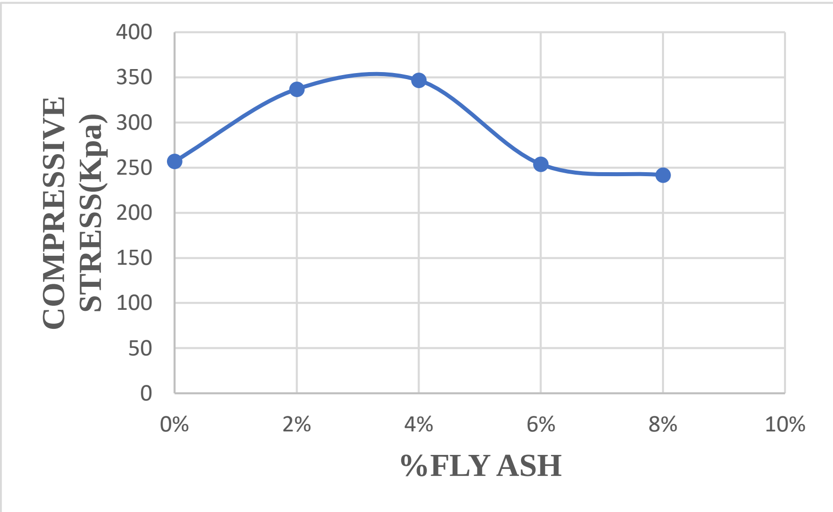

Fig 5.37 Compressive stress v/s % fly ash

From **Fig 5.37**, it can be seen that the compressive stress was found out to be maximum on 4% addition of fly ash whereas after that the compressive stress decreases and becomes the least on 8% addition of waste. So, optimum of 4% addition of fly ash is necessary for attaining maximum strength.

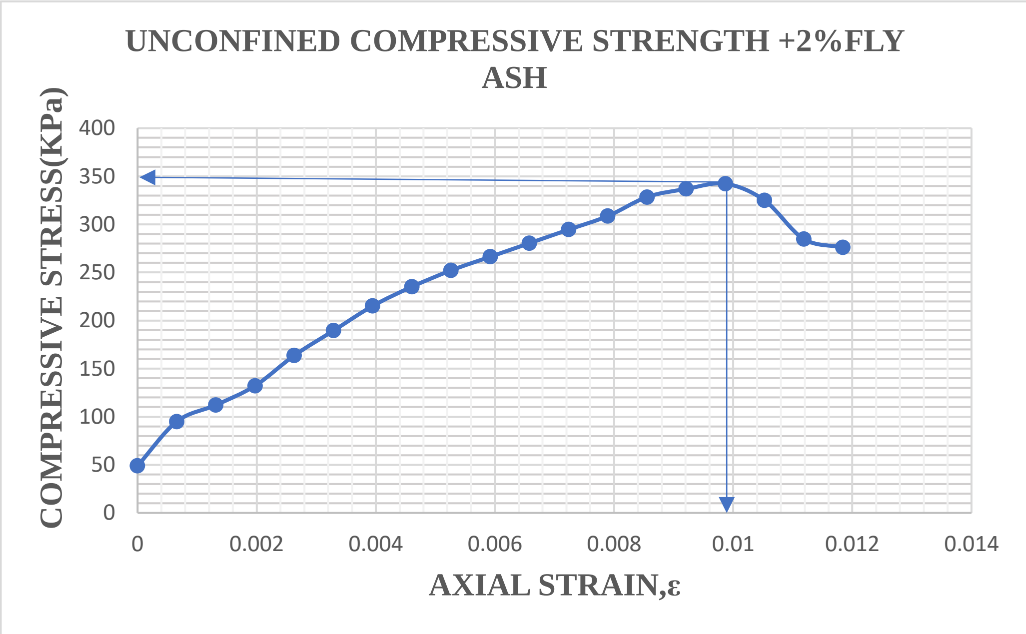

Fig 5.38 Unconfined Compressive Strength graph for 2% fly ash (static compaction)

As it can be seen from the above graph **Fig 5.38** that the compressive stress of the soil was found out to be 342.27KPa.

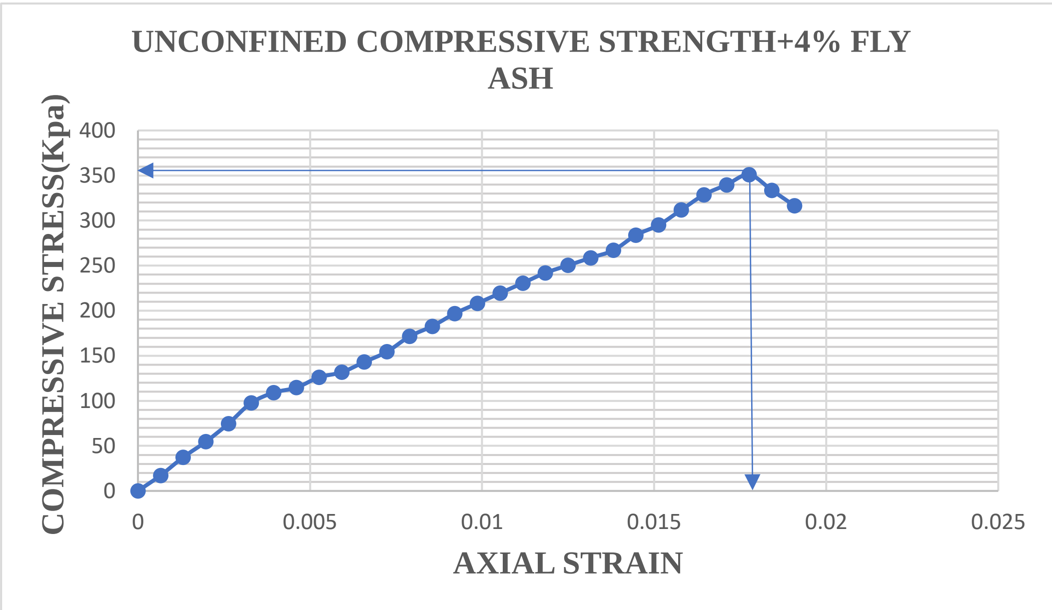

Fig 5.39 Unconfined Compressive Strength graph for 4% fly ash (static compaction)

As it can be seen from the above **Fig 5.39**, the compressive stress of the soil was found out to be increase to 350.86KPa from the 2% addition of fly ash.

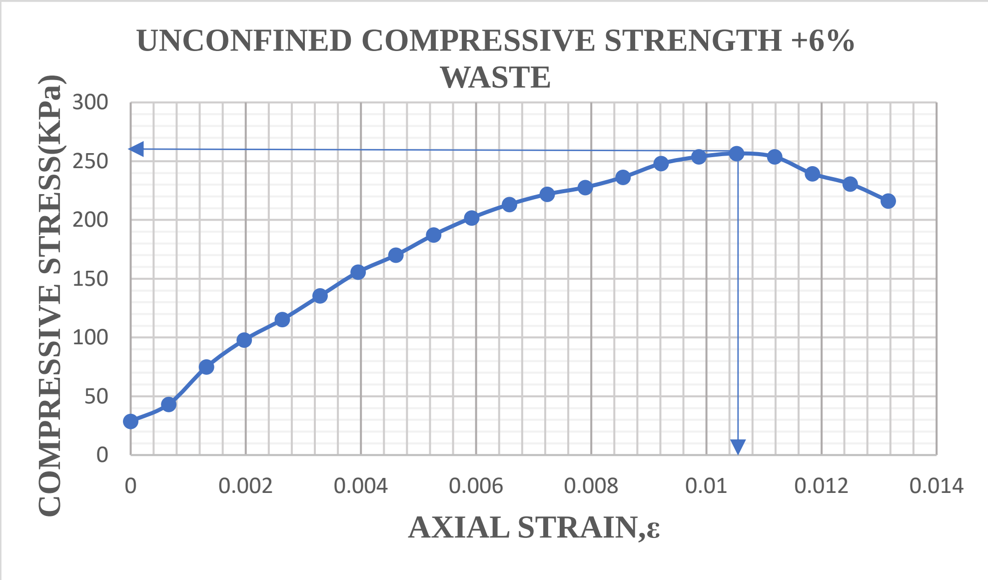

Fig 5.40 Unconfined Compressive Strength graph for 6% fly ash (static compaction)

From the above **Fig 5.40**, the compressive stress was found out to be 256.62KPa which decreases from the 4% addition of fly ash.

