A STUDY ON
IMPROVEMENT OF SOIL PROPERTIES USING STABILIZERS

A dissertation submitted in Partial Fulfillment of the Requirement for the Award of the Degree
of

MASTER OF TECHNOLOGY

IN

CIVIL ENGINEERING

(*With specialization in Geotechnical Engineering*)

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DECLARATION

I, hereby declare that the work presented in the dissertation **"A STUDY ON IMPROVEMENT OF SOIL PROPERTIES USING STABILIZERS"** 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, and guidance 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 improvement by various methods such as controlled compaction or addition of admixtures like lime, fly ash 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 marble dust,stone dust, brick dust, demolition materials, rice husk, bagasse ash, wheat husk, 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 marble dust and waste plastic as 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
MD	Marble Dust
WP	Waste Plastic
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

CONTENTS

CHAPTER 1

INTRODUCTION

1.1 General

Soil is the fundamental construction material in the geotechnical engineering field. Soil, an unconsolidated material with significant chemical composition variability, is either a foundation component or a raw material used in construction. Therefore, soil properties are expected to be influenced by 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. In general, soil is composed of solid minerals, liquids, organic materials, and gasses. 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 enhances soil shear strength and control of shrink-swell properties, improving load bearing capacity for pavements and foundations by increasing the sub-grade's load-bearing capacity. 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 by adding admixture or cost-effective practices like most importantly use of waste materials. The stabilising materials can be natural or industrial wastes. Agricultural and farming wastes including rice husk ash,bagasse ash,chicken eggshells, Fly ash, ground granulated blast furnace slag (GGBFS), Marble powder (MP), Plastic waste, Portland cement, lime,sand, waste glass, bacteria,construction and demolition (C&D) waste etc. can be used for the purpose. Also, fibrous materials like jute fibre, polypropylene fibre, coir fibre etc. can be used. Rice husk is an agricultural waste generated during the processing of rice from paddy in rice mills. Most waste materials are non-biodegradable, leading to environmental pollution and significant impacts on human life and biodiversity. Wastes can be a cost-effective solution for improving soil properties if they serve as a beneficial addition agent. 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. As soil scarcity and location become more challenging and expensive, the importance of improving soil quality through soil stabilization is increasing. 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 are exploring the use of industrial wastes like rice husk and fly ash to improve soil's geotechnical properties. 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 being used are marble dust and waste plastic. **Parte Shyam Singh** and **R. K. Yadav(2017)** conducted a study on the impact of marble dust on the engineering characteristics of black cotton soil. The study found that the addition of marble dust to black cotton soil altered the Procter compaction parameters. The addition of marble dust to BC soil has resulted in a decrease in the OMC and an increase in the maximum dry density. **Okagbue** and **Onyeobi's (1999)**study demonstrated that the addition of marble dust significantly enhances the geotechnical parameters of red tropical soils. The study found that the strength and CBR values increased with the reduction of plasticity and the highest value was achieved with 8% marble dust. **Sabat and Nanda (2011)** study on soil stabilization with marble dust on highway shoulders found that adding marble dust reduces clay content and increases coarser particles. The process decreases the liquid limit, increases the shrinkage limit, and decreases the plasticity index of the soil, thereby affecting the swelling percentage. In **Shiva Kumar et al.(2016)** study, the addition of plastic strips to black cotton soils was conducted, and samples were tested for the presence of UCS and CBR. The strips were added in various percentages by dry weight (0.05%, 0.1%, 0.15%, 0.2%) with dimensions of 3 mm x 20 mm. The study found that at 0.2% plastic, the maximum dry density is only 0.1% higher than plain clay soil, and this is achieved at similar water content. The maximum dry density decreases with an increase in optimum water content beyond 0.2% plastic. The addition of plastic waste strips to clay soil increases its UCS strength by up to 0.2%. In **Arpitha et al.(2017)** Shopping bags were used to reinforce clay soil, and California Bearing Ratio (CBR) tests were conducted on randomly reinforced soil with varying plastic strip percentages 0.5%, 1%, 1.5%, 2%, 2.5%. CBR was improved by adding of waste bags strips in soil with appropriate amounts improved strength and deformation behavior of sub grade soils substantially. The addition of waste bags strips to soil significantly improved the strength and deformation behavior of sub-grade soils.

1.2 Types of Soil Stabilization Techniques

Soil stabilization methods can be categorized into 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 examples of the first type, which enhances soil's inherent shear strength, while the second type involves : mechanical stabilization, stabilization with cement ,lime, bitumen, and chemicals etc.

1)Mechanical stabilization: This refers to the alteration of soil compaction through the addition or removal of specific elements, also known as 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 provides strength and hardness, while the fine fraction offers cohesion and acts as a filler for the coarse fraction's voids.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 surfaing the liquid limit should not exceed 35% and plasticity index should be between 4-9.

2)Lime stabilization: Hydrated(slaked) lime is highly effective in treating heavy, plastic clayey soils. Lime can be used alone or in combination with cement, bitumen, or fly ash. Lime is primarily utilized for stabilizing the foundations and subgrades of roads. Lime is a natural chemical that decreases 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.

3)Cement Stabilization: Engineered soil, water, and Portland cement mixture creates a semi-bound material with granular properties, improving soil shear and compressive strength. Cement, with advanced properties, is one of the cheapest binders available globally, with unit prices varying based on distribution network and manufacturing plant proximity.

4)Bitumen stabilization: Bituminous soil stabilisation is a widely used method using bitumen, asphalt, and tar materials. Bitumens are hydrocarbons, asphalts are petroleum-based, and tars are bituminous condensates from organic materials. Bituminous material stabilizes soil by binding particles or protecting it from water damage, or both effects may occur together. The mechanism of bitumen stabilization primarily involves asphalt in cohesionless and cohesive soils.

5)Chemical stabilization: Chemical stabilization is a process that utilizes reagents like quicklime, Calciment Lime Kiln Dust LKD, cement, or other industrial co-products to enhance the strength of subgrade soil. 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 temperature of the order of 400-600℃ causes irreversible changes in clay minerals. The soil becomes non-plastic, less water sensitive and non-expensive. The method consisted in burning a mixture of liquid fuel and air injected into the ground through a network of pipes.

1.3 Materials used in soil stabilization

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

1.4 Application of Soil Stabilization

The process of soil stabilization is useful inbthe following application

- Reducing the permeability of soils.
- Increasing the bearing capacity of the foundation soils.
- Increasing the shear strength of soils.
- Improving the durability under adverse moisture and stress conditions.
- To enhance unfavourable 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 base of the highway and air fields.

1.5 Marble dust(MD) and its potentiality

Wastes have become an integral part of our daily lives. The disposal and dumping of used and unwanted wastes has become a significant issue for society. The waste materials which I used as a stabilizing agent while performing the laboratory experiments are marble dust and waste plastic. The long-term performance of any building project is significantly influenced by the soundness of the underlying soils. Unstable soils can pose significant challenges to pavements. The issue of inadequate road networks in many regions has led to increased demand and road distress, resulting in frequent repairs. The thickness of pavement layers is influenced by subgrade features, and due to economic and natural factors, a variety of common materials is often used in road preparation. Identifying and treating poor subgrade soils is a crucial objective, with replacing substandard soil being a common option. Road projects are both costly and unrealistic due to the large volume of work involved. The use of waste marble dust as sustainable materials for subgrade stabilization offers significant benefits in reducing environmental pollution. Marble dust is produced through the cutting and polishing of marble stone. It is one of the industry generated waste material. Marble dust, rich in calcium, silica, and alumina, plays a crucial role in soil stabilization.

Fig 1.1 Marble Dust

1.6 Waste plastic(WP)

Plastic strips in geotechnical applications enhance soil engineering properties and regulate plastic waste usage, reinforcing soil with plastic strips for improved soil engineering. Polyethylene waste plastic bags are utilized for soil stabilization in geotechnical infrastructure applications like road bases, embankments, and slope stabilization. The random mixing of plastic waste with soil enhances soil's weak bearing capacity, reduces settlement, provides lateral stability, and increases liquefaction resistance. Stabilization strategies aim to enhance soil strength, resilience, erosion control, workability, and buildability by improving soil workability and resilience. Stabilization can significantly enhance soil properties, rather than simply replacing or scraping the material. The choice of a stabilizing agent can often be influenced by availability or financial consideration. Plastic shopping bags, carrier bags, or plastic grocery bags are a type of plastic bag used as shopping bags and made from various kinds of plastic. Plastic bags are usually made from polyethylene, which consists of long chains of ethylene monomers. Ethylene is derived from natural gas and petroleum. The polyethylene used in most plastic shopping bags is either low-density (resin identification code 4) or, more often, high-density (resin identification code 2). Color concentrates and other additives are often used to add tint to the plastic.

Fig 1.2 Waste Plastic

1.7 Objective of the study

- To imrove the shear strength of the soil and thus increase the bearing capacity.
- To reduce the settlement of the structure on the soil.
- Reduce the water absorption capacity of the soil.
- Increase durability and strength of the soil.
- Reduces plasticity index, lower permeability and reduction of pavement thickness by increasing the bearing capacity by addition of marble dust and waste plastic.
- Comparative study of both marble dust and waste plastic which shows better results on addition to the soil.
- Comparative study of the dynamic compaction and static compaction of Unconfined Compressive Strength Test and CBR 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.
- 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 utilized to enhance soil stability in slopes and other similar locations.
- Sometimes soil stabilization is also used to prevent soil erosion or formation of dust, which is very useful especially in dry and arid weather.
- It is more economical in terms of costs and energy.
- Minimizes and decreases volume instability, swelling and shrinkage control.
- Reduces soil permeability, plasticity index, 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

Sreekumar. V. Babu and Mary Rebekah Sharmila. S(2017) this paper studies about stabilization of expansive soil using marble dust as a stabilizer. Tests that are carried out includes specific gravity, liquid limit, plastic limit, free swell index, Standard proctor test , Unconfined compressive strength test, Clifornia bearing ratio. Marble dust were added in varying percentages (3,6,9,12,15%) along with varying curing periods of (3,7,14) days to the soil sample to determine its strength characteristics. The unconfined compressive strength (UCS) of untreated soil sample is 99.2 kN/m2.On addition of marble dust, unconfined compressive strength increases to 286.5 kN/m2 for 14 day curing period. The California Bearing ratio (CBR) of soil increased from 5.19% for the virgin soil sample to 8.83% for 9% marble dust addition for 0 day curing period. Further addition of marble dust reduces the CBR value. The maximum CBR value was obtained as 14.5% at 14 days curing period for 9% addition of marble dust. The strength characteristics UCC and CBR are increasing up to 9% addition of marble dust and then decreases with further addition. Hence 9% marble dust addition can be regarded as the optimum percentage for stabilizing the soil sample.

Singh, and Yadav, (2014) [3] carried out study on the effect of marble dust on the index properties of black cotton soil. Marble dust was taken in the ratio of 0% to 40% by the dry weight of the soil. Results concluded that the plasticity index of the black soil decreased gradually from 28.35% to 16.67%, while the shrinkage limit increased from 8.06% to 18.34% at 40% addition of marble dust. Apart from this the expansiveness of the soil reduced from being very high to low on addition of marble powder, thus making the soil suitable for construction.

Kumar, et al., (2015) [6] had studied Marble dust addition, showed improved performance in problematic soils with the help of caption exchange reaction. The presence of excess Ca2+ions is responsible for the improved performance. The liquid Limit of soil sample is 61%. Soil sample is classified as Highly Compressible clay (CH). The unconfined compressive strength (UCS) of untreated soil sample is 99.2 kN/m2. On addition of marble dust, unconfined compressive strength increases to 286.5 kN/m2.

Prajapati, et al., (2016) **[7]** carried out the stabilization on silty sands using marble dust and used foundry sand as stabilizing agents. The mix involved utilization of foundry and marble dust separately with the soil starting from 5% upto 30% with a difference of 5%. The testing procedure involved tests like the California Bearing Ratio test, Standard Proctor Test, Hydrometer analysis, Particle Size Distribution, Liquid Limit and Plastic Limit. For the CBR testing results it was observed that the foundry sand helped in increasing the CBR value of the soil while with the addition of marble dust the CBR value decreased. The maximum CBR value i.e. 6.8%, of the soil was achieved with addition of 30% foundry sand by weight of soil as against 4.82% of the normal soil. While the CBR value decreased to 4.3% from 5% with the addition of marble dust. At 30% addition of marble dust the CBR value was found to be least.

Pokale, et al., (2015) Using brick dust as a waste material, the study carried out an experimental inquiry on the stabilization of Black Cotton soil. According to experimental studies, moisture content (MC) drops after 7 and 28 days, with 30% BD MC falling to 26.46%, indicating that replacing brick dust is a more effective solution.

Joe and Rajesh (2015) found that the maximum dry density and ideal moisture content of the soil treated with industrial waste had significantly improved. Using less expensive additives can lower the amount of industrial waste needed while maintaining the same level of material cost. The biggest improvement was shown in the soils, where all of them became non-plastic after receiving enough industrial waste.

Prakash, and Raveendran, (2016) The most common test for figuring out how strong stabilized soil is is the UCC test. The results indicate that the strength characteristics of the soil are improved with the addition optimum percentage of paper sludge when compared to RHA and the improvement was found to be 96%.The cementation of paper sludge and the pozzolanic reaction are responsible for the soil's strength. The results of the tests show that using paper sludge to stabilize the soil can be used as a ground improvement technique for constructions.

Adarsh Minhas (2016) has studied about stabilization of alluvial soil using marble dust and found that the addition of marble powder in the soil sample the OMC increased. The addition of marble powder to alluvial soil results in variation in OMC, with the same variation observed in all three cases (5, 10, and 15%).And prominent improvement seen in CBR values when natural soil is replaced by the addition of marble dust. The addition of marble dust significantly enhances the CBR values when natural soil is replaced.

Tarkeshwar Pramanik, S. Kishor Kumar and J.P. Singh (2016) has studied about the behaviour of Soil for Sub Grade by using Marble Dust and Ground Granulated Blast Furnace Slag and found that The characteristics of soils vary significantly with Marble dust-GGBS content. The Optimum Moisture Content (OMC) increases and Maximum Dry Density (MDD) decreases with increase in percentage of Marble dustGGBS and With increases 20%-20% of Marble dust and GGBS percentage compressive strength of soil increases.. CBR value for soaked and unsoaked condition increases with increases in percentage of Marble dust and GGBS.

Altug (2015) the main objective of this research was to investigate the possibility of utilizing waste marble dust in stabilizing problematic soils (especially swelling clays). The study examined marble dust addition ratios (0%), 5%, 10%, 20%, and 30% by weight, examining physical, mechanical, and chemical properties of soil and marble dust samples.

Stoltz et al. (2014) probed the effect of weathering of lime treated clayey soils by alternate cycles of wetting and drying on the hydro- mechanical properties of the stabilized soil. The results of the study showed a progressive increase in swelling and loss of strength of the stabilized soil with increase in number of wetting and drying cycles.

Tarun Kumar, Suryaketan (2018) has studied about the effect of mixed plastic strips in soil,a series of standard proctor and unsoaked CBR tests have been conducted and based on this it is observed that the maximum dry density of plastic mix soil decreases with increase of percentage of plastic strips, and for CBR increases with increase of percentage of plastic strips within a certain limit. It is observed that the maximum CBR value is obtained when the percentage of the plastic strips is 0.8% of dry weight of soil. Hence 0.8% of strips having length of 2cm is considered as required amount.

Kiran kumar Patil, Shruti Neeralagi (2017) In this study they used plastic bottle strips and plastic bag strips for stabilization. From this study conclusion made is there is increase in CBR value of a soil and maximum CBR is achieved when 0.75% amount of plastic bottle strips are added to the soil after further additionof the stripsthere is decrease in the CBR value. In case of plastic bag strips,it has been observed that2% of the total weigh of the soil is the optimum proportion of the strips, we can also state from this study that strips cut out of plastic bottles are better option than strips of soil bags, to increase the CBR value of the soil.

Sayli D. Madavi, Divya Patel (2017) This study reviews the experimental program conducted for stabilization of black cotton soil in the Amravati,aCapital of newly formed Andhra Pradesh state. They performed series of CBR testings to find out optimum amount of plastic content is required for obtaining maximum CBR value. It can be concluded that CBR percentage goes on increasing up to 4% plastic content inthe soil and thereon it decreases with increasing the plastic content.Hence, we can say that 4% of plastic content is the optimum content of plastic waste in the soil

Sharan Veer Singh, Mahabir Dixit,(2017) This paper focus on the soil stabilization by using plastic waste products.The plastic inclusion can improve the strength thus increasing the soil bearing capacity of the soil. Useof plastic waste as reinforcement which reduces the disposal problem of the waste materials. Research has been done in India to determine the suitability of these waste materials for Indian roads. Based on these the further study is requiredto find out the optimum amount of the percentage of plastic waste content.

Shiva Kumar et al. (2016) study, plastic strips were added to black cotton soils and the sample were tested for carrying unconfined compression test and CBR. The strips were added in various percentages by dry weight (0.05%, 0.1%, 0.15% and 0.2%) with dimensions of 3 mm x 20 mm. It was found that at 0.2% plastic the max dry density is only 0.1% more than plain clay soil and is achieved at similar water content when compared to clay soil. But it is observed that beyond 0.2% plastic the decrease in max dry density with increase of optimum water content. The unconfined compressive strength for clay soil is increased due to inclusion of plastic waste strips the strength of the soil is increased up to addition of 0.2% of plastic strips. The California bearing ratio values are increasing with the increase in percentage of waste plastic strips and it was found CBR increased about 192 % at 0.2% [11].

Arpitha et al. (2017) shopping bags were used to reinforce clay soil and a series of California Bearing Ratio (CBR) tests were carried out on randomly reinforced soil by varying percentage 0.5%, 1%, 1.5%, 2%, 2.5% of plastic strips by weight of soil, with different lengths and proportions. CBR was improved by adding of waste bags strips in soil with appropriate amounts improved strength and deformation behavior of sub grade soils substantially. Also, from the results obtained after performing the test with plastic bag strips that 2% of the total weight of the soil is the optimum proportion of strips cut from waste bags to be included to the soil as reinforcement. But it decreases when furthered amount of plastic bags strips is added.

Agarwal et al. (2015) used expansive soil that was reinforced with plastic bags, with dimensions of 1 cm * 1 cm, 2 cm * 2 cm, 3 cm * 3cm, 4 cm * 4cm. and percentages of 0.05%, 0.1%, 0.15% and 0.2% by mass of soil, and CBR test was carried out. It was found that the most favorable ground stabilization condition could be achieved when plastic bag pieces of size 2 cm * 2 cm were used at a proportion of 0.1%. Under this condition the value of CBR was found increased around 43% than the soil without plastic bags.

Chakraborty et al. (2018), the study was undertaken to evaluate the effects of the waste plastic polyethylene on the geotechnical properties of two locally available sands (Brahmaputra and Kulsi sand). And a series of direct shear tests on the two sand samples reinforced with polyethylene plastic strips were conducted. The influence of changing percentage of bags (0.10%, 0.20%, 0.30%, 0.45%, 0.60%, 0.70% and 0.75% by weight using various dimensions of the bags strips is studied. The polyethylene plastic bags strips' length varied from 15 mm to 45 mm and width varied from 5 mm to 15 mm.

CHAPTER 3

METHODOLOGY

3.1 General

This chapter represents different laboratory tests which were done on untreated soil sample to determine the various properties.

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. To check the behavior of the soil, different tests were done on the untreated soil sample. 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 compression test(static and dynamic) and CBR(static & dynamic, soaked & unsoaked) . 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 marble dust mixed to identify how its behavior changes with the addition of waste. About 100-110kg of soil sample is collected from Assam Engineering College , Hostel 6,Guwahati. After the collection of the soil samle, 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 samles results in disturbance of the soil structure.

Fig 3.1 Collection of soil sample from AEC, Hostel 6

Fig 3.2 Air dried soil sample after collection

3.4 Collection of waste material:

The wastes that are used in this study are marble dust and plastic waste. Marble has been collected from a tiles shop near Jalukbari (Tiles N Sanitary Mart) and after collecting it is grined using hammer. Secondly, plastic has been collected from a grocery shop located at Sundarbari, Guwahati.

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 code 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 (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 sieve used for performing this experiment is 425μ passing.

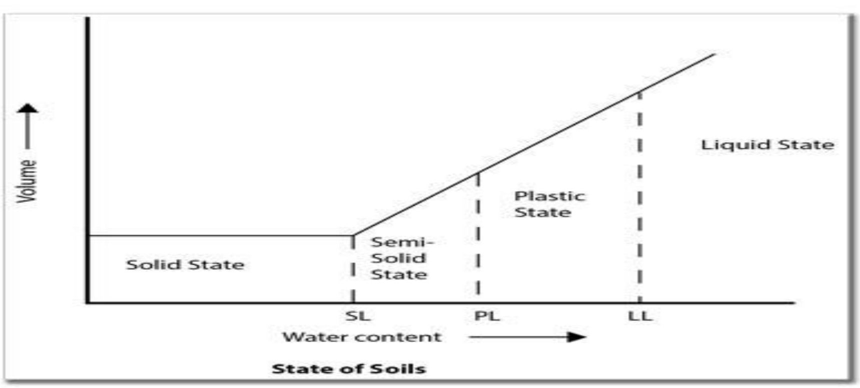


Fig 3.3 Consistency limits of soil

(Source:https://www.google.com/search?q=consistency+limits+of+soil)

Fig 3.4 Liquid limit test

(Source: https://www.google.com/search?q=liquid+limits+of+soil+cone+pene)

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_P-W_L

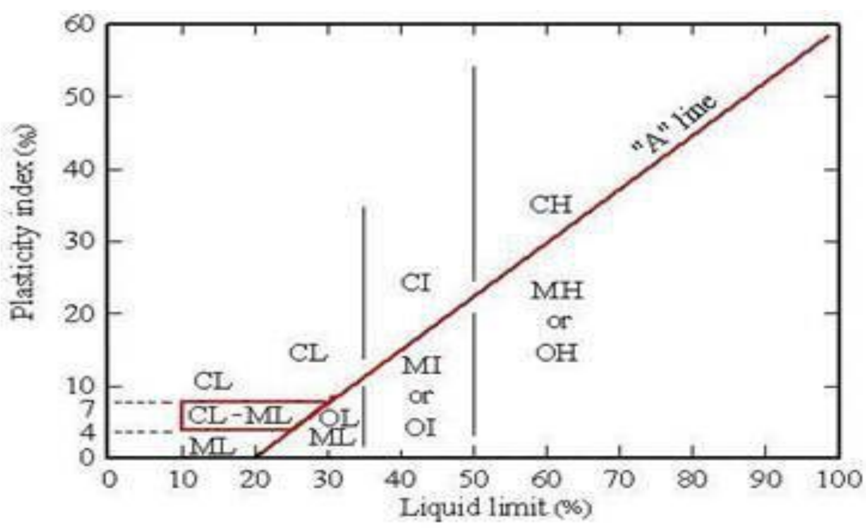


Fig 3.5 Plasticity Chart
(Source: https://www.google.com/search?q=plasticity+chart&oq=plasticity+chart**)**

Fig 3.6 Plastic Limit thread

3.5.4 Detremination 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℃. 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 blance, 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 denoted as M_1
- 5-10 g 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-

$G = M_2 - M_1/(M_4 - M_1) - (M_3 - M_2)$

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 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 numbers 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. Inthis 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.

3.5.5.2 Static compaction method:

In static compaction processes, the soil is compacted by a gradually applied static force. In practice, the loose soil is confined in a container and compaction is achieved by the gradual movement of a piston.

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 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 3.7 Mixing of sample for proctor test(addition of marble dust)

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 specimens fails in compression without any confining pressure.

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

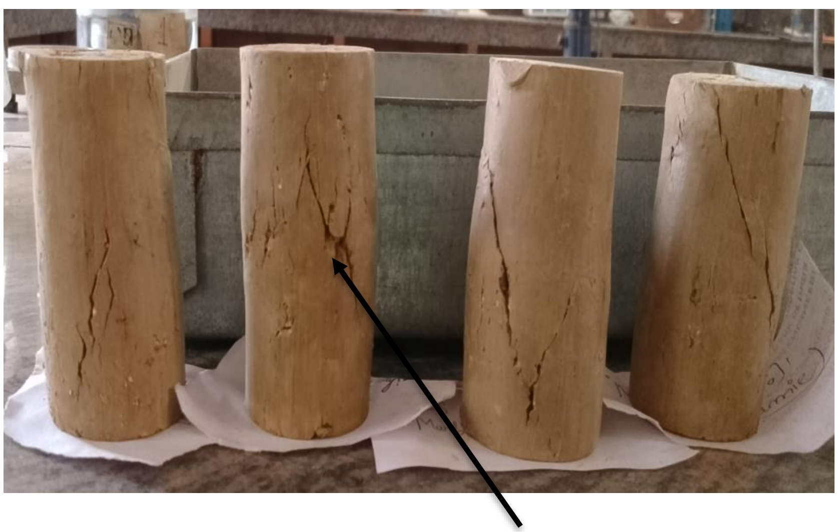
Where P = axial load at failure, A=corrected area= $A_0/(1-\varepsilon)$

A_0= 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 3.8 Unconfined Compression Machine



Failure plane

Fig 3.9 The above picture shows the sample after failure at different waste percentages

3.5.7 Determination of California Bearing Ratio (CBR):

In this experimental study, unsoaked and soaked CBR test are performed on the collected soil sample according to the IS specification IS: 2720 (Part-16)-1987. The unsoaked soil sample are prepared at optimum moisture content and kept it for 24hours maturation. The tests were performed on remolded specimen by means of dynamic compaction. To determine the soaked CBR value, soil sample is soaked in water for 4 days prior to the test.

3.5.7.1 Preparation of soil sample by dynamic compaction:

In this dynamic method of compaction for unsoaked CBR, 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 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 3 layers 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 3.10 CBR mould, base plate collar,spacer disc, rammer

Fig 3.11 Compacted soil with CBR mould after load penetration

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 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 1nd 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 3.12 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 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 x 100

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

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

Table3.2 Standard load used in CBR test

Penetration depth(mm)	Unit standard load(kg/cm^3)	Toatl 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 reults 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 of soil sample

SIEVE SIZE(mm)	RETAINED(g)	% RETAINED	CUMMULATIVE % RETAINED	% FINER
4.75	0	0	0	100
2	2.49	1.24	0.92	99.08
1.18	3.82	1.91	2.83	97.17
0.6	3.32	1.66	4.50	95.51
0.425	9.36	4.68	9.18	90.82
0.3	2.26	1.13	10.31	89.69
0.15	3.56	1.78	12.08	87.92
0.075	4.50	2.25	14.33	85.67

Sand = 100 - 85.67= 14.33%

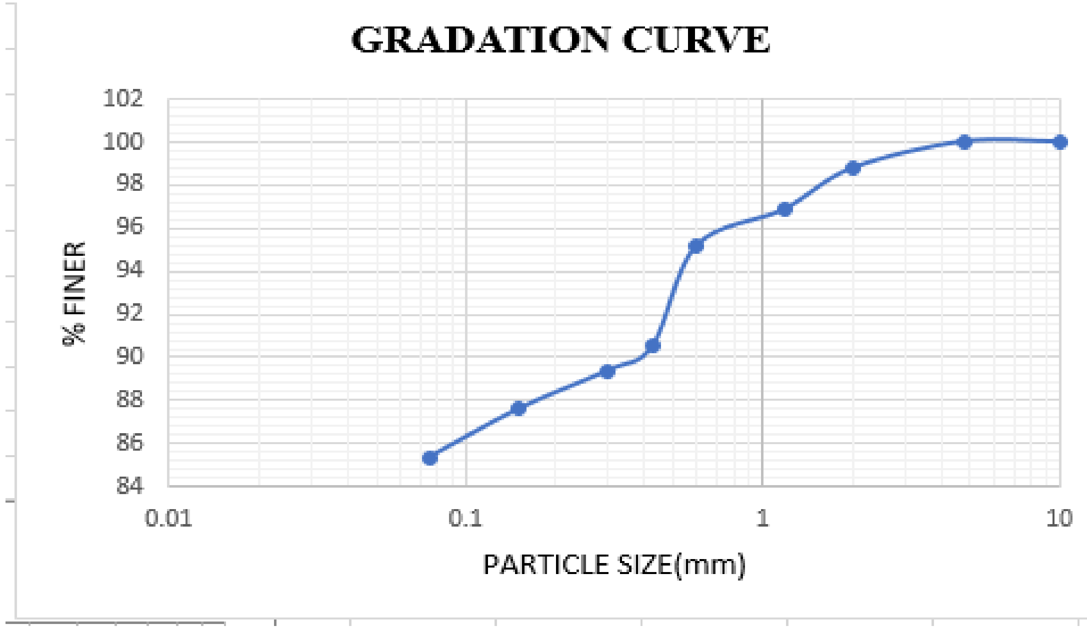


Fig 4.1 Particle Size Distribution Curve of soil sample

Table 4.2 Particle size distribution of Marble Dust

Total mass of Waste Marble Dust = 500g

SIEVE SIZE(mm)	RETAINED(g)	%RETAINED	CUMMULATIVE % RETAINED	%FINER
10	0	0	0	100
4.75	5.081	1.01	1.01	98.98
2	20.29	4.06	5.07	94.93
1.18	71.88	14.38	19.45	80.55
0.6	76.94	15.39	34.83	65.16
0.425	57.39	11.48	46.31	53.68
0.3	47.08	9.41	55.73	44.27
0.15	79.16	15.83	71.56	28.43
0.075	54.20	10.84	82.40	17.59