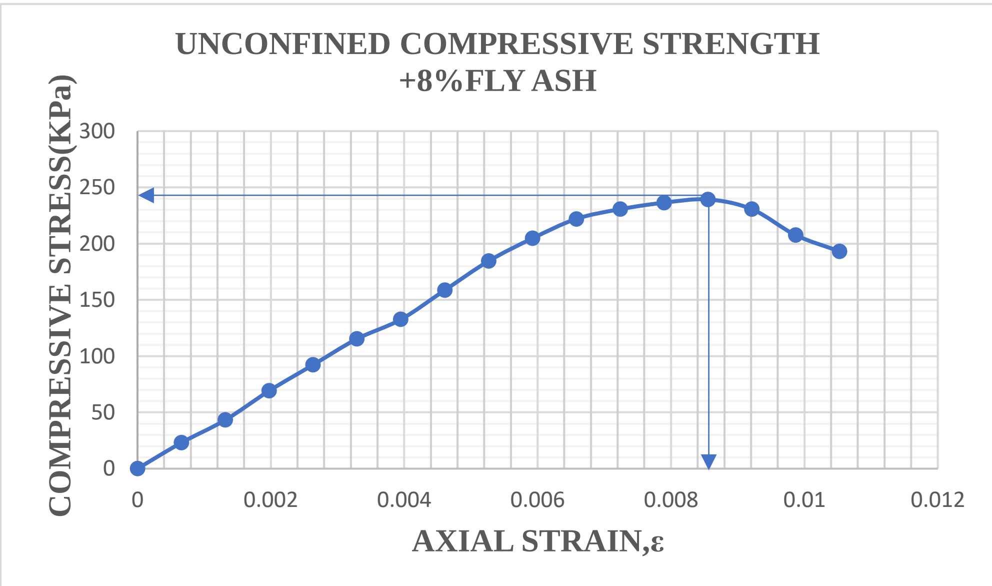

Fig 5.41 Unconfined Compressive Strength graph for 8% fly ash (static compaction)

From the above **Fig 5.41**, the compressive stress was found out to be 239.27KPa which decreases from the 6% addition of fly ash and it is found out to be the least compressive stress amongst the other waste addition.

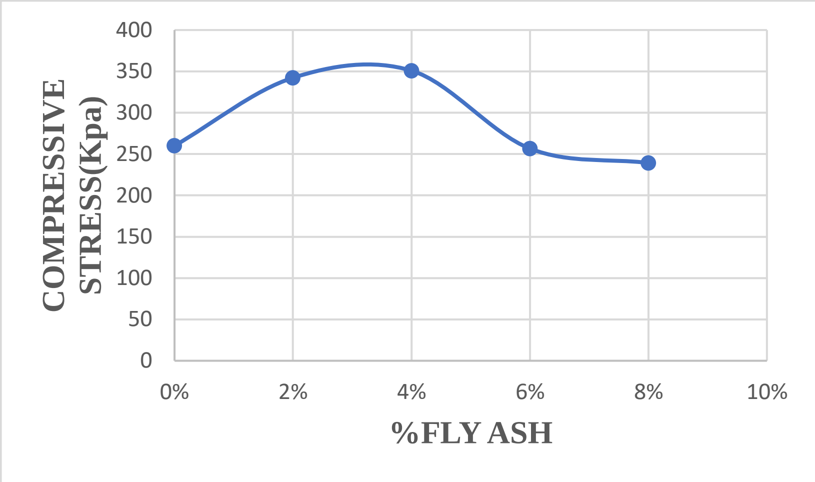

Fig 5.42 Compressive stress v/s % fly ash

From the above **Fig 5.42**, it can be seen that the maximum compressive stress was found out to be at 4% waste addition as it was same scenario like the dynamic compaction but after that the compressive stress found out to be decreased on 6% and 8% waste addition. The maximum decrease in strength was found out to on 8% waste addition.

Table 5.32 Load penetration data for CBR test for 0.5% fly ash

PENETRATION(mm)	LOAD ON PISTON (kg)
0	0
0.5	37.3
1	49.8
1.5	59.5
2	69
2.5	78.1
3	87.4
4	102.6
5	115.4
7.5	141.2
10	160.6
12.5	185.6

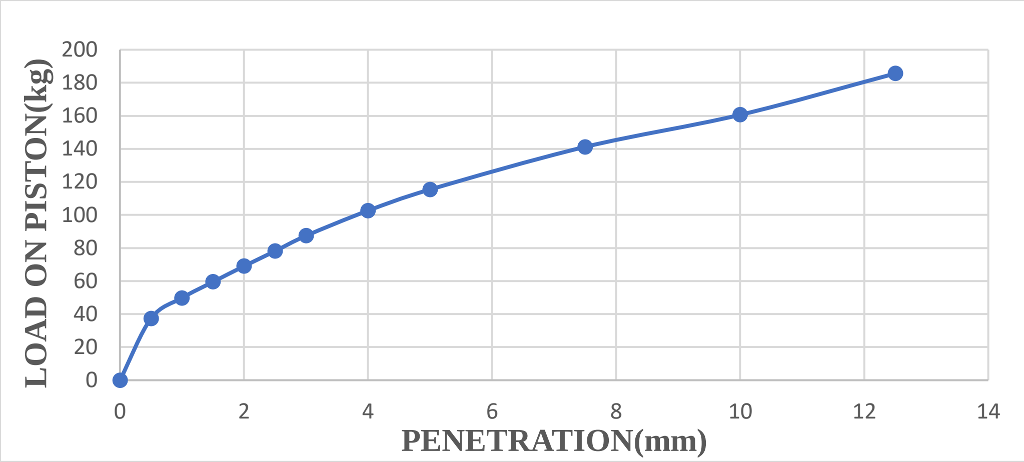

Fig 5.43 Load penetration graph for 0.5% addition of fly ash

2.5= 5.70%

5=5.62%

From the above **Fig 5.43**, it can be seen that CBR value at 2.5mm penetration was found out to be more i.e., 5.70%.

Table 5.33 Load penetration data for CBR test for 1% fly ash

PENETRATION(mm)	LOAD ON PISTON (kg)
0	0
0.5	35.2
1	50.3
1.5	63.7
2	76.4
2.5	88.7
3	99.7
4	121.4
5	141.4
7.5	183
10	215.7
12.5	255.3

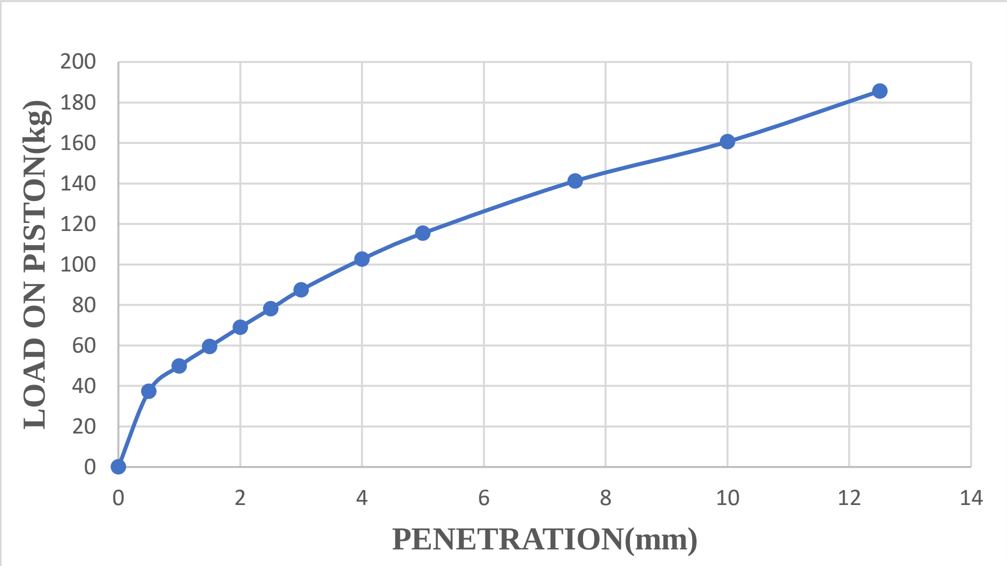

Fig 5.44 Load penetration graph for 1% addition of fly ash

2.5=6.47%
5=6.88

From the above **Fig 5.44**, it can be seen that CBR value at 5mm penetration was found out to be more i.e., 6.88%.

Table 5.34 Load penetration data for CBR test for 1.5% fly ash

PENETRATION(mm)	LOAD ON PISTON (kg)
0	0
0.5	69.5
1	95.7
1.5	120.9
2	140.6
2.5	160.7
3	178.3
4	195.4
5	210.84
7.5	243.3
10	277.1
12.5	305.3

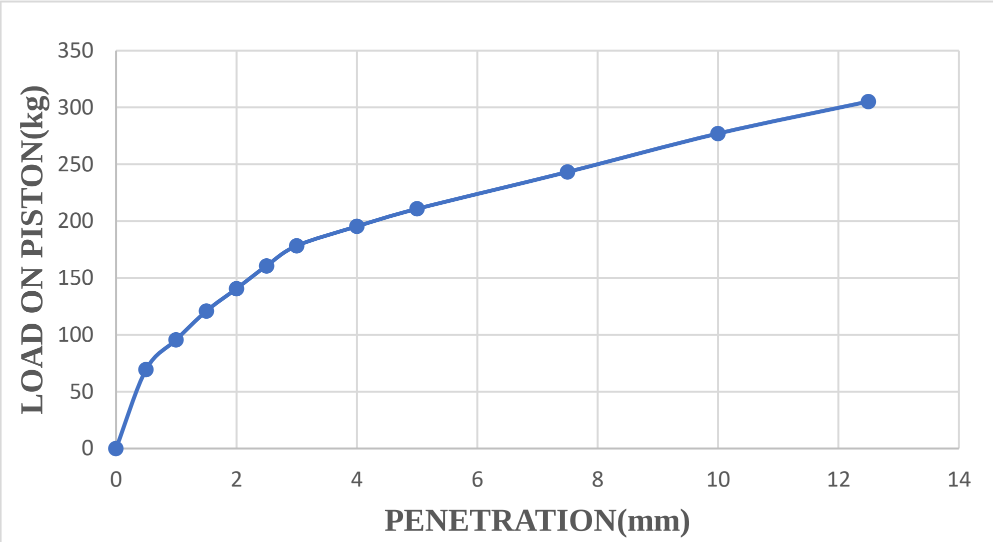

Fig 5.45 Load penetration graph for 1.5% addition of fly ash

2.5=13.48%
5=10.26%

From the above **Fig 5.45**, it can be seen that CBR value at 2.5mm penetration was found out to be more i.e., 13.48%.

Table 5.35 Load penetration data for CBR test for 2% fly ash

PENETRATION(mm)	LOAD ON PISTON (kg)
0	0
0.5	6.9
1	12.7
1.5	18.2
2	25.3
2.5	30.6
3	34.3
4	38.8
5	41.5
7.5	43.9
10	47.8
12.5	52.3

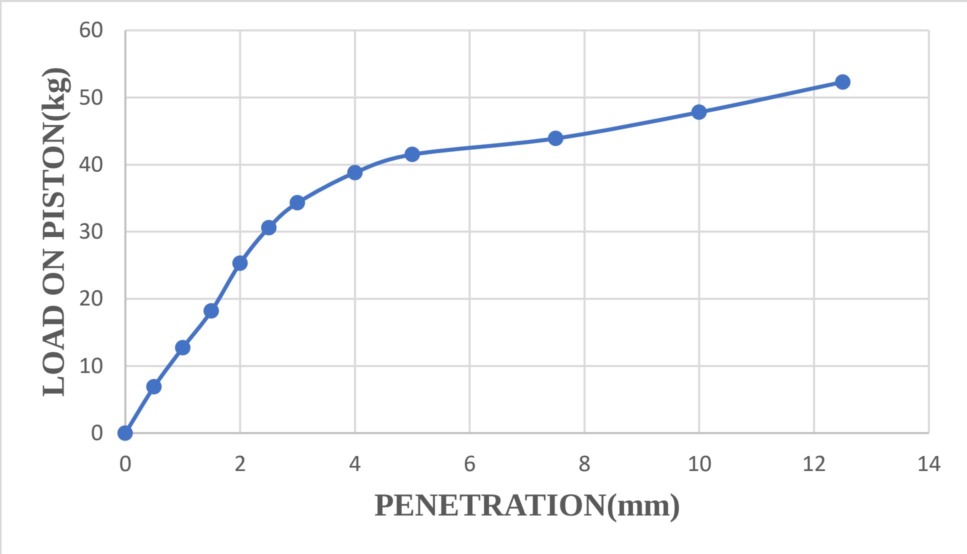

Fig 5.46 Load penetration graph for 2% addition of fly ash

2.5=2.24%
5=2.02%

From the above **Fig 5.46**, it can be seen that CBR value at 2.5mm penetration was found out to be 2.24% which is the least value compared to other percentage.

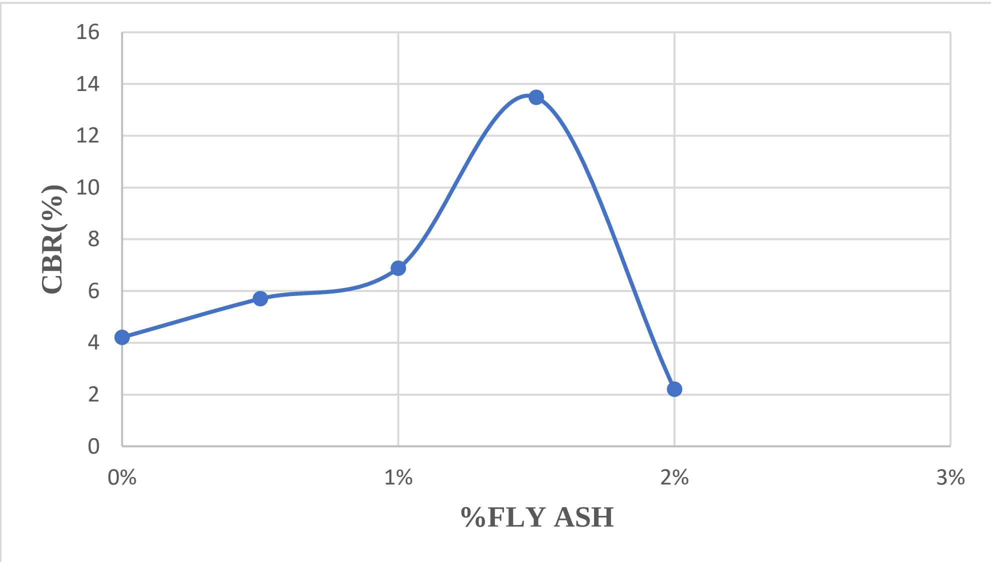

Fig 5.47 % fly ash v/s CBR value

It can be seen from the above **Fig 5.47**, that CBR value at 1.5% found out to be the most compared to the other values and the least value found out on be on 2%waste addition.

Table 5.36 Test values of fly ash addition to soil

%waste	Liquid limit	Plastic limit	Plasticity index	OMC (%)	MDD (g/cc)	UCS (Static) (Kpa)	UCS (Dynamic) (Kpa)	CBR value (%)
0%	39.1	22.8	16.3	21.2	1.67	260.10	256.93	*4.21
2%	29.8	21.5	8.3	19.4	1.63	342.27	336.93	5.70
4%	28.8	20.6	8.2	18.2	1.70	350.86	346.82	6.88
6%	26.2	18.1	8.1	17	1.72	256.62	253.78	13.48
8%	25	17.4	7.6	15.35	1.75	239.27	241.79	2.24

*CBR values are found out at waste percentages of 0.5%,1%,1.5% and 2%addition of fly ash.

5.4 Combine graphs of sugarcane bagasse and fly ash

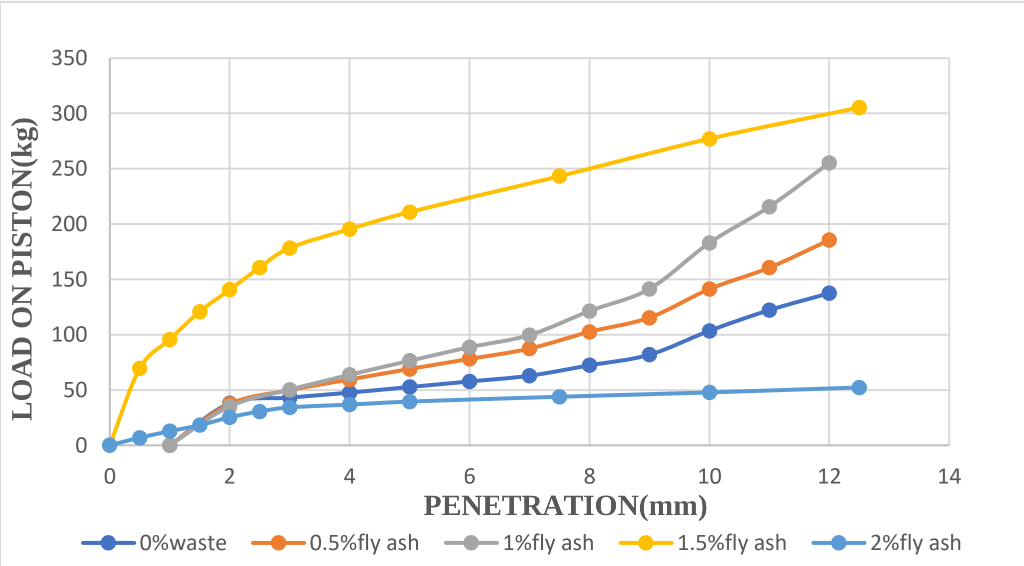

Fig 5.48 CBR values for various percentage of fly ash addition

From the **Fig 5.48**, the maximum value of CBR was found to be on 1.5% addition of fly ash.