Fineness Modulus(Waste Marble Dust) = 3.16

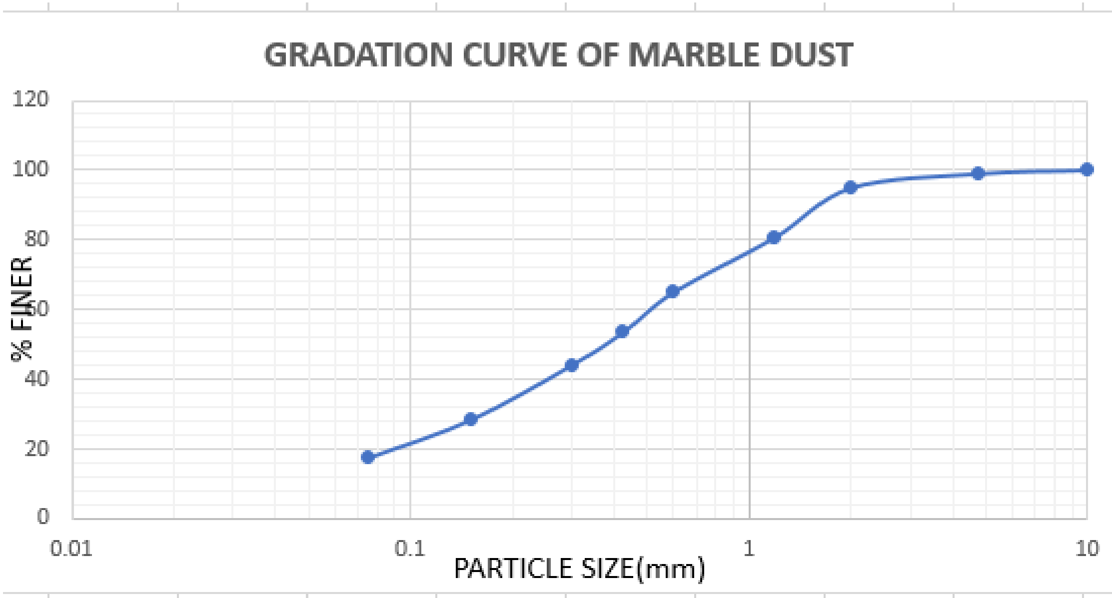


Fig 4.2 Particle Size Distribution Curve of Marble Dust

4.2.2 Liquid Limit Test

Total mass of the sample = 450g

Table 4.3 Values for water content determination for liquid limit

Cone penetration(mm)	Weight of container(g) M1	Wet mass+ container(g) M2	Dry mass+ container(g) M3	Weight of water(g)	Weight of dry soil(g)	Moisture content (%)
16	7.45	27.01	2.23	4.78	14.78	32.34
18	6.98	24.17	19.52	4.65	13.31	34.93
21	9.22	26.59	21.37	5.22	12.15	42.96
22	11.32	24.10	19.87	4.23	8.55	49.47
23	10.12	23.61	19.38	4.23	9.26	45.68

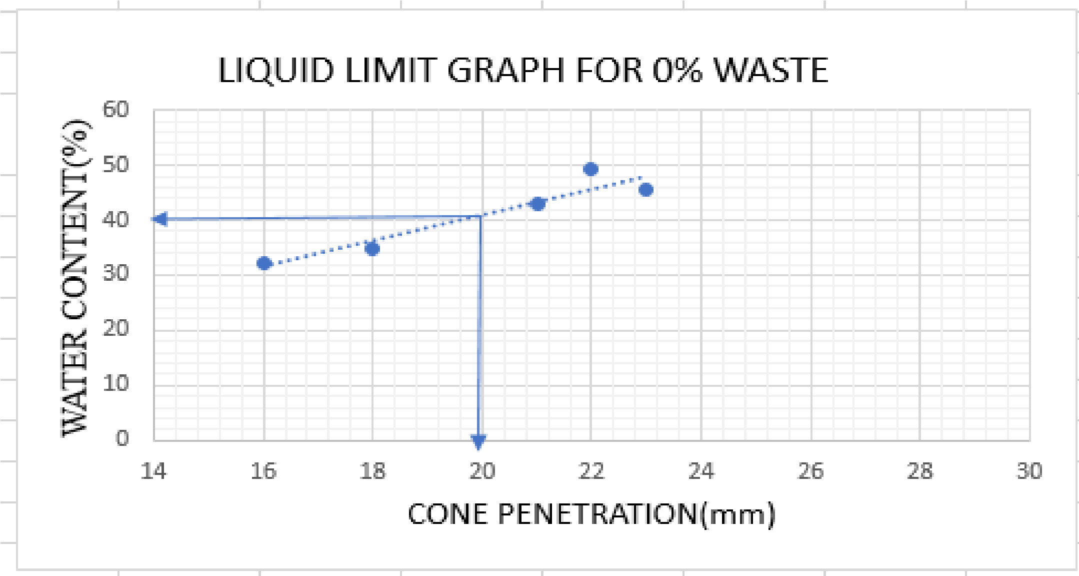


Fig 4.3 Liquid limit graph for 0% waste

Liquid limit (LL) = 41.08%

4.2.3 Plastic limit test

Total mass of the sample taken = 450g

Table 4.4 Values for water content determination for plastic limit

Sl No	WT.OF CONTAINER(g)	WT.OF CONTAINER +MOIST SOIL(g)	WT.OF CONTAINER +DRY SOIL(g)	WT.OF WATER (g)	WT.OF DRY SOIL(g)	WATER CONTENT (%)
1	8.953	9.444	9.351	0.093	0.398	23.367
2	8.316	10.419	10.035	0.384	1.719	22.339
3	9.544	11.006	10.725	0.281	1.181	23.793

Plastic limit= 23.17%, from **Table 4.4**

Plasticity index = 17.91%

A-line(PI) = 15.39

Soil type = CI soil(Clay with Intermediate Plasticity)

4.2.4 Determination of specific gravity

Total mass of the sample taken = 5-10g

Table 4.5 Specific gravity values

Density bottle no	Mass of density bottle and stopper,M1	Mass of density bottle,stopper and soil.M2	Mass of density bottle,stopper,soil and water,M3	Mass of density bottle,stopper and water,M4	Specific gravity(G)
1	27.138	32.282	81.019	77.814	2.653
2	31.789	37.673	86.903	83.214	2.681
3	21.283	28.029	75.017	70.814	2.653

Specific gravity = 2.662

4.2.5 Standard proctor test

Diameter of mould = 100mm
Volume of mould = 1000cc
Height of mould = 127.5mm
Weight of sample taken = 2kg
Empty mould + base plate = 4234g

Table 4.6 Standard Proctor Test values for 0% waste

Mass of compacted soil+mould with base plate(g)	Mass of empty container(g)	Mass of container+ wet soil(g)	Mass of container +dry soil(g)	Bulk density(kN/m3)	Water content(%)	Dry density(kN/m3)
5979	9.265	20.847	19.461	17.392	13.594	15.311
6060	9.721	20.11	18.662	18.185	16.195	15.65
6115	8.637	20.334	18.50	18.726	18.595	15.79
6130	8.563	19.98	18.125	18.874	19.40	15.808
6126	9.345	20.628	18.510	18.835	23.110	15.30
6121	9.643	21.506	19.106	18.806	25.362	15.001

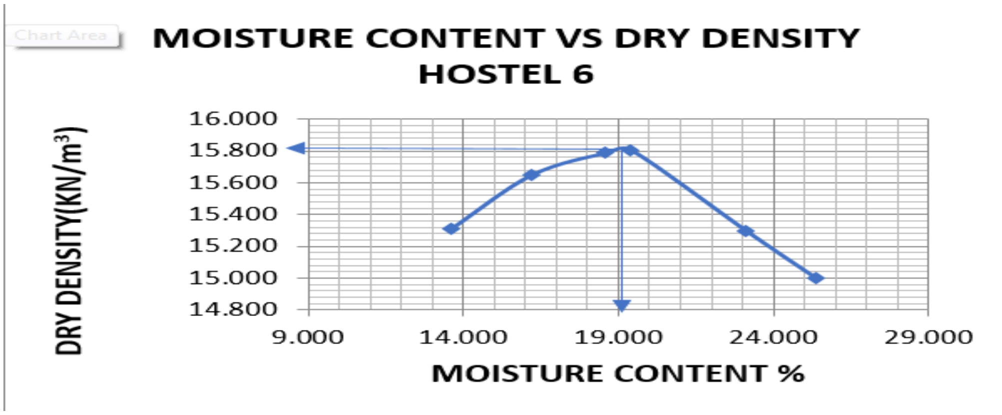


Fig 4.4 Compaction curve for 0% waste

Results – OMC(Optimum Moisture Content)= 19.1%(From Fig4.4)
MDD(Maximum dry density) = 15.88kN/m^3

4.2.6 Unconfined compressive strength test

Initial diameter= 38mm

Initial length = 76mm

Initial area = 11.34cm²

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

Table4.7 Unconfined compression test values for 0% waste(Static compaction)

DEFORMATION DIAL READING	AXIAL DEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA (cm²)	PROVING RING DIAL READING	AXIAL FORCE(Kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	24	7.92	68.45
100	0.1	0.00132	11.35	31	10.23	88.35
150	0.15	0.00197	11.36	35	11.55	99.69
200	0.20	0.00263	11.37	38	12.54	108.16
250	0.25	0.00329	11.38	41	13.53	116.62
300	0.30	0.00461	11.38	45	14.85	127.91
350	0.35	0.00526	11.39	48	15.84	136.35
400	0.40	0.00526	11.40	51	16.83	144.78
450	0.45	0.00592	11.41	55	18.15	156.03
500	0.50	0.00658	11.42	58	19.14	164.43
550	0.55	0.00724	11.42	61	20.13	172.82
600	0.60	0.00789	11.43	65	21.45	184.03
650	0.65	0.00855	11.44	68	22.44	192.40
700	0.70	0.00921	11.45	71	23.43	200.75
750	0.75	0.00987	11.45	75	24.75	211.92
800	0.80	0.01053	11.46	78	25.74	220.25
850	0.85	0.01118	11.47	80	26.4	225.75
900	0.90	0.01184	11.48	83	27.39	234.06
950	0.95	0.01250	11.48	86	28.38	**242.36**
1000	1.00	0.01316	11.49	78	25.74	219.67
1050	1.05	0.01382	11.50	67	22.11	188.56

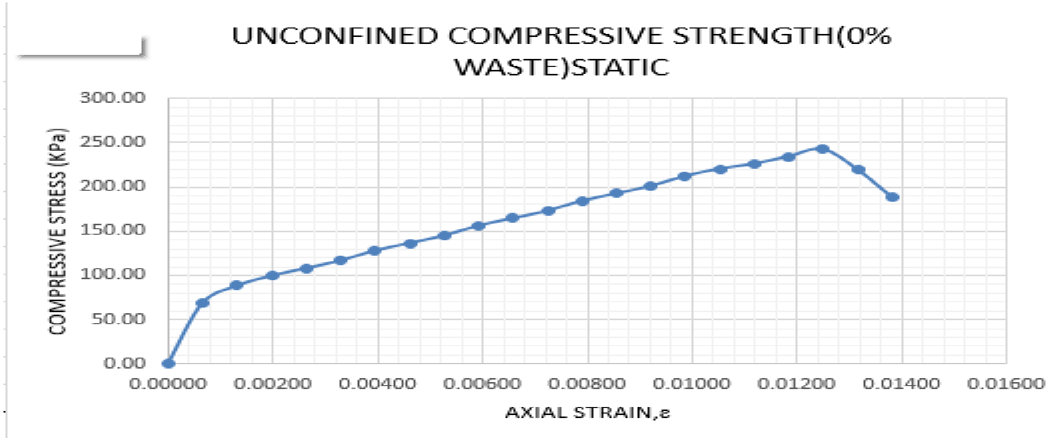


Fig 4.5 UCS graph for 0% waste (Static compaction)

Compressive stress = 242.36 kPa

Table4.8 Unconfined compression test values for 0% (Dynamic compaction)

DEFORMATION DIAL READING	AXIALDEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	24	7.92	68.45
100	0.1	0.00132	11.35	31	10.23	88.35
150	0.15	0.00197	11.36	35	11.55	99.69
200	0.20	0.00263	11.37	38	12.54	108.16
250	0.25	0.00329	11.38	41	13.53	116.62
300	0.30	0.00461	11.38	45	14.85	127.91
350	0.35	0.00526	11.39	48	15.84	136.35
400	0.40	0.00526	11.40	51	16.83	144.78
450	0.45	0.00592	11.41	55	18.15	156.03
500	0.50	0.00658	11.42	58	19.14	164.43
550	0.55	0.00724	11.42	61	20.13	172.82
600	0.60	0.00789	11.43	65	21.45	184.03
650	0.65	0.00855	11.44	68	22.44	192.40
700	0.70	0.00921	11.45	69	22.77	195.10
750	0.75	0.00987	11.45	71	23.43	200.62
800	0.80	0.01053	11.46	73	24.09	206.13
850	0.85	0.01118	11.47	82	27.06	**231.39**
900	0.90	0.01184	11.48	79	26.07	222.78
950	0.95	0.01250	11.48	76	25.08	214.18

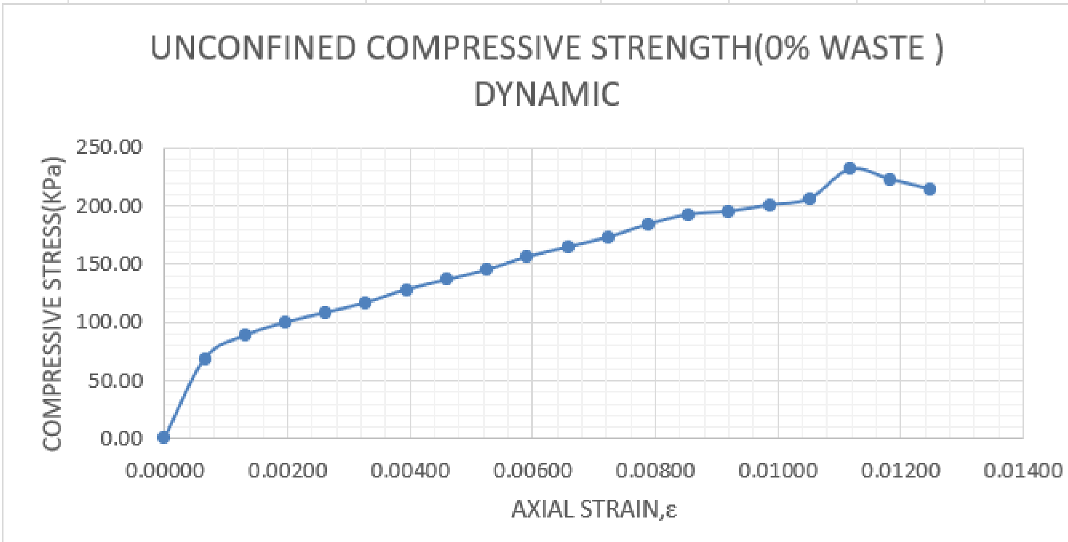


Fig 4.6 UCS graph for 0% waste (Dynamic Compaction)

Compressive stress= 231.39 kPa

4.2.7 CBR Test of untreated soil

Table 4.9 Load penetration data for CBR test of untreated soil (Static Compaction)

PENETRATION(mm)	LOAD ON PISTON(kg)
0	0
0.5	27.3
1	44.7
1.5	52.6
2	61.2
2.5	65.6
3	71.5
4	82.7
5	90.0
7.5	102.2
10	113.8
12.5	119.4