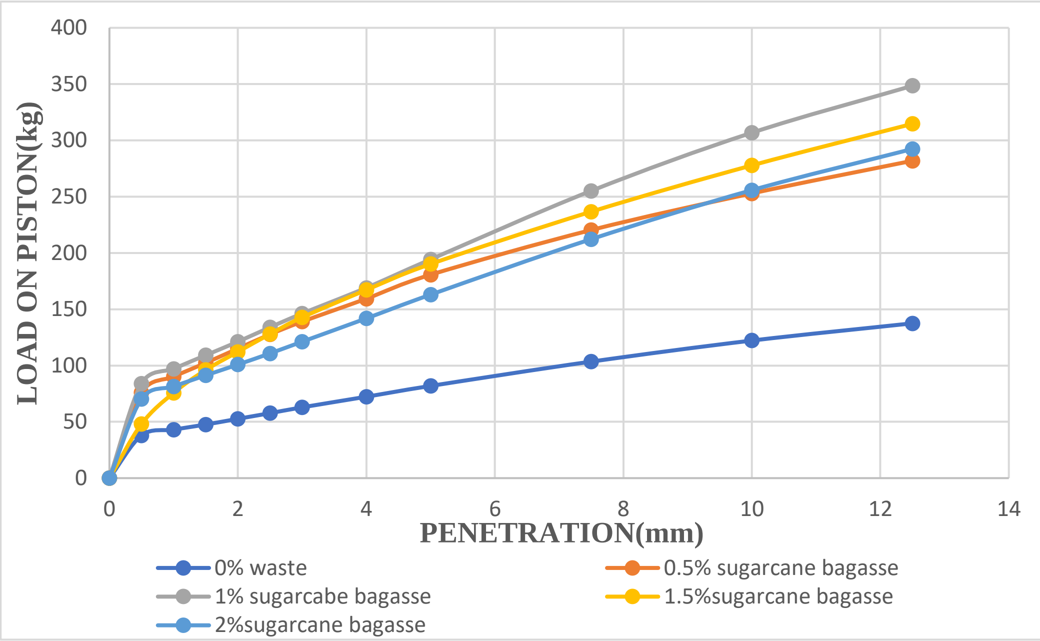

Fig 5.49 CBR values for various percentage of sugarcane bagasse addition

From **Fig 5.49**, the maximum value of CBR was found to be on 1% addition of SB after that it decreases.

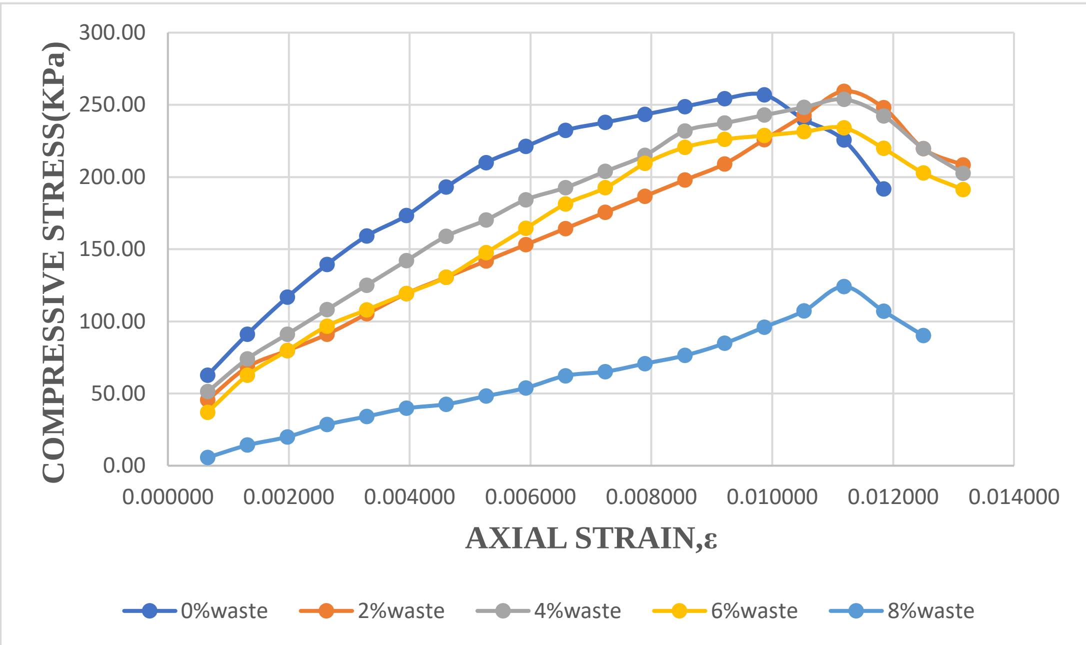

Fig 5.50 Graph for UCS test for various percentage of sugarcane bagasse (dynamically compacted)

From **Fig 5.50**, the highest UCS value was found out to be on 2% addition of SB by dynamic compaction and after that the value decreases.

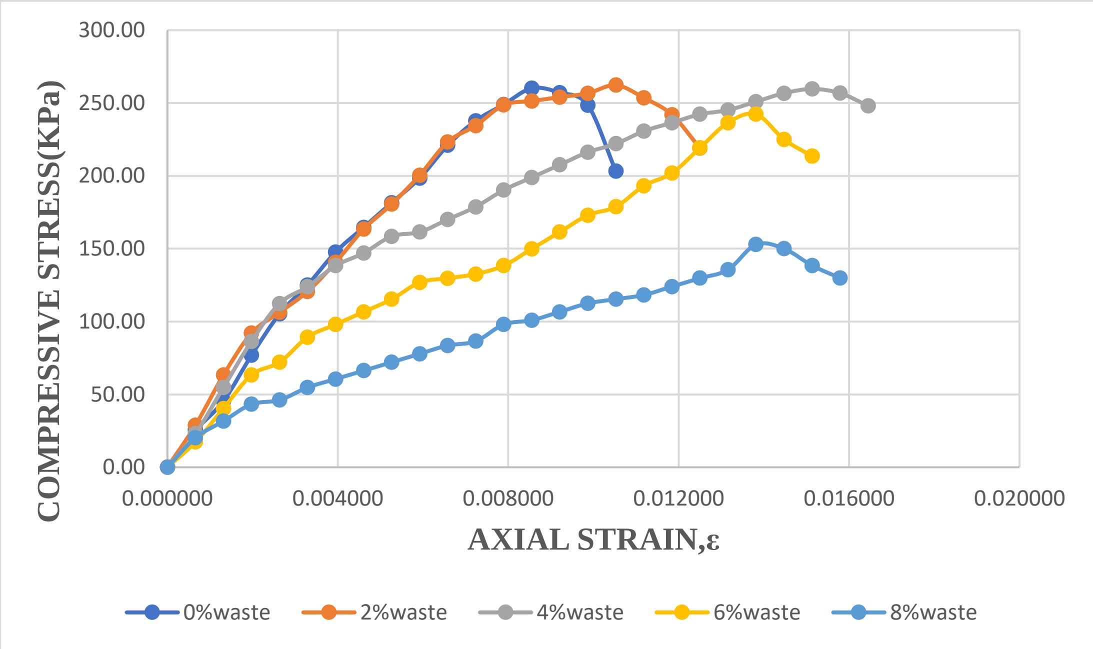

Fig 5.51 Graph for UCS test for various percentage of sugarcane bagasse (statically compacted)

From **Fig 5.51**, the maximum value of UCS was found to be on 2% addition of SB by static compaction. The static compaction values show higher UCS than dynamic compaction.