2.5mm CBR value = 4.79%

5.0mm CBR value = 4.38%

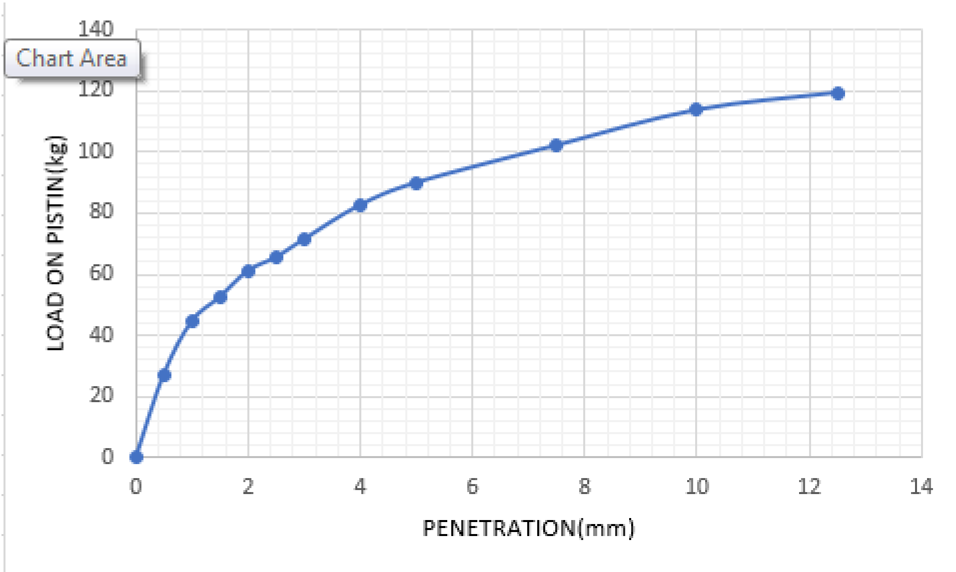


Fig 4.7 Load penetration curve for untreated soil (Static Compaction)

Table 4.10 Load penetration data for CBR test of untreated soil (Dynamic Compaction)

PENETRATION(mm)	LOAD ON PISTON(kg)
0	0
0.5	26.7
1	36.2
1.5	45.1
2	54.2
2.5	61.9
3	71.6
4	78.2
5	84.9
7.5	89.4
10	97.3
12.5	112.7

2.5mm CBR value = 4.52%

5.0mm CBR value = 4.13%

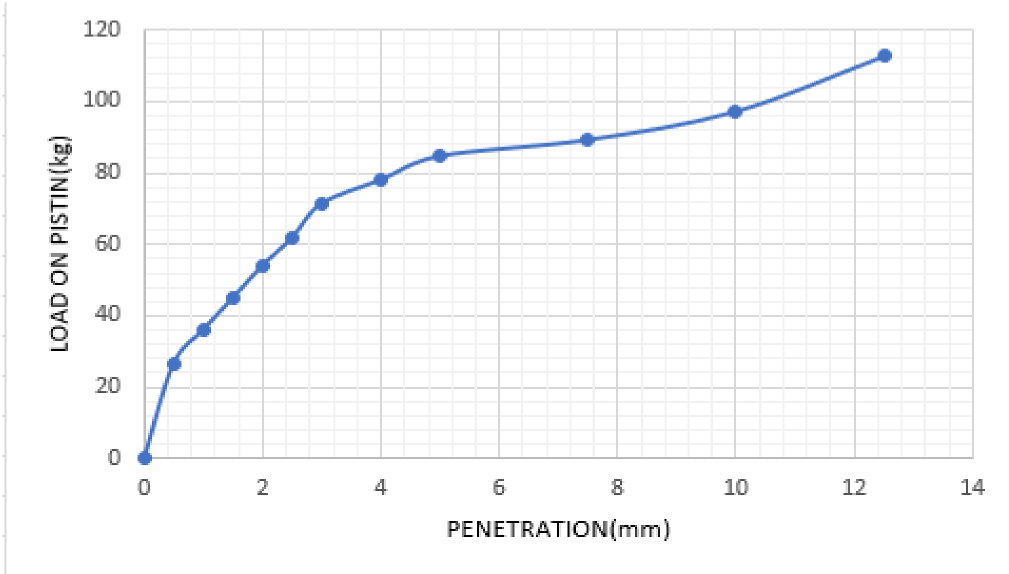


Fig 4.8 Load penetration curve for untreated soil (Dynamic Compaction)

CHAPTER 5
TEST RESULTS WITH ADDITION OF WASTE

5.1 Introduction

This chapter represents different percentages of marble dust added to the soil corresponding results are discussed below-

5.2 Test results of soil mixed with marble dust

Table 5.1 Standard Proctor Test for 2% marble dust

Empty mould +base plate=4234g

Mass of compacted soil+mould with base plate(g)	Mass of empty container(g)	Mass of container +wet soil(g)	Mass of container+dry soil(g)	Mass of water (g)	Mass of dry soil(g)	Bulk density(KN/m3)	Water content (%)	Dry density(kN/m3)
5971	9.955	25.026	23.626	1.4	13.671	17.315	10.241	15.706
6048	8.675	21.581	20.081	1.5	11.406	18.070	13.151	15.970
6127	8.637	20.04	18.44	1.66	9.803	18.845	16.322	16.201
6178	9.674	21.433	19.533	1.9	9.859	19.345	19.272	16.220
6173	9.045	20.091	18.082	2.00	9.037	19.296	22.231	15.787
6167	9.955	21.89	19.49	2.4	9.535	19.257	25.170	15.385

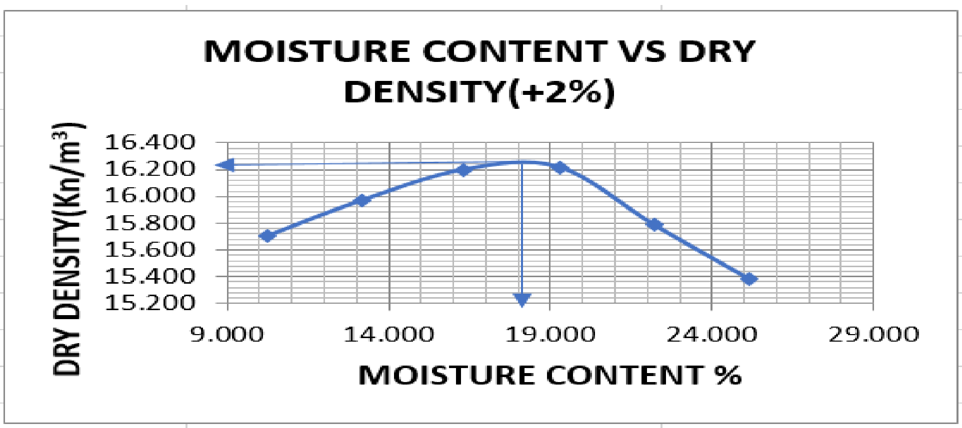


Fig 5.1 Compaction curve for 2% (marble dust)

Here from Fig 5.1 the moisture content of the soil while adding 2% waste decreases to 18.3% and the MDD increases to 16.26 kN/m^3. 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% marble dust

Empty mould +base plate=4234g

Mass of compacted soil+mould with base plate(g)	Mass of empty container(g)	Mass of container+wet soil(g)	Mass of container+dry soil(g)	Mass of water(g)	Mass of dry soil(g)	Bulk density(kN/m3)	Water content(%)	Dry density(kN/m3)
6421	9.145	19.38	18.396	0.984	9.251	21.729	10.637	19.640
6538	8.563	21.141	19.641	1.5	11.078	22.877	13.540	20.149
6681	8.637	19.693	18.093	1.6	9.456	24.280	16.920	20.766
6753	9.721	20.523	18.792	1.731	9.071	24.986	19.083	20.982
6729	9.342	20.124	18.11	2.014	8.768	24.751	22.970	20.127
6696	9.643	21.89	19.49	2.4	9.847	24.447	24.373	19.656

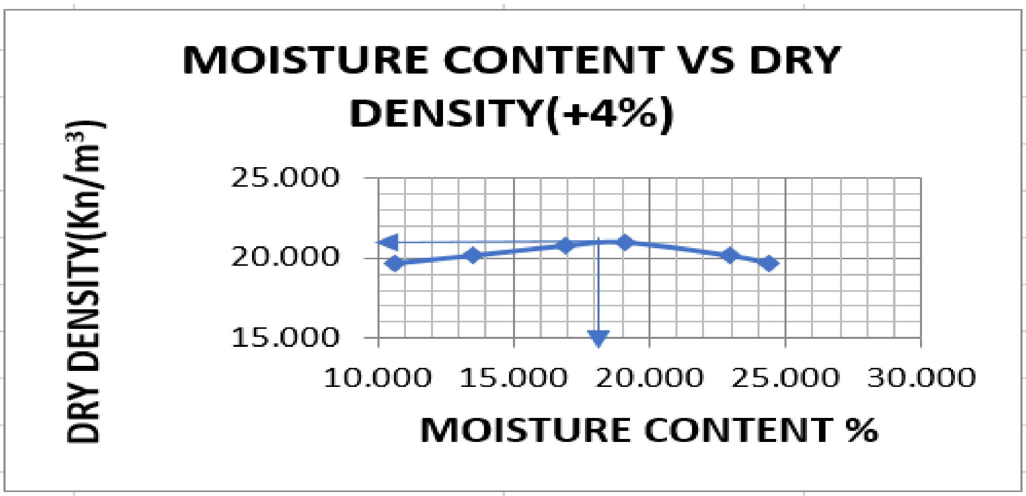


Fig 5.2 Compaction curve for 4% (marble dust)

Here from Fig 5.2 the moisture content of the soil while adding 4% waste decreases to 18.1% and the MDD increases to 21.00 kN/m^3. This means that the strength of the soil increases compared to the 2% waste marble dust with less water content than 2% waste.

Table 5.3 Standard Proctor Test for 6% marble dust

Empty mould +base plate=4234g

Mass of compacted soil+mould with base plate(g)	Mass of empty container(g)	Mass of container +wet soil(g)	Mass of container+dry soil(g)	Mass of water(g)	Mass of dry soil(g)	Bulk density(kN/m3)	Water content(%)	Dry density(kN/m3)
6258	9.145	20.465	19.365	1.1	10.22	20.130	10.763	18.174
6338	8.563	19.521	18.221	1.3	9.658	20.915	13.460	18.434
6439	8.637	20.174	18.574	1.6	9.937	21.906	16.101	18.868
6517	9.721	20.929	19.131	1.798	9.410	22.671	19.107	19.034
6510	9.342	20.662	18.56	2.102	9.218	22.602	22.803	18.405
6502	9.643	21.484	19.084	2.4	9.441	22.543	25.421	17.974

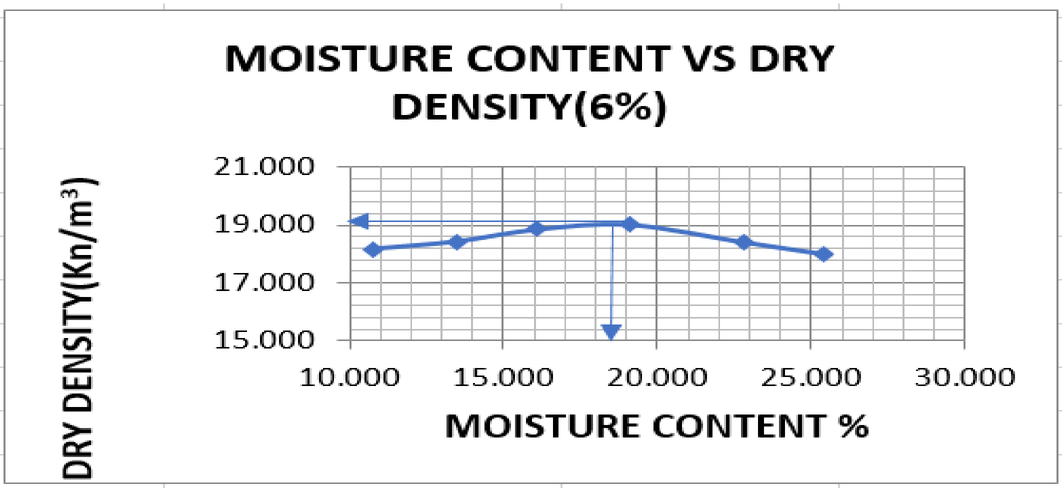


Fig 5.3 Compaction curve for 6% (marble dust)

Here from Fig 5.3 the moisture content of the soil while adding 6% waste increases to 18.6% and the MDD decreases to 19.20 kN/m^3. This means that maximum dry density decreases and moisture content increases.

Table 5.4 Standard Proctor Test for 8% marble dust

Empty mould +base plate=4234g

Mass of compacted soil+mould with base plate(g)	Mass of empty container(g)	Mass of container+wet soil(g)	Mass of container +dry soil(g)	Mass of water(g)	Mass of dry soil(g)	Bulk density(kN/m3)	Water content(%)	Dry density(kN/m3)
6208	9.342	21.164	20.063	1.101	10.721	19.64	10.270	17.811
6300	9.721	19.863	18.662	1.201	8.941	20.542	13.433	18.11
6373	8.637	20.107	18.5	1.607	9.863	21.258	16.293	18.28
6448	8.563	20.025	18.125	1.9	9.562	21.994	19.87	18.348
6442	9.145	20.717	18.61	2.107	9.465	21.935	22.261	17.941
6423	9.643	21.506	19.106	2.4	9.463	21.768	25.362	17.364

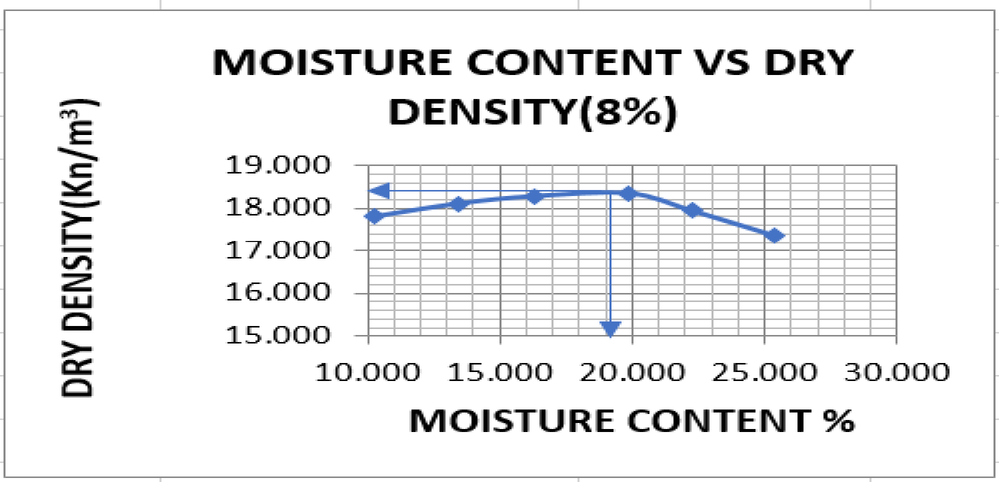


Fig 5.4 Compaction curve for 8% (marble dust)

Here from Fig 5.4 the moisture content of the soil while adding 8% waste increases to 19.3% and the MDD decreases to 18.4 kN/m^3. This means that maximum dry density decreases and moisture content increases.

Table 5.5 Maximum dry density V/s percentage of marble dust

Percentages of waste increment(%)	Maximum dry density(kN/m^3)
0%	15.88
2%	16.26
4%	21.00
6%	19.2
8%	18.4

Results: Maximum dry density attained at 4% marble dust

Whereas after that the Maximum dry density decreases. This makes a statement that minimum waste addition should be 4% because after that the dry density decreases. To attain maximum strength of the soil stabilization, minimum 4% is necessary.