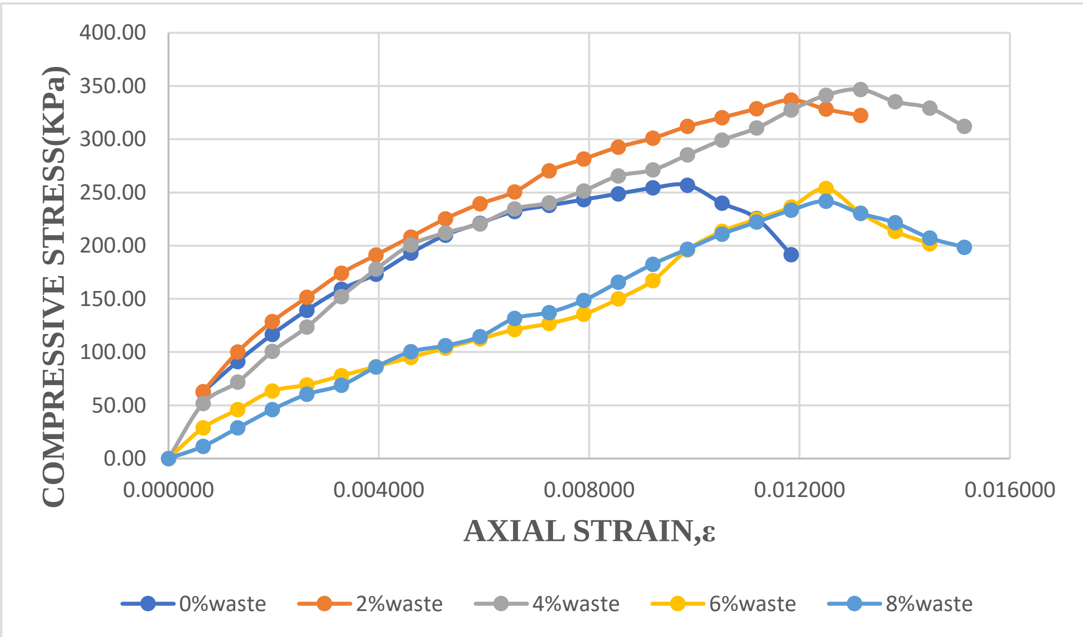

Fig 5.52 Graph for UCS test for various percentage of fly ash (dynamically compacted)

From **Fig 5.52**, the maximum value of UCS was found to be on 4% addition of fly ash after that the UCS value decreases.

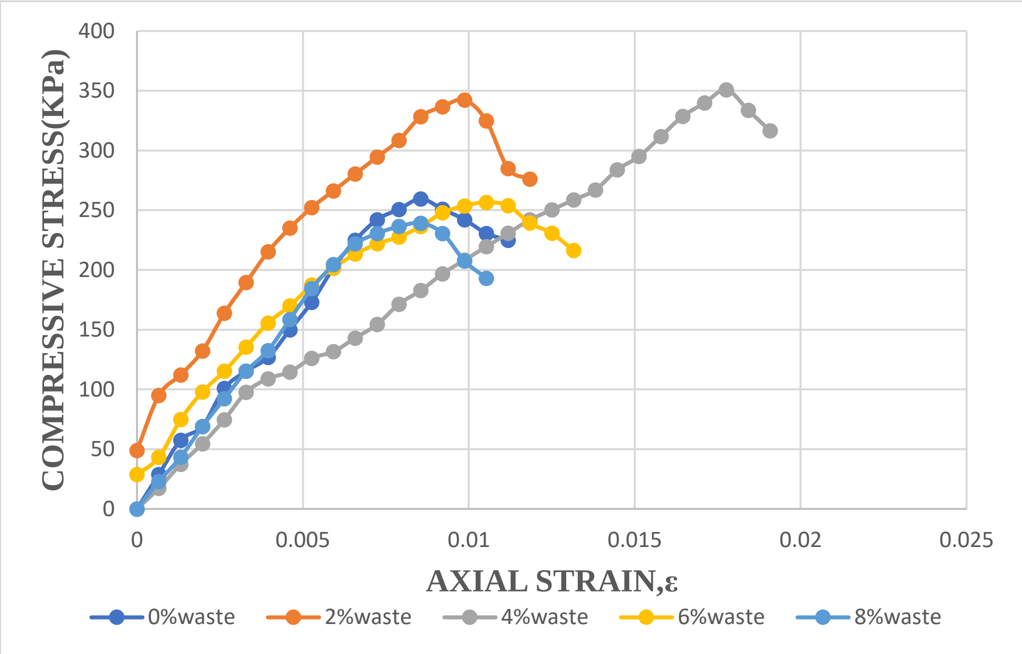

Fig 5.53 Graph for UCS test for various percentage of fly ash (statically compacted)

From **Fig 5.53**, the maximum value of UCS was found to be on 4% addition of fly ash and the static values are higher compared to dynamic compaction values.

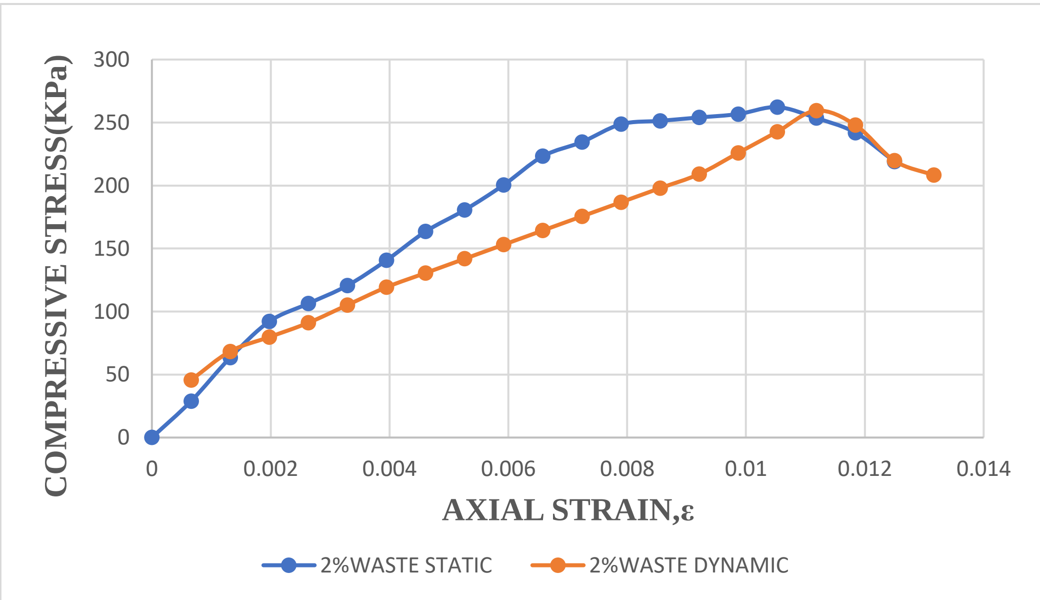

Fig 5.54 Superimposed graph for UCS test for 2% of sugarcane bagasse (statically and dynamically compacted)

From **Fig 5.54**, the static compaction values of UCS shows higher results than dynamic compaction of UCS.

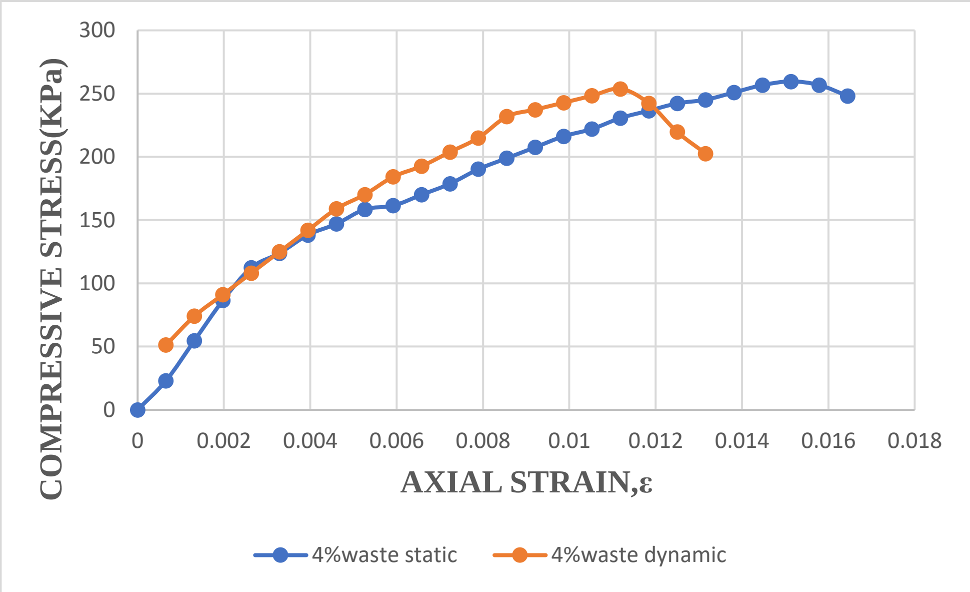

Fig 5.55 Superimposed graph for UCS test for 4% of sugarcane bagasse (statically and dynamically compacted)

From **Fig 5.55**, the static compaction values of UCS shows higher results than dynamic compaction of UCS.

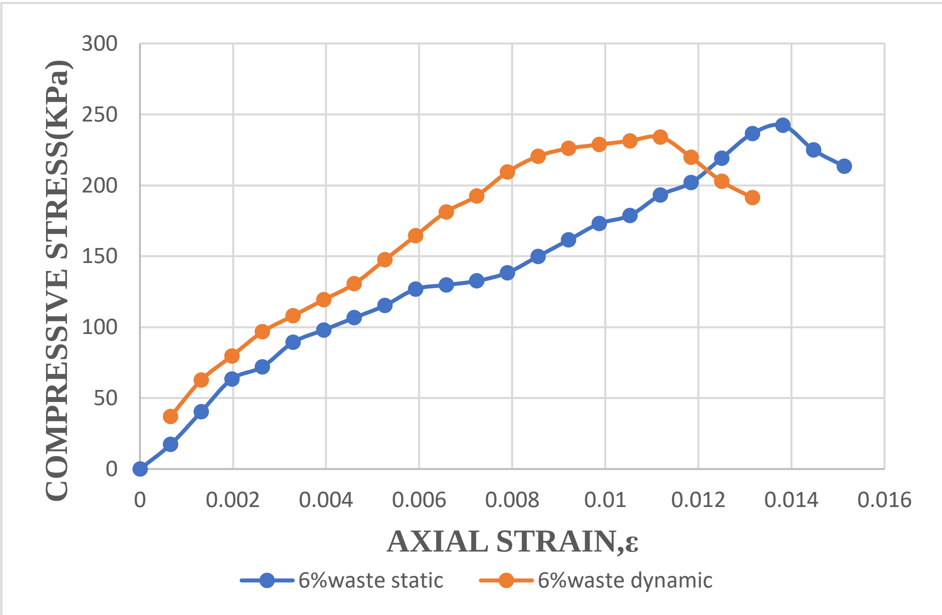

Fig 5.56 Superimposed graph for UCS test for 6% of sugarcane bagasse (statically and dynamically compacted)

From **Fig 5.56**, the maximum value of UCS was found out to be 242.27KPa by static compaction method whereas dynamic compaction value of UCS was found to be 234.04KPa.

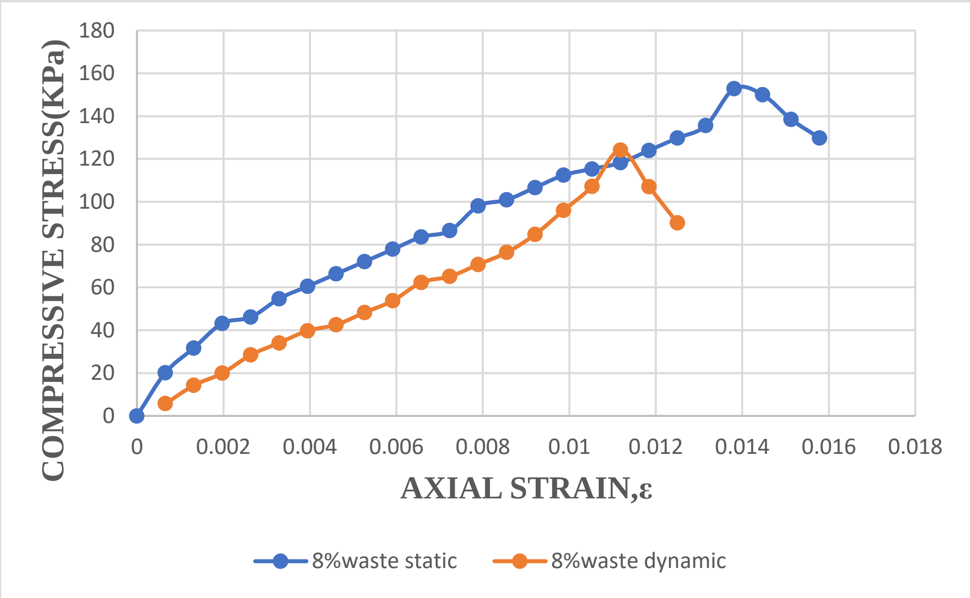

Fig 5.57 Superimposed graph for UCS test for 8% of sugarcane bagasse (statically and dynamically compacted)

From **Fig 5.57**, the maximum value in this percentage of SB was found out to be 152.68KPa by the method of dynamic compaction.

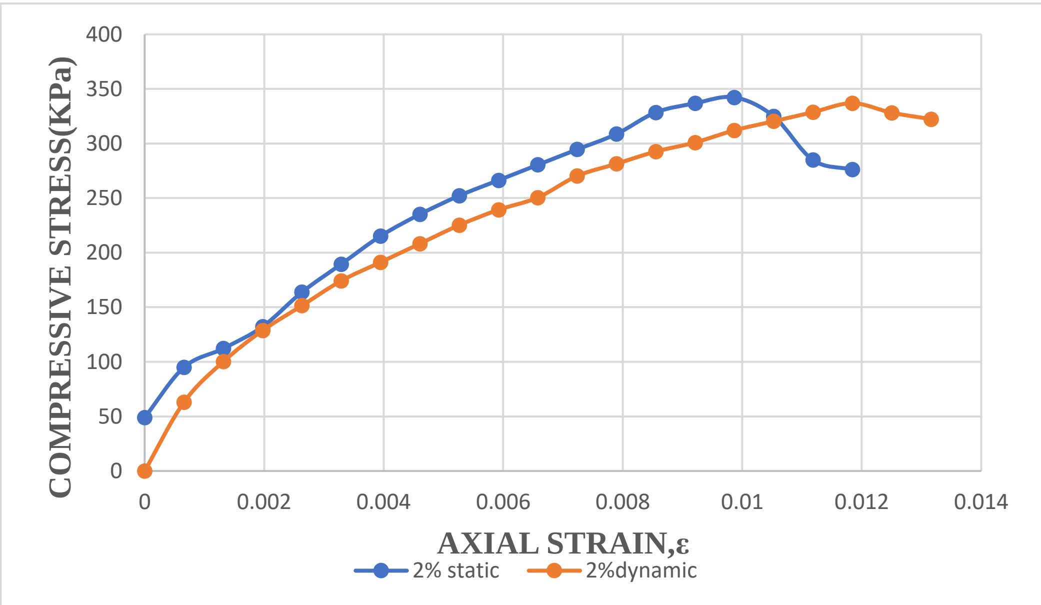

Fig 5.58 Superimposed graph for UCS test for 2% of fly ash (statically and dynamically compacted)

From **Fig 5.58**, the static compaction UCS value shows higher results than dynamic compaction of UCS values. Static compaction UCS = 342.27KPa, Dynamic compaction UCS = 336.93KPa.

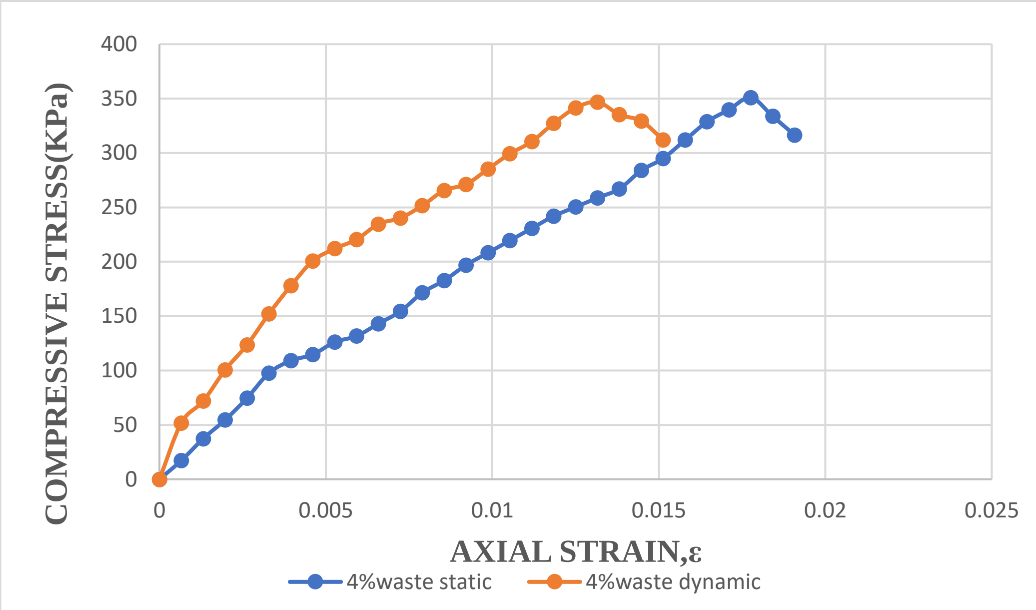

Fig 5.59 Superimposed graph for UCS test for 4% of fly ash (statically and dynamically compacted)

From **Fig 5.59**, the static compaction UCS value shows higher results than dynamic compaction of UCS values. Static compaction UCS = 350.86Kpa, Dynamic compaction UCS = 346.82Kpa.