Table5.6 Unconfined compression test values for 2% marble dust (Static compaction)

DEFORMATION DIAL READING	AXIALDEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(Kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	25	8.25	71.30
100	0.1	0.00132	11.35	38	12.54	108.30
150	0.15	0.00197	11.36	48	15.84	136.71
200	0.20	0.00263	11.37	53	17.49	150.85
250	0.25	0.00329	11.38	58	19.14	164.98
300	0.30	0.00395	11.38	61	20.13	173.39
350	0.35	0.00461	11.39	67	22.77	196.00
400	0.40	0.00526	11.40	74	24.42	210.07
450	0.45	0.00592	11.41	76	25.08	215.60
500	0.50	0.00658	11.42	80	26.4	226.80
550	0.55	0.00724	11.42	82	27.06	232.32
600	0.60	0.00789	11.43	83	27.39	234.99
650	0.65	0.00855	11.44	85	28.05	240.50
700	0.70	0.00921	11.45	86	28.38	243.17
750	0.75	0.00987	11.45	87	28.71	245.83
800	0.80	0.01053	11.46	88	29.04	248.49
850	0.85	0.01118	11.47	90	29.7	253.97
900	0.90	0.01184	11.48	92	30.36	259.44
950	0.95	0.01250	11.48	94	31.02	**264.90**
1000	1.00	0.01316	11.49	89	29.37	250.65
1050	1.05	0.01382	11.50	85	28.05	239.22

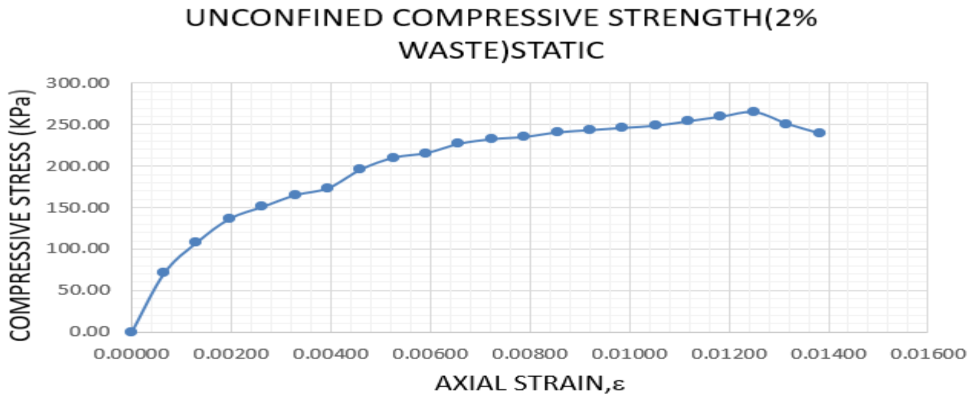


Fig 5.5 Unconfined compressive strength graph for 2% marble dust

From Fig 5.5 The compressive stress for 2% waste mix was found to be 264.90 kPa which has increased from the untreated soil. The increase in percentage 22.54% of the untreated soil.

Table5.7 Unconfined compression test values for 4% marble dust (Static compaction)

DEFORMATION DIAL READING	AXIAL DEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(Kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	32	10.56	91.26
100	0.1	0.00132	11.35	41	13.53	116.85
150	0.15	0.00197	11.36	49	16.17	139.56
200	0.20	0.00263	11.37	56	18.48	159.39
250	0.25	0.00329	11.38	58	19.14	164.98
300	0.30	0.00395	11.38	61	20.13	173.39
350	0.35	0.00461	11.39	65	21.45	184.64
400	0.40	0.00526	11.40	69	22.77	195.87
450	0.45	0.00592	11.41	71	23.43	201.42
500	0.50	0.00658	11.42	74	24.42	209.79
550	0.55	0.00724	11.42	76	25.08	215.32
600	0.60	0.00789	11.43	78	25.74	220.84
650	0.65	0.00855	11.44	80	26.4	226.35
700	0.70	0.00921	11.45	82	27.06	231.86
750	0.75	0.00987	11.45	83	27.39	234.53
800	0.80	0.01053	11.46	85	28.05	240.02
850	0.85	0.01118	11.47	89	29.37	251.15
900	0.90	0.01184	11.48	91	30.03	256.62
950	0.95	0.01250	11.48	95	31.35	**267.72**
1000	1.00	0.01316	11.49	91	30.03	256.28
1050	1.05	0.01382	11.50	83	27.39	233.59

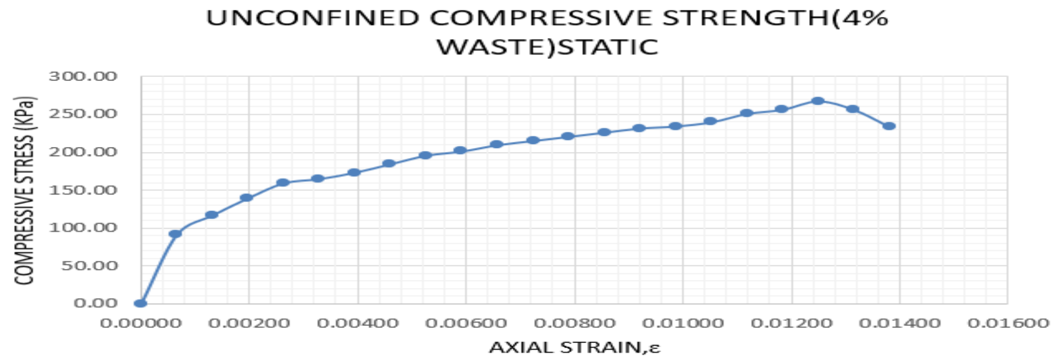


Fig 5.6 Unconfined compressive strength graph for 4% marble dust

From Fig 5.6 The compressive stress for 4% waste mix was found to be 267.72kPa which has increased from the untreated soil. The increase in percentage 2.82% of the 2% marble dust mix soil and this is the maximum compressive stress attained compared to untreated soil as well as from the waste mix.

Table5.8 Unconfined compression test values for 6% marble dust (Static compaction)

DEFORMATION DIAL READING	AXIAL DEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(Kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	26	8.58	74.15
100	0.1	0.00132	11.35	33	10.89	94.05
150	0.15	0.00197	11.36	41	13.53	116.77
200	0.20	0.00263	11.37	47	15.51	133.78
250	0.25	0.00329	11.38	56	18.48	159.29
300	0.30	0.00395	11.38	61	20.13	173.39
350	0.35	0.00461	11.39	67	22.11	190.32
400	0.40	0.00526	11.40	74	24.42	210.07
450	0.45	0.00592	11.41	79	26.07	224.11
500	0.50	0.00658	11.42	80	26.4	226.80
550	0.55	0.00724	11.42	82	27.06	232.32
600	0.60	0.00789	11.43	83	27.39	234.99
650	0.65	0.00855	11.44	84	27.72	237.67
700	0.70	0.00921	11.45	86	28.38	243.17
750	0.75	0.00987	11.45	87	28.71	245.83
800	0.80	0.01053	11.46	90	29.7	254.14
850	0.85	0.01118	11.47	92	30.36	**259.61**
900	0.90	0.01184	11.48	88	29.04	248.16
950	0.95	0.01250	11.48	76	25.08	214.18
1000	1.00	0.01316	11.49	74	24.42	208.40
1050	1.05	0.01382	11.50	71	23.43	199.82

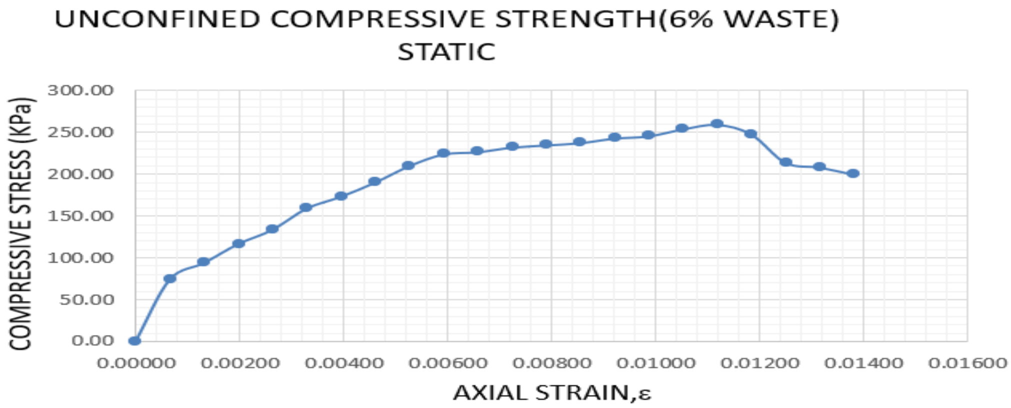


Fig 5.7 Unconfined compressive strength graph for 6% marble dust

From Fig 5.7 The compressive stress for 6% waste mix was found to be 259.61kPa. This has decreased from the 4% waste mix.

Table5.9 Unconfined compression test values for 8% marble dust (Static compaction)

DEFORMATION DIAL READING	AXIALDEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(Kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	29	9.57	82.71
100	0.1	0.00132	11.35	42	13.86	119.70
150	0.15	0.00197	11.36	49	16.17	139.56
200	0.20	0.00263	11.37	56	18.48	159.39
250	0.25	0.00329	11.38	58	19.14	164.98
300	0.30	0.00395	11.38	61	20.13	173.39
350	0.35	0.00461	11.39	63	20.79	178.96
400	0.40	0.00526	11.40	65	21.45	184.52
450	0.45	0.00592	11.41	68	22.44	192.91
500	0.50	0.00658	11.42	70	23.1	198.45
550	0.55	0.00724	11.42	73	24.09	206.82
600	0.60	0.00789	11.43	75	24.75	212.34
650	0.65	0.00855	11.44	78	25.74	220.69
700	0.70	0.00921	11.45	80	26.4	226.20
750	0.75	0.00987	11.45	82	27.06	231.70
800	0.80	0.01053	11.46	84	27.72	237.19
850	0.85	0.01118	11.47	85	28.05	239.86
900	0.90	0.01184	11.48	87	28.71	245.34
950	0.95	0.01250	11.48	89	29.37	**250.81**
1000	1.00	0.01316	11.49	85	28.05	239.38
1050	1.05	0.01382	11.50	81	26.73	227.96

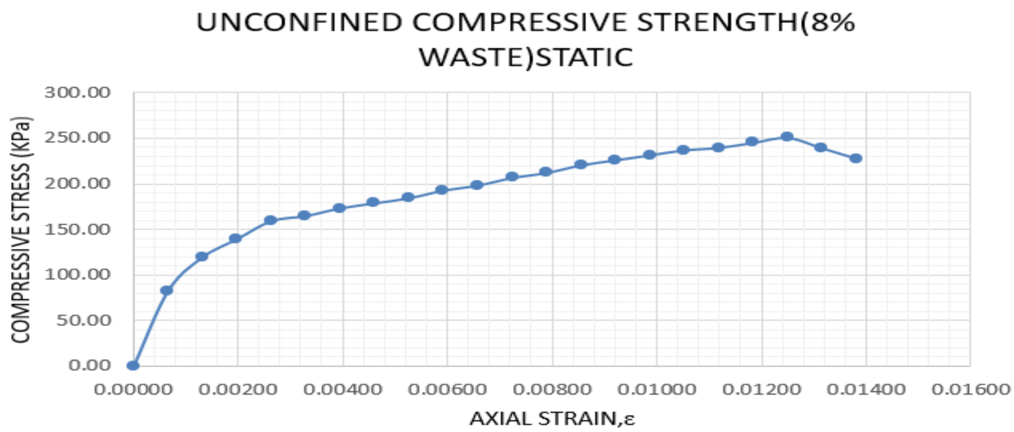


Fig 5.8 Unconfined compressive strength graph for 8% marble dust

From Fig 5.8 The compressive stress for 8% waste mix was found to be 250.81kPa. This has decreased from the 6% waste mix which is the least among the other mixes.

Table 5.10 Unconfined compressive stress of soil V/s Percentage of marble dust (Static Compaction)

Percentages of waste increment(%)	Unconfined compressive stress(kPa)
0%	242.36
2%	264.90
4%	267.72
6%	259.61
8%	250.81

Results- Maximum strength attained at 4% waste mix. This means that minimum 4% waste should be added to the soil to attain maximum compressive strength because after that the strength decreases.

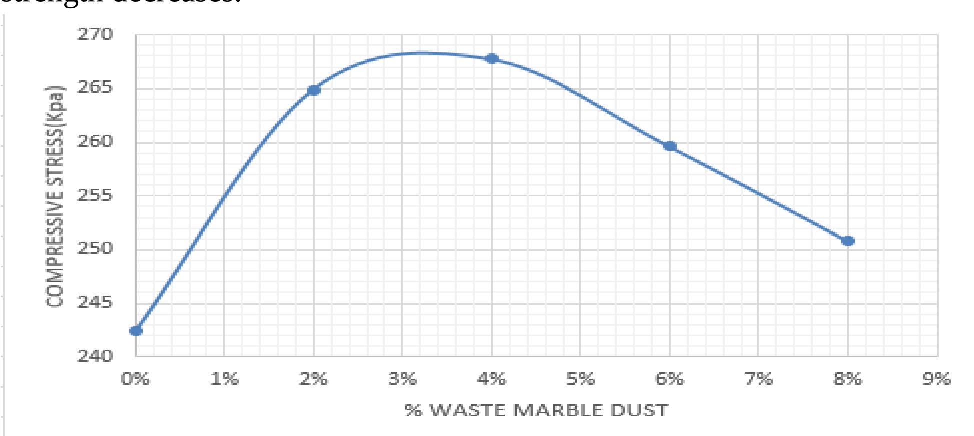


Fig 5.9 Compressive Stress v/s % Marble Dust

Maximum strength attained at 4% Waste Marble Dust .As it can be seen from the **Fig 5.9** that the compressive stress value was found to be maximum at 4% waste addition after that the value declines.

Table5.11 Unconfined compression test values for 2% marble dust (Dynamic compaction)

DEFORMATION DIAL READING	AXIAL DEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(Kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	16	5.28	45.63
100	0.1	0.00132	11.35	28	9.24	79.80
150	0.15	0.00197	11.36	38	12.54	108.23
200	0.20	0.00263	11.37	46	15.18	130.93
250	0.25	0.00329	11.38	55	18.15	156.44
300	0.30	0.00395	11.38	59	19.47	167.71
350	0.35	0.00461	11.39	65	21.45	184.64
400	0.40	0.00526	11.40	67	22.11	190.20
450	0.45	0.00592	11.41	69	22.77	195.75
500	0.50	0.00658	11.42	71	23.43	201.29
550	0.55	0.00724	11.42	73	24.09	206.82
600	0.60	0.00789	11.43	76	25.08	215.18
650	0.65	0.00855	11.44	79	26.07	223.52
700	0.70	0.00921	11.45	81	26.73	229.03
750	0.75	0.00987	11.45	83	27.39	234.53
800	0.80	0.01053	11.46	85	28.05	240.02
850	0.85	0.01118	11.47	86	28.38	242.08
900	0.90	0.01184	11.48	88	29.04	248.16
950	0.95	0.01250	11.48	93	30.69	**262.08**
1000	1.00	0.01316	11.49	89	29.37	250.65
1050	1.05	0.01382	11.50	83	27.39	233.49

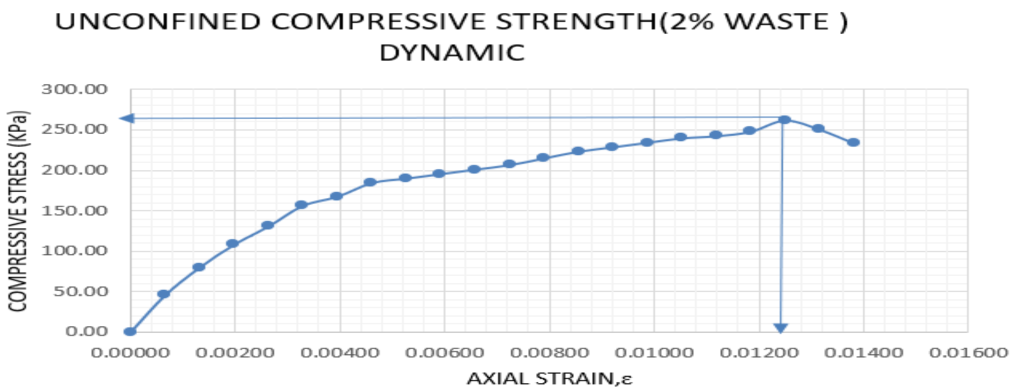


Fig 5.10 Unconfined compressive strength graph for 2% marble dust (Dynamic Compaction)

From the above **Fig 5.10** it can be seen that on dynamically compacted sample of UCS, the compressive stress was found out to be 262.08kPa. Compared to the statically compaction, dynamic compaction values for UCS was found to be less.

Table5.12 Unconfined compression test values for 4% marble dust (Dynamic compaction)

DEFORMATION DIAL READING	AXIALDEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	25	8.25	71.30
100	0.1	0.00132	11.35	28	9.24	79.80
150	0.15	0.00197	11.36	39	12.87	111.08
200	0.20	0.00263	11.37	48	15.84	136.62
250	0.25	0.00329	11.38	57	18.81	162.13
300	0.30	0.00395	11.38	59	19.47	181.80
350	0.35	0.00461	11.39	64	21.12	190.20
400	0.40	0.00526	11.40	67	22.11	192.31
450	0.45	0.00592	11.41	68	22.44	201.29
500	0.50	0.00658	11.42	71	23.43	206.82
550	0.55	0.00724	11.42	73	24.09	215.18
600	0.60	0.00789	11.43	76	25.08	223.52
650	0.65	0.00855	11.44	79	26.07	229.03
700	0.70	0.00921	11.45	81	26.73	234.53
750	0.75	0.00987	11.45	83	27.30	240.02
800	0.80	0.01053	11.46	85	28.05	245.18
850	0.85	0.01118	11.47	94	30.97	**264.89**
900	0.90	0.01184	11.48	88	29.04	248.16
950	0.95	0.01250	11.48	81	26.73	228.27

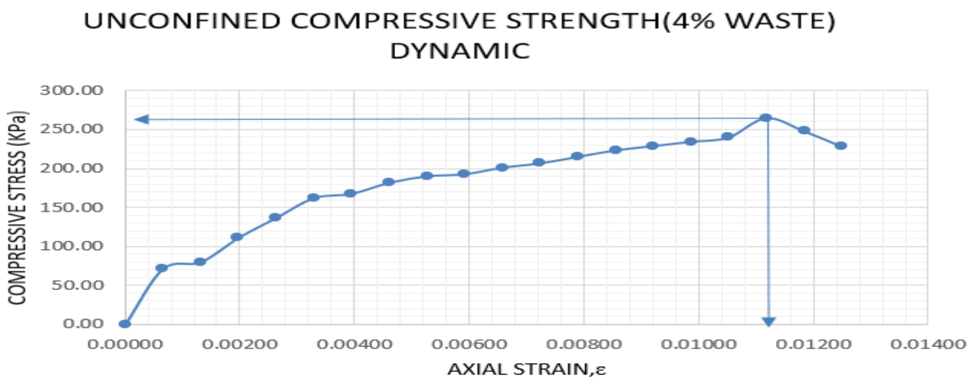


Fig 5.11 Unconfined compressive strength graph for 4% marble dust (Dynamic Compaction)

As it can be seen from Fig 5.11 for dynamically compacted sample the compressive stress of the sample was found out to be 264.90kPa.

Table5.13 Unconfined compression test values for 6% marble dust (Dynamic compaction)

DEFORMATION DIAL READING	AXIALDEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(Kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	21	6.93	59.89
100	0.1	0.00132	11.35	32	10.56	91.20
150	0.15	0.00197	11.36	40	13.2	113.93
200	0.20	0.00263	11.37	48	15.84	136.62
250	0.25	0.00329	11.38	55	18.15	156.44
300	0.30	0.00395	11.38	60	19.8	170.55
350	0.35	0.00461	11.39	67	22.11	190.32
400	0.40	0.00526	11.40	74	24.42	210.07
450	0.45	0.00592	11.41	77	25.41	218.44
500	0.50	0.00658	11.42	80	26.4	226.80
550	0.55	0.00724	11.42	82	27.06	232.32
600	0.60	0.00789	11.43	83	27.39	234.99
650	0.65	0.00855	11.44	84	27.72	237.67
700	0.70	0.00921	11.45	86	28.38	243.17
750	0.75	0.00987	11.45	87	28.71	245.83
800	0.80	0.01053	11.46	89	29.37	251.31
850	0.85	0.01118	11.47	91	30.03	**256.79**
900	0.90	0.01184	11.48	82	27.06	231.24
950	0.95	0.01250	11.48	79	26.07	222.63

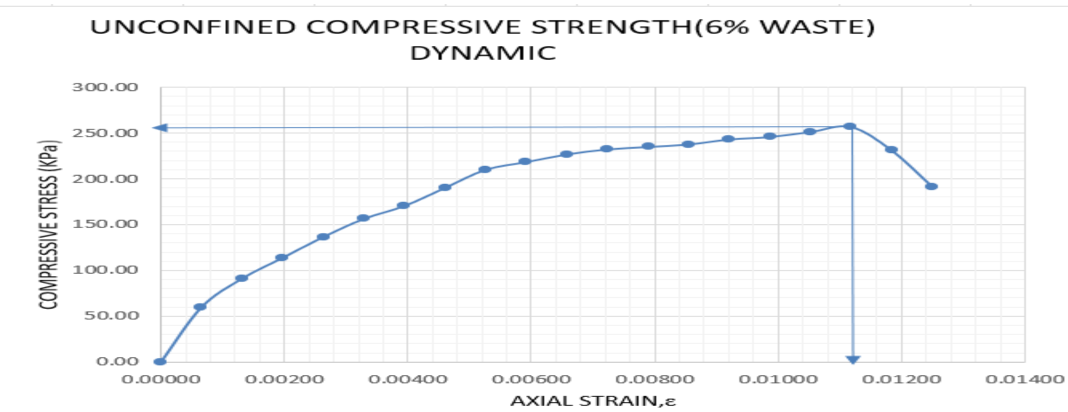


Fig 5.12 Unconfined compressive strength graph for 6% marble dust (Dynamic Compaction)

As it can be seen from Fig 5.12 for dynamically compacted sample the compressive stress of the sample was found out to be 256.79kPa which decreases from the 4% waste addition.

Table5.14 Unconfined compression test values for 8% marble dust (Dynamic compaction)

DEFORMATION DIAL READING	AXIALDEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(Kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	25	8.25	71.30
100	0.1	0.00132	11.35	28	9.24	79.80
150	0.15	0.00197	11.36	39	12.87	111.08
200	0.20	0.00263	11.37	48	15.84	136.62
250	0.25	0.00329	11.38	57	18.81	162.13
300	0.30	0.00395	11.38	59	19.47	167.71
350	0.35	0.00461	11.39	64	21.12	181.80
400	0.40	0.00526	11.40	67	22.11	190.20
450	0.45	0.00592	11.41	68	22.44	192.91
500	0.50	0.00658	11.42	71	23.43	201.29
550	0.55	0.00724	11.42	73	24.09	206.82
600	0.60	0.00789	11.43	76	25.08	215.18
650	0.65	0.00855	11.44	78	25.74	220.69
700	0.70	0.00921	11.45	79	26.07	223.37
750	0.75	0.00987	11.45	80	26.4	226.05
800	0.80	0.01053	11.46	82	27.06	231.55
850	0.85	0.01118	11.47	88	28.97	**247.76**
900	0.90	0.01184	11.48	86	28.38	242.52
950	0.95	0.01250	11.48	84	27.72	236.72

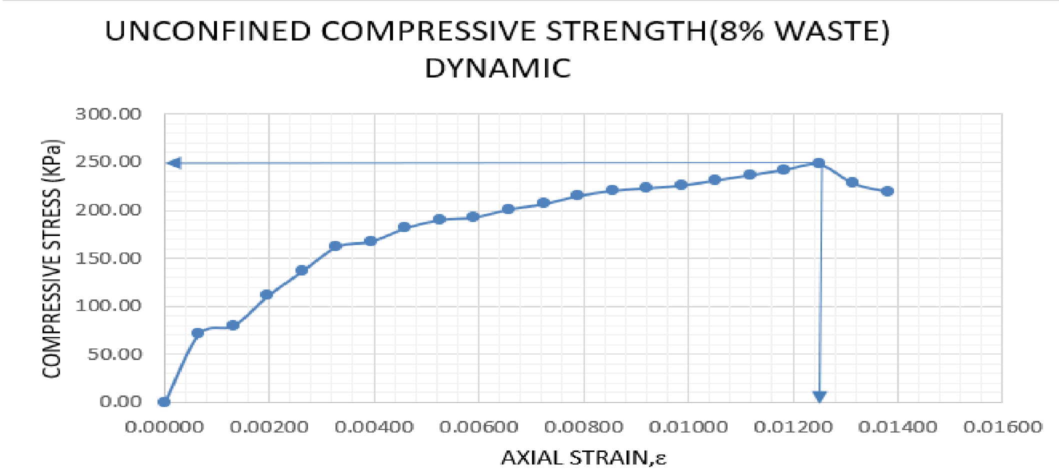


Fig 5.13 Unconfined compressive strength graph for 8% marble dust (Dynamic Compaction)

As it can be seen from Fig 5.13 for dynamically compacted sample the compressive stress of the sample was found out to be 247.99 kPa which decreases from the 6% waste addition.

Table 5.15 Load penetration data for CBR test for 2% Marble Dust (Static Compaction)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	46.3
1	68.9
1.5	87.2
2	103.5
2.5	107.1
3	117.4
4	135.6
5	146.1
7.5	160.3
10	174.3
12.5	190.5

2.5mm CBR value = 7.82%

5.0mm CBR value = 7.11%

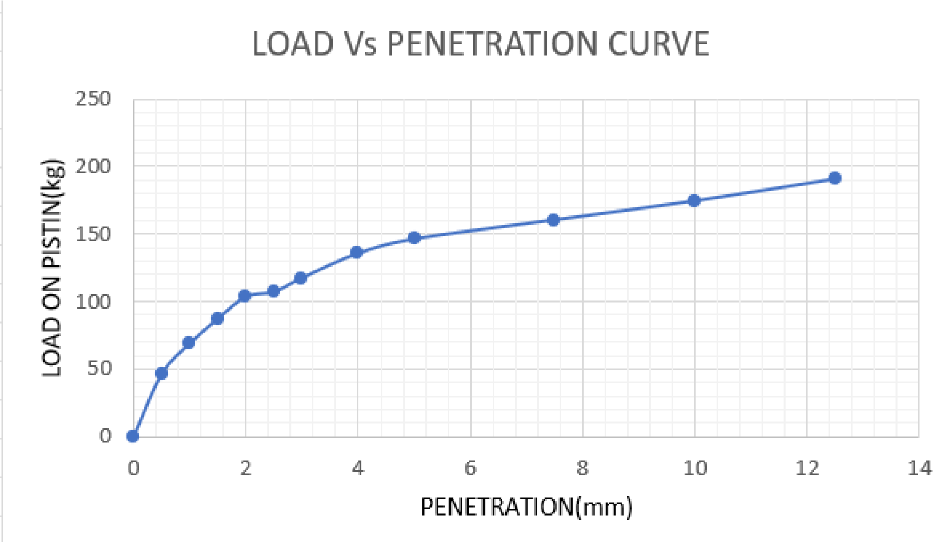


Fig 5.14 Load penetration curve for 2%Marble dust (Static Compaction)

It can be seen from the **Fig 5.14** the CBR value at 2.5mm was found to be more compared to the 5mm CBR value.

Table 5.16 Load penetration data for CBR test for 4% Marble Dust (Static Compaction)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	52.6
1	67.8
1.5	92.4
2	112.6
2.5	141.4
3	167.3
4	192.8
5	204.7
7.5	209.1
10	214.9
12.5	217.1

2.5mm CBR value = 10.32%

5.0mm CBR value = 9.96%

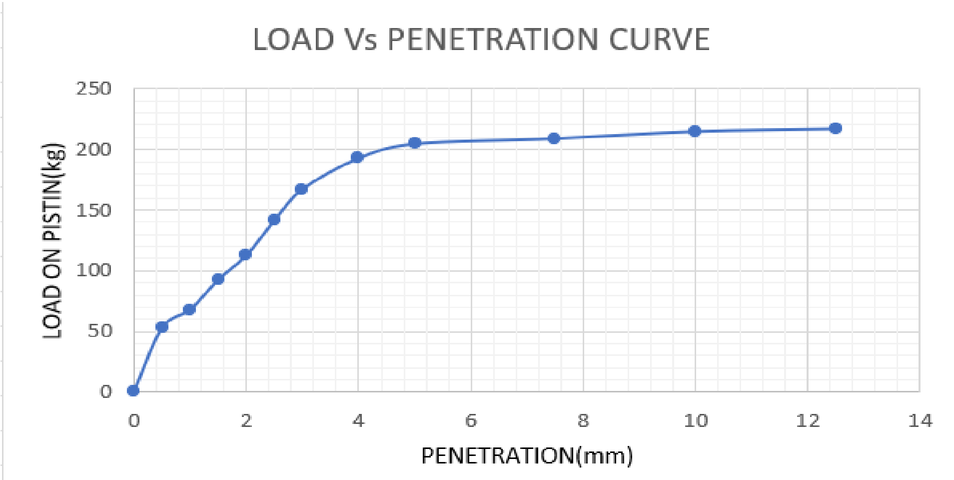


Fig 5.15 Load penetration curve for 4%Marble dust (Static Compaction)

From the above **Fig 5.15** the CBR of the soil found out to be more in case of 2.5mm penetration and it increases from 2% of addition of waste.