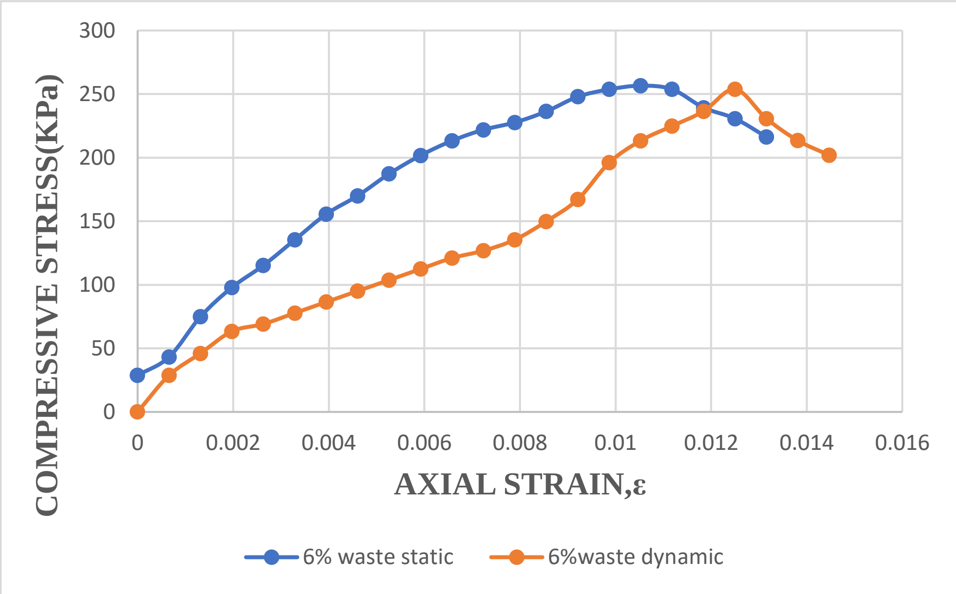

Fig 5.60 Superimposed graph for UCS test for 6% of fly ash (statically and dynamically compacted)

From **Fig 5.60**, the static compaction UCS value shows higher results than dynamic compaction of UCS values.

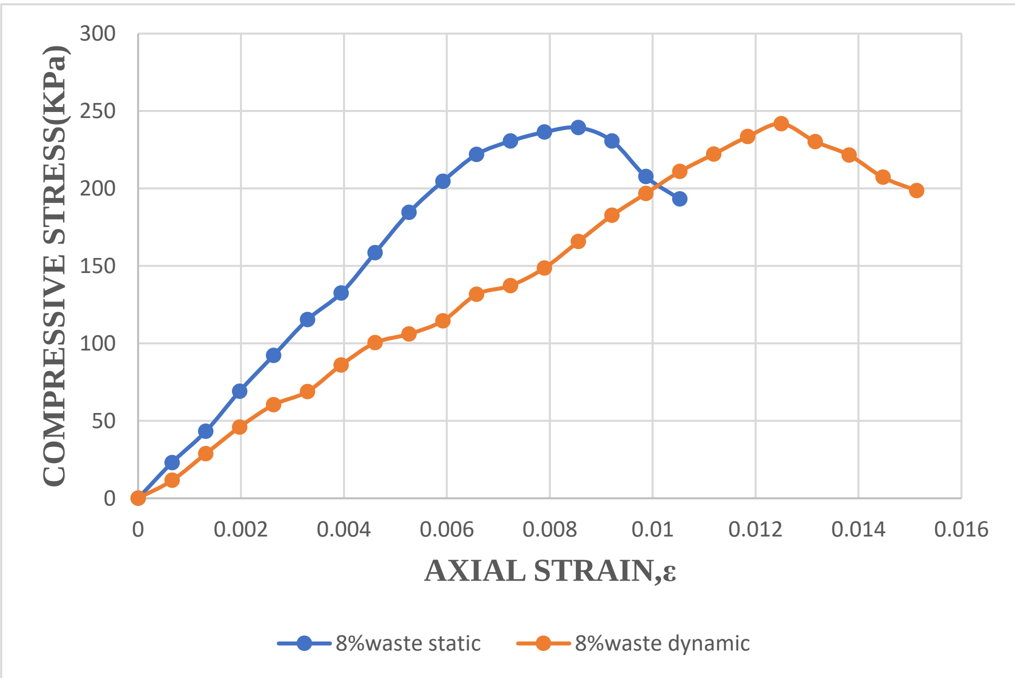

Fig 5.61 Superimposed graph for UCS test for 8 % of fly ash (statically and dynamically compacted)

From **Fig 5.61**, the dynamic compaction UCS value shows higher results than static compaction of UCS values.

CHAPTER 6
DISCUSSION OF TEST RESULTS

Laboratory tests were carried out considering both the waste i.e., sugarcane bagasse (SB) and fly ash. One is considered to be a natural waste and the other is considered as an industrially available waste. Both the waste showed different characteristics on its addition to the soil. Both the waste is added solely to the soil and results were noted down. Using sugarcane bagasse, in the Standard Proctor test, the optimum waste was found out to be on addition of 2% waste and after that its maximum dry density decreases. The sugarcane bagasse is a fibrous material which contain polysaccharides which are best known to bind soil particles together to gain maximum strength by reducing void ratio. At 2% addition of waste, the OMC of the soil also decreases compared to the untreated soil which means that at 2% optimum waste addition the strength attained by the soil is found out to be maximum and the particles of the soil gets maximum compacted.

Again, similarly in Unconfined Compressive Strength test, the maximum stress was found out to be on 2% waste addition in case of dynamic compaction whereas in case of static compaction the maximum strength was found out to be on 2% waste addition showing a very small difference as the method of compaction plays a very important role while comparing static and dynamic compaction. In case of dynamic compaction load is applied in three layers with dynamic force which creates a fact that not every layer gets equally compacted but in case of static compaction equally load is applied instead of in layers so the soil gets equally compacted keeping the homogeneity of the soil constant. So, in both the scenarios optimum percentage of waste addition was found out to be at 2% which gives maximum result. Compaction is a process that essentially alters the soil structure. A/c to Dario et.al., there can be predominance of interparticle force that were destroyed by the dynamic compaction, producing structure with low strength. The static compaction values of UCS at varying percentages of waste shows high UCS values compared to the UCS values by dynamic compaction. SB alone can give strength to the soil but studies also suggest that SB along with other additive like mixing the bagasse with lime or cement can give maximum strength to the soil and durability as well.

But in CBR test, the percentage of waste addition were 0.5%, 1%, 1.5%, and 2% in which the optimum value of CBR was found out to be on 1% addition of waste and after that it decreases. SB can show even better results with addition of other materials as mentioned above as lime or cement etc. This happens because SB alone doesn't content any chemicals within it, it is a natural waste but to give more prominent result this can also be used in combination to other materials having chemical values.

Another waste that was considered in this study is fly ash which is an industrially and easily available waste material. Using this waste, Liquid limit, plastic limit, Standard proctor test, Unconfined Compressive strength test (static and dynamic) and CBR test were done. In liquid limit test, as the percentage of waste increases, the liquid limit of the soil decreases. This happens because addition of fly ash reduces the thickness of the diffuse double layer of the clay particles which causes flocculation of clay particles and increase the coarser particle content by substituting finer soil particles with coarser fly ash particles which causes decrease in liquid limit of the soil (Bidula Bose 2012). It has been found out that maximum decrease in liquid limit was found out to be 14% as compared to the untreated soil on 8% addition of fly ash. Again, in plastic limit test, the plastic limit of the soil decreases on increasing waste percentage. Maximum decrease in percentage is 5.4% which was found to be on 8% addition of waste. Fly ash is also of non-plastic nature which on addition to soil decreases the plasticity of the soil up to a certain extent.