Table 5.17 Load penetration data for CBR test for 6% Marble Dust (Static Compaction)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	56.8
1	87.3
1.5	117.1
2	139.2
2.5	156.6
3	178.2
4	207.4
5	221.1
7.5	234.7
10	237.1
12.5	245.6

2.5mm CBR value = 11.43%

5.0mm CBR value = 10.76%

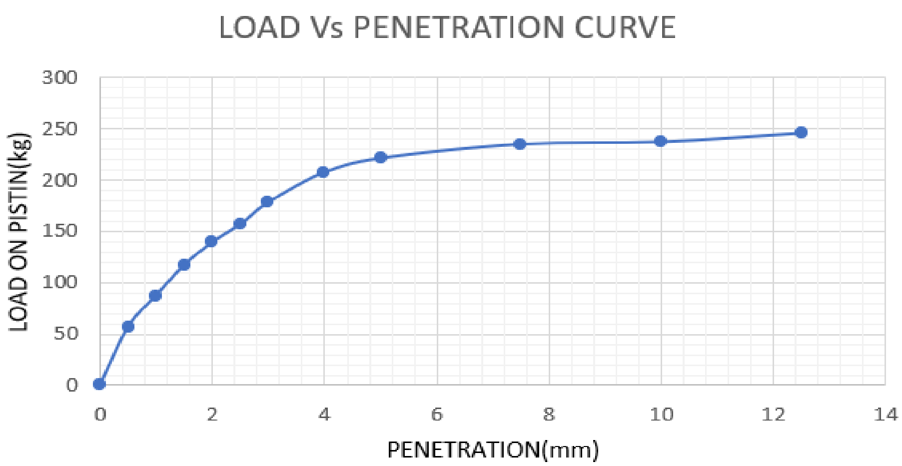


Fig 5.16 Load penetration curve for 6%Marble dust (Static Compaction)

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 4% of addition of waste.

Table 5.18 Load penetration data for CBR test for 8% Marble Dust (Static Compaction)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	58.9
1	86.3
1.5	107.2
2	121.4
2.5	147.8
3	168.2
4	191.5
5	209.8
7.5	214.3
10	219.1
12.5	227.7

2.5mm CBR value = 10.79%
5.0mm CBR value = 10.21%

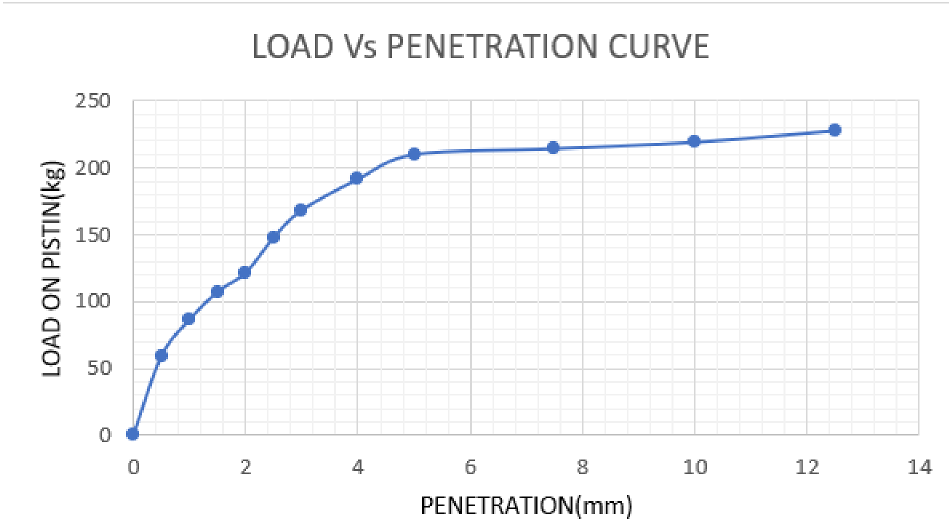


Fig 5.17 Load penetration curve for 8%Marble dust (Static Compaction)

From the above **Fig 5.17** the CBR of the soil found out to be more in case of 2.5mm penetration and it decreases from 6% of addition of waste.

Table 5.19 Load penetration data for CBR test for 2% Marble Dust (Dynamic Compaction)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	44.2
1	65.9
1.5	87.3
2	90.7
2.5	99.1
3	116.3
4	131.9
5	141.1
7.5	149.2
10	164.7
12.5	179.4

2.5mm CBR value = 7.23%
5.0mm CBR value = 6.87%

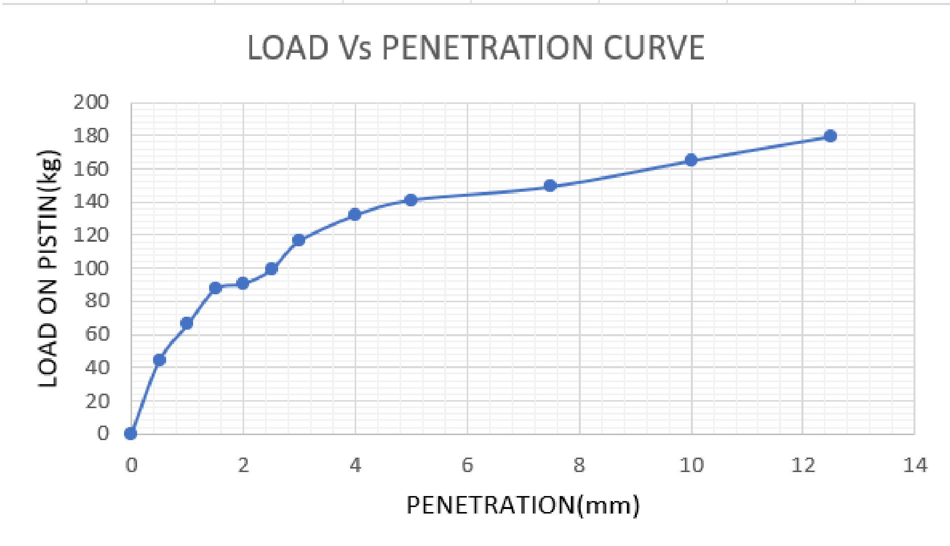


Fig 5.18 Load penetration curve for 2%Marble dust (Dynamic Compaction)

From the above **Fig 5.18** the CBR of the soil found out to be more in case of 2.5mm penetration. Compared to the statically compaction, dynamic compaction values for CBR was found to be less.

Table 5.20 Load penetration data for CBR test for 4% Marble Dust (Dynamic Compaction)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	54.7
1	76.2
1.5	91.6
2	115.2
2.5	134.9
3	156.4
4	173.8
5	189.9
7.5	191.2
10	196.3
12.5	199.2

2.5mm CBR value = 9.85%
5.0mm CBR value = 9.24%

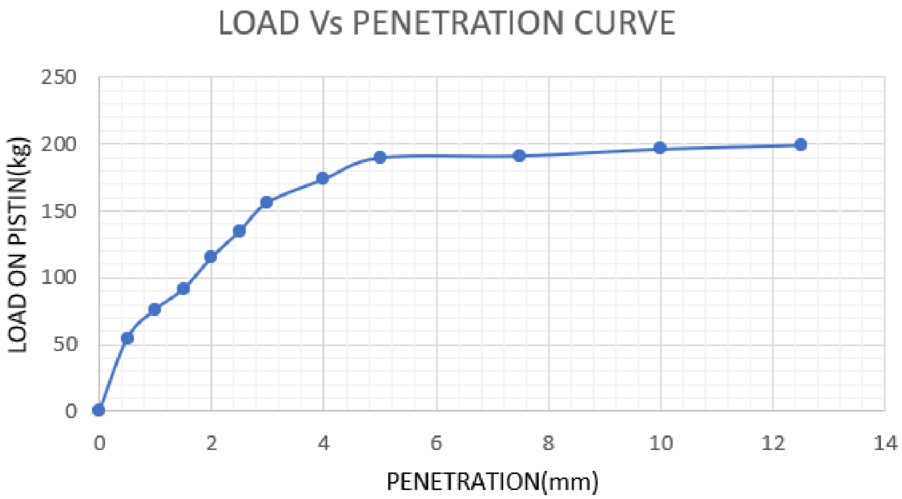


Fig 5.19 Load penetration curve for 4%Marble dust (Dynamic Compaction)

From the above **Fig 5.19** the CBR of the soil found out to be more in case of 2.5mm penetration and it increases from 2% of addition of waste.

Table 5.21 Load penetration data for CBR test for 6% Marble Dust (Dynamic Compaction)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	55.2
1	76.9
1.5	91.4
2	121.7
2.5	146.3
3	167.4
4	189.7
5	208.0
7.5	219.2
10	227.3
12.5	236.8

2.5mm CBR value = 10.68%
5.0mm CBR value = 10.12%

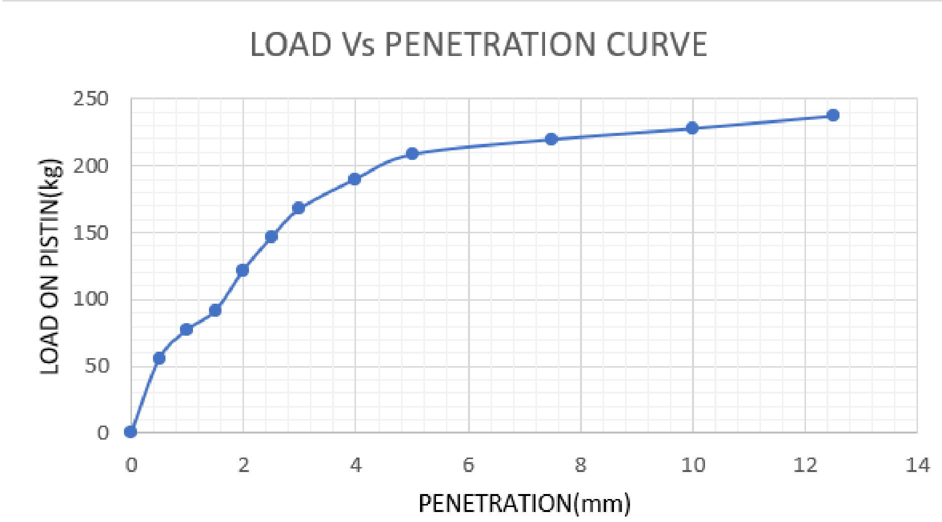


Fig 5.20 Load penetration curve for 6%Marble dust (Dynamic Compaction)

From the above **Fig 5.20** the CBR of the soil found out to be more in case of 2.5mm penetration and it increases from 4% of addition of waste.

Table 5.22 Load penetration data for CBR test for 8% Marble Dust (Dynamic Compaction)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	49.2
1	62.7
1.5	86.4
2	117.3
2.5	128.9
3	157.1
4	172.5
5	190.1
7.5	197.33
10	203.5
12.5	216.4

2.5mm CBR value = 9.41%
5.0mm CBR value = 9.25%

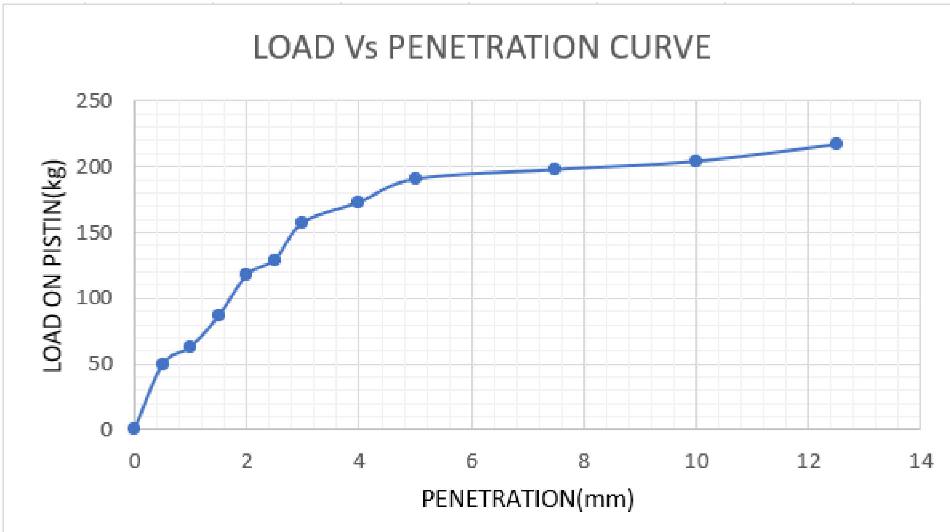


Fig 5.21 Load penetration curve for 8%Marble dust (Dynamic Compaction)

From the above **Fig 5.21** the CBR of the soil found out to be more in case of 2.5mm penetration and it decreases from 6% of addition of waste.

5.3 Test results of soil mixed with waste plastic

Table 5.23 Standard Proctor Test for 0.5% waste plastic

Empty mould +base plate=4234g

Mass of compacted soil+mould with base plate(g)	Mass of empty container(g)	Mass of container +wet soil(g)	Mass of container+dry soil(g)	Mass of water(g)	Mass of dry soil(g)	Bulk density(kN/m3)	Water content (%)	Dry density(kN/m3)
6048	8.475	21.581	20.081	1.5	11.606	18.070	12.924	16.002
6127	8.237	20.04	18.440	1.6	10.203	18.845	15.682	16.290
6543	8.615	22.529	20.413	2.116	11.798	22.926	17.935	19.439
6541	9.317	22.04	19.998	2.042	10.681	22.906	19.118	19.230
6530	9.412	20.682	18.637	2.045	9.225	22.798	22.168	18.662

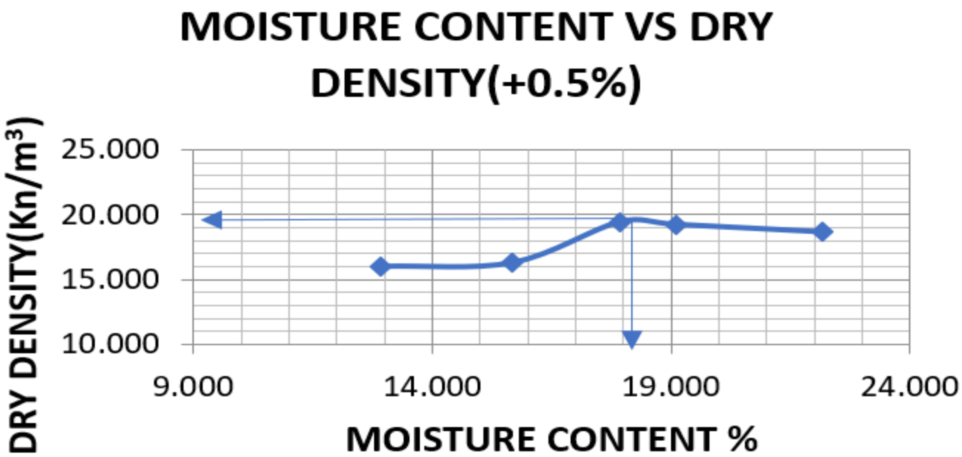


Fig 5.22 Compaction curve for 0.5% waste plastic

Here from **Fig 5.22** the moisture content of the soil while adding 0.5% waste plastic decreases to 18.2% and the MDD increases to 19.4 kN/m^3.