In Standard Proctor test, it has been seen that the OMC of the soil decreases with addition of waste but the MDD of the soil increases. This can be explained as the water content increases, the particles develop larger and larger water films around them, which tend to lubricate the particles and make them easier to be move one over another and come to a denser configuration, resulting in a higher dry unit weight and lower air voids. But the dry unit weight continues to increase till the optimum moisture content is reached. Beyond it, the water starts to replace the soil particles and dry density decreases. In case of cohesive soil, there is an attractive force which is van der Waals' force which acts between the soil particles and a repulsive force is directly related to the size of diffuse double layer. If the net force between the particle is attractive then flocculated structure takes place whereas if the force between the particle is repulsive then dispersed structure takes place. At lower water content, the diffuse layer is not fully developed and the force between particles is attractive force. As the water content is increased, the double layer expands and the interparticle repulsive force increases making the particles slide one over another and gets more closely packed resulting in higher dry unit weight. The double layer expansion is complete at the optimum moisture content and that’s the reason dry unit weight bis maximum at this stage. But beyond OMC, the water tends to occupy space of soil grains resulting in decrease in dry density of the soil. So, on increasing addition of fly ash the OMC decreases and MDD increases which means that at lower water content, maximum dry density of the soil increases. At 8% addition of waste, the OMC decreased is maximum i.e., from 21.2% to 15.35% with a decrease percentage of 5.88%. Whereas the maximum dry density also attained at 8% addition of waste than untreated soil which is from 1.67g/cc to 1.75g/cc.

In the Unconfined compressive strength test by dynamic compaction, the maximum strength was found out to be on 4% addition of waste which is 346.82Kpa which falls under the category of very stiff soil and after that it decreases. This indicates that the quantity of fly ash induces pozzolanic reaction and cemented material because of which the strength of the soil increases, while additional quantity of fly ash acts as unbounded silt particles, which has neither appreciable friction and cohesion causing decrease in strength of the soil (Bidula Bose 2012). But in static compaction, maximum strength was found out to be 350.86Kpa on 4% addition of waste and after that it decreases. This may happen because of the mode of compaction between both the methods. But the static compaction values of UCS are found out to be high compared to the dynamic compaction of UCS values because in dynamic compaction the structure of the soil changes due to sudden impact on the soil whereas, in static compaction the load is applied statically with no dynamic force induced in it. Because of this, in static compaction the structure of the soil doesn’t change. Soil structure means the mode of arrangement of the soil particles relative to each other and the force acting between them to hold them in their positions. But in clayey soil, because of its smaller particles it possesses larger specific surface area which creates strong surface bonding forces between particles which results in strong and dense soil structure, thus increasing the strength of the soil. A/C to Hafez et.al, 2011, in dynamic compaction, the bottom layer obtains more energy as compared to the middle and upper layers. The soil specimens prepared by static compaction is stiff, stronger and less plastic than the soil specimens prepared by dynamic compaction. The dynamic compaction method has a desired effect of creating a dispersed soil structure. The static compaction can also be described as a faster and simpler method to be carried out in laboratory in short duration compared to dynamic compaction.

In CBR test, maximum value of unsoaked CBR was found to be on 1.5% addition of waste which is 13.48% and after that it decreases. Sudden increase in CBR value was found out to be on 1.5% addition of fly ash. This happens because the cohesion component of fly ash particles increases giving higher CBR values at 1.5% addition of fly ash (Bidula Bose 2012). This peak value happens because of the better packing of the mixture at constant compactive effort. But the decrease in CBR values happens because of the low rate of strength gain characteristics of the fly ash.

Table 6.1 Tests results showing all values of treated and untreated soil

Test performed	Untreated soil(values)
Liquid limit	39.1%
Plastic limit	22.8%
Plasticity index	16.3%
Specific gravity	2.664
OMC (Standard Proctor test)	21.2%
MDD	1.67g/cc
UCS (Static)	260.10KPa
UCS (Dynamic)	256.93KPa
CBR (unsoaked)	4.21%

Treated soil(values)			
Test performed	%waste	SB	Fly ash
Liquid limit	2%	Nil	29.8%
	4%	Nil	28.8%
	6%	Nil	26.2%
	8%	Nil	25%
Plastic limit	2%	Nil	21.5%
	4%	Nil	20.6%
	6%	Nil	18.1%
	8%	Nil	17.4%
Plasticity index	2%	Nil	8.3%
	4%	Nil	8.2%
	6%	Nil	8.1%
	8%	Nil	7.6%
OMC	2%	16%	19.4%
	4%	15.3%	18.2%
	6%	17%	17%
	8%	17.1%	15.35%
MDD	2%	2.153g/cc	1.63g/cc
	4%	1.210g/cc	1.70g/cc
	6%	1.660g/cc	1.72g/cc
	8%	1.590g/cc	1.75g/cc
UCS (Static)	2%	262.23 KPa	342.27KPa
	4%	259.60 KPa	350.85KPa
	6%	242.27 KPa	256.62KPa
	8%	152.68 KPa	239.27KPa
UCS (Dynamic)	2%	259.41 KPa	336.93KPa
	4%	253.77 KPa	346.82KPa
	6%	234.04 KPa	253.78KPa
	8%	124.07 KPa	241.79KPa
CBR	0.5%	9.32%	5.70%
	1%	9.78%	6.88%
	1.5%	9.34%	13.48%
	2%	8.09%	2.24%

CHAPTER 7
CONCLUSION AND SCOPE FOR FUTURE STUDY

7.1 Conclusions

Following conclusions can be made from the experimental test results are as follows-

1. The liquid limit, plastic limit, plasticity index of the soil was found to be decreased as compared to the untreated soil on increasing addition of fly ash.
2. The liquid limit was found to be lowest on 8% addition of fly ash which indicates that the water holding capacity of the soil decreases as well as the flowing capacity of the soil w.r.t to the untreated soil.
3. In the plastic limit test, the plastic limit of the soil has also decreased compared to the untreated soil. It concludes that the plasticity of the soil decreases w.r.t. to the untreated soil on addition of fly ash.
4. In Standard Proctor test, the maximum dry density was found to be maximum on 2% addition of SB which means that the soil particles got more compacted during 2% waste addition. But after that on increasing addition of waste the OMC increases on 6% and 8% addition of SB compared to the 2% and 4% waste.
5. By performing the Unconfined Compressive Strength test by dynamic compaction, the compressive strength of each sample was determined. From the different percentages of sugarcane bagasse used, the maximum compressive stress was found to be at 2% waste addition. After that the compressive stress gradually decreases and at 8% waste addition the compressive stress decreased the most compared to the other mixes.
6. But in static compaction of Unconfined compressive strength test, the maximum strength of the soil was found out to be on 2% addition of SB and after that it decreases but the results of static compaction are more compared to the dynamic compaction.
7. In the Standard Proctor test on addition of fly ash, the OMC of the soil decreases on increasing waste content whereas the MDD of the soil increases.
8. In UCS test by dynamic compaction, the maximum strength attained at 4% addition of fly ash. Similarly, by static compaction maximum strength attained at 4% addition of fly ash. Static compaction values are more compared to the dynamic compaction values of UCS.

9. The unconfined compressive strength of statically compacted soil using fly ash exhibit higher values compared to the SB waste. It may be due to difference in the mode of compaction and interaction of the soil particle.
10. In CBR test, results using fly ash, the peak value is more compared to the SB waste.
11. The soil stabilization with waste fibrous improves the strength of the soil up to a certain extent for the clayey soil after that the strength decreases.
12. Comparing both the waste, fly ash shows more prominent result compared to the sugarcane bagasse.
13. In comparison of static and dynamic compaction of UCS, results using fly ash are more compared to the sugarcane bagasse which may be due to least disturbance in static compaction.
14. In static compaction using both the waste shows high UCS values compared to the values by dynamic compaction.

7.2 Scope for future study

1. Study can be carried out using waste percentage of fly ash beyond 8%.
2. Study by using sugarcane bagasse can be extended by using it in the forms of ash.

3.Rather the study with sugarcane bagasse can be carried out by using it with other chemically active waste like cement, lime etc.

4.Comparison can be carried out using CBR test by method of both dynamic compaction and static compaction.

5.In unconfined compressive strength test, curing of the samples can also be carried out at 3days, 7days and 21days to check in its strength.

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