Table 5.24 Standard Proctor Test for 1% waste plastic

Empty mould +base plate=4234g

Mass of compacted soil+mould with base plate(g)	Mass of empty container(g)	Mass of container+wet soil(g)	Mass of container+dry soil(g)	Mass of water(g)	Mass of dry soil(g)	Bulk density (kN/m3)	Water content(%)	Dry density(kN/m3)
6216	8.914	19.479	18.396	1.083	9.482	19.718	11.422	17.697
6288	8.763	21.141	19.643	1.498	10.88	20.421	13.768	17.950
6435	8.637	20.898	19.072	1.826	10.435	21.867	17.499	18.610
6428	9.732	20.702	18.896	1.806	9.164	21.798	19.708	18.209
6410	9.846	19.958	18.110	1.738	8.264	21.621	22.362	17.670

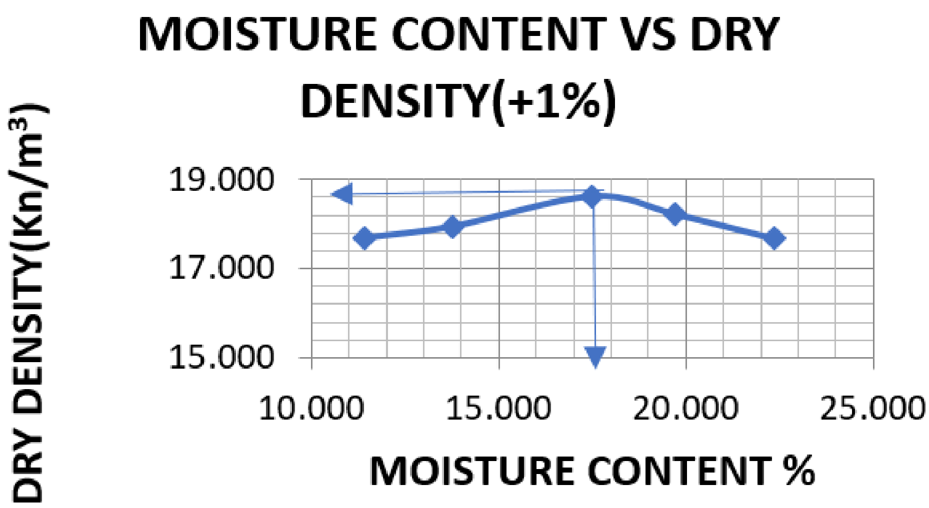


Fig 5.23 Compaction curve for 1% waste plastic

Here from **Fig 5.23** the moisture content of the soil while adding 1% waste increases to 18.4% and the MDD decreases to 18.78 kN/m^3.

Table 5.25 Standard Proctor Test for 1.5% waste plastic

Empty mould +base plate=4234g

Mass of compacted soil+mould with base plate(g)	Mass of empty container(g)	Mass of container+wet soil(g)	Mass of container+dry soil(g)	Mass of water(g)	Mass of dry soil(g)	Bulk density(kN/m3)	Water content(%)	Dry density(kN/m3)
6184	9.416	21.399	20.063	1.336	10.647	19.40	12.548	17.237
6275	8.763	19.618	18.221	1.397	9.458	20.293	14.771	17.681
6443	8.837	20.505	18.674	1.831	9.837	21.944	18.613	18.500
6439	9.721	20.947	19.131	1.816	9.410	21.906	19.299	18.362
6427	9.342	20.601	18.56	2.041	9.218	21.788	22.141	17.838

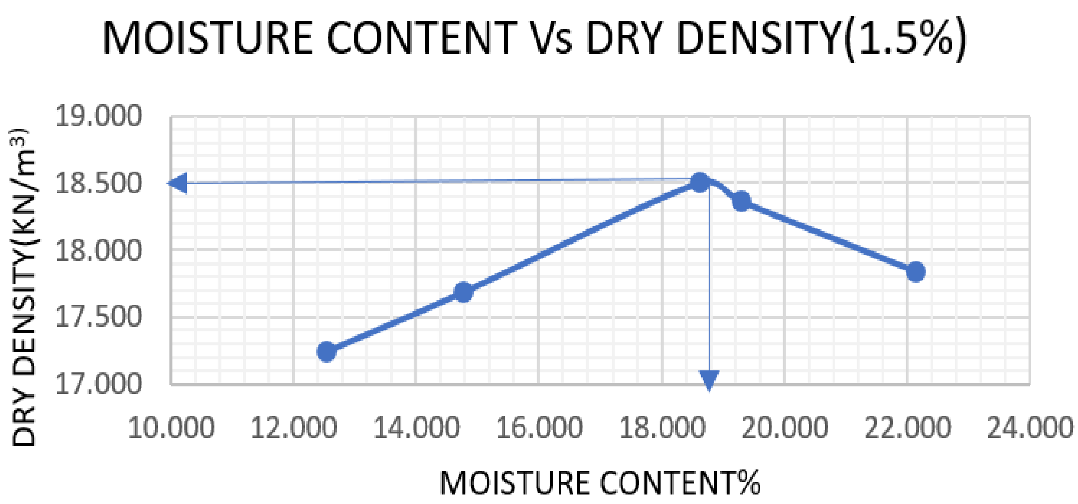


Fig 5.24 Compaction curve for 1.5% waste plastic

Here from Fig 5.24 the moisture content of the soil while adding 1.5% waste increases to 18.9% and the MDD decreases to 18.5 kN/m^3.

Table 5.26 Standard Proctor Test for 2% waste plastic

Empty mould +base plate=4234g

Mass of compacted soil+mould with base plate(g)	Mass of empty container(g)	Mass of container+wet soil(g)	Mass of container +dry soil(g)	Mass of water(g)	Mass of dry soil(g)	Bulk density(kN/m3)	Water content (%)	Dry density(kN/m3)
6156	9.416	21.399	20.063	1.336	10.647	19.133	12.548	17.00
6228	9.653	20.049	18.662	1.387	9.009	19.84	15.396	17.193
6348	9.145	20.281	18.523	1.758	9.378	21.009	18.746	17.692
6345	8.563	20.025	18.125	1.900	9.562	20.984	19.87	17.505
6341	9.145	20.717	18.61	2.107	9.465	20.944	22.261	17.131

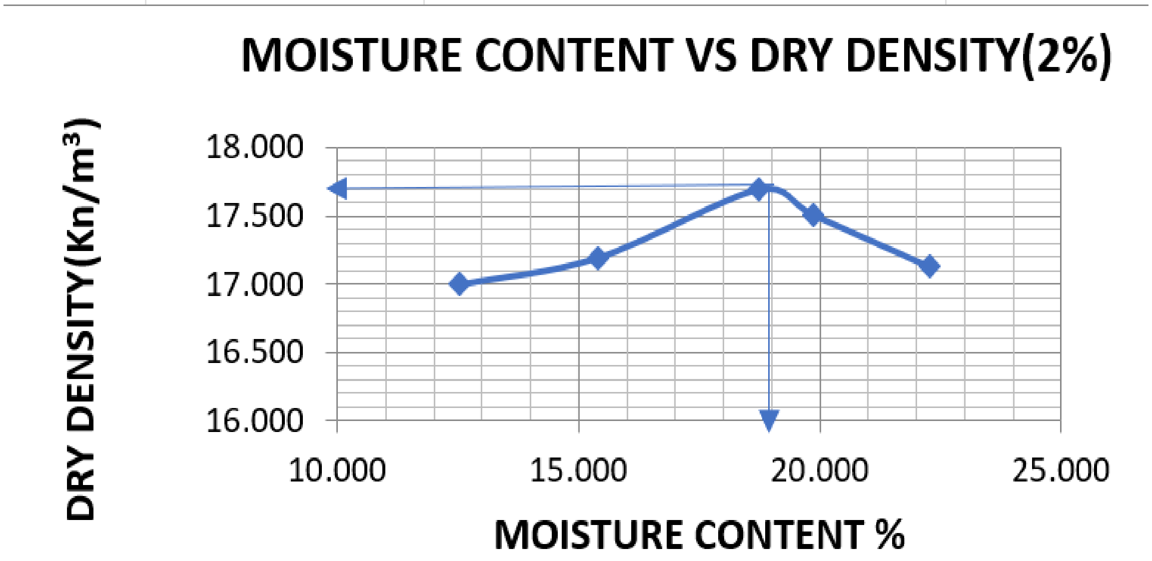


Fig 5.25 Compaction curve for 2% waste plastic

Here from Fig 5.25 the moisture content of the soil while adding 2% waste increases to 19.3% and the MDD decreases to 18.4 kN/m^3

Table 5.27 Maximum dry density V/s percentage of waste plastic

Percentages of waste increment(%)	Maximum dry density(kN/m^3)
0%	15.88
0.5%	19.4
1%	18.78
1.5%	18.5
2%	18.4

Results: Maximum dry density attained at 0.5% waste plastic

Whereas after that the Maximum dry density decreases. This makes a statement that minimum waste addition should be 0.5% because after that the dry density decreases. To attain maximum strength of the soil stabilization, minimum 0.5% is necessary.

Table5.28 Unconfined compression test values for 0.5% waste plastic (Static compaction)

DEFORMATION DIAL READING	AXIALDEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(Kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	31	10.23	88.41
100	0.1	0.00132	11.35	48	15.84	136.80
150	0.15	0.00197	11.36	58	19.14	165.19
200	0.20	0.00263	11.37	61	20.13	173.62
250	0.25	0.00329	11.38	63	20.79	179.20
300	0.30	0.00395	11.38	65	21.45	184.76
350	0.35	0.00461	11.39	68	22.44	193.16
400	0.40	0.00526	11.40	69	22.77	195.87
450	0.45	0.00592	11.41	71	23.43	201.42
500	0.50	0.00658	11.42	72	23.76	204.12
550	0.55	0.00724	11.42	73	24.09	206.82
600	0.60	0.00789	11.43	75	24.75	212.34
650	0.65	0.00855	11.44	76	25.08	215.03
700	0.70	0.00921	11.45	78	25.74	220.55
750	0.75	0.00987	11.45	80	26.4	226.05
800	0.80	0.01053	11.46	82	27.06	231.55
850	0.85	0.01118	11.47	83	27.39	234.22
900	0.90	0.01184	11.48	85	28.05	239.70
950	0.95	0.01250	11.48	87	28.71	**245.18**
1000	1.00	0.01316	11.49	85	28.05	239.38
1050	1.05	0.01382	11.50	79	26.07	222.33

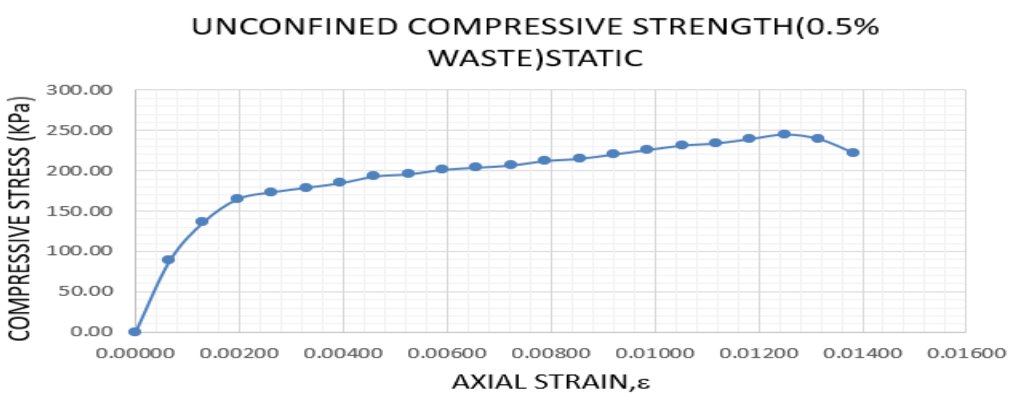


Fig 5.26 Unconfined compressive strength graph for 0.5% waste plastic(Static Compaction)

From Fig 5.26 The compressive stress for 0.5% waste mix was found to be 245.18 kPa which has increased from the untreated soil. The increase in percentage 2.82% of the untreated soil

Table5.29 Unconfined compression test values for 0.5% waste plastic (Dynamic compaction)

DEFORMATION DIAL READING	AXIALDEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(Kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	16	5.28	45.63
100	0.1	0.00132	11.35	28	9.24	79.80
150	0.15	0.00197	11.36	38	12.54	108.23
200	0.20	0.00263	11.37	46	15.18	130.93
250	0.25	0.00329	11.38	55	18.15	156.44
300	0.30	0.00395	11.38	59	19.47	167.71
350	0.35	0.00461	11.39	65	21.45	184.64
400	0.40	0.00526	11.40	67	22.11	190.20
450	0.45	0.00592	11.41	69	22.77	195.75
500	0.50	0.00658	11.42	70	23.1	198.45
550	0.55	0.00724	11.42	73	24.09	206.82
600	0.60	0.00789	11.43	74	24.42	209.51
650	0.65	0.00855	11.44	76	25.08	215.03
700	0.70	0.00921	11.45	78	25.74	220.55
750	0.75	0.00987	11.45	79	26.07	223.22
800	0.80	0.01053	11.46	81	26.73	228.72
850	0.85	0.01118	11.47	84	27.72	**237.04**
900	0.90	0.01184	11.48	83	27.39	234.06
950	0.95	0.01250	11.48	81	26.73	228.27

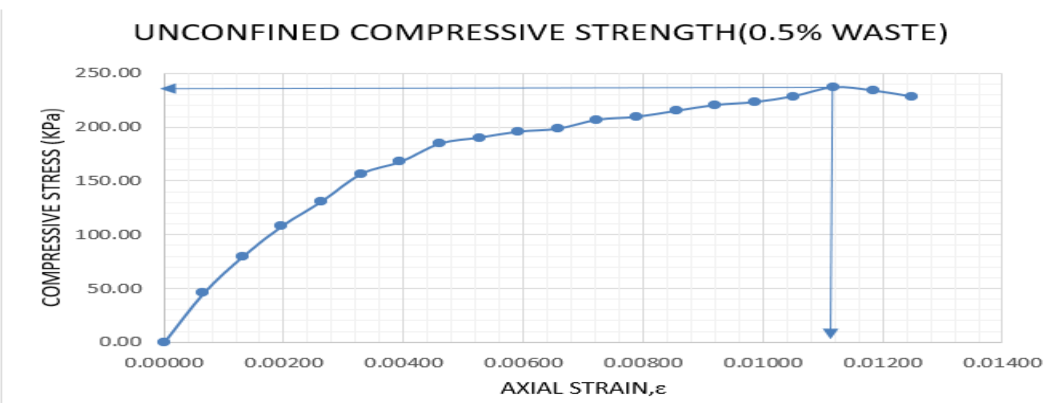


Fig 5.27 Unconfined compressive strength graph for 0.5% waste plastic(dynamic Compaction)

From Fig 5.27 The compressive stress for 0.5% waste mix was found to be 237.04 kPa which has increased from the untreated soil. Compared to statically compaction, dynamically compaction of UCS for 0.5% waste is less.

Table5.30 Unconfined compression test values for 1% waste plastic (Static compaction)

DEFORMATION DIAL READING	AXIALDEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	32	10.56	91.26
100	0.1	0.00132	11.35	41	13.53	116.85
150	0.15	0.00197	11.36	49	16.17	139.56
200	0.20	0.00263	11.37	56	18.48	159.39
250	0.25	0.00329	11.38	58	19.14	164.98
300	0.30	0.00395	11.38	61	20.13	173.39
350	0.35	0.00461	11.39	65	21.45	184.64
400	0.40	0.00526	11.40	69	22.77	195.87
450	0.45	0.00592	11.41	71	23.43	201.42
500	0.50	0.00658	11.42	74	24.42	209.79
550	0.55	0.00724	11.42	76	25.08	215.32
600	0.60	0.00789	11.43	78	25.74	220.84
650	0.65	0.00855	11.44	80	26.4	226.35
700	0.70	0.00921	11.45	81	26.73	229.03
750	0.75	0.00987	11.45	82	27.06	231.70
800	0.80	0.01053	11.46	86	28.38	242.84
850	0.85	0.01118	11.47	90	29.7	253.97
900	0.90	0.01184	11.48	92	30.36	259.44
950	0.95	0.01250	11.48	93	30.69	**262.08**
1000	1.00	0.01316	11.49	91	30.03	256.28
1050	1.05	0.01382	11.50	86	28.38	242.03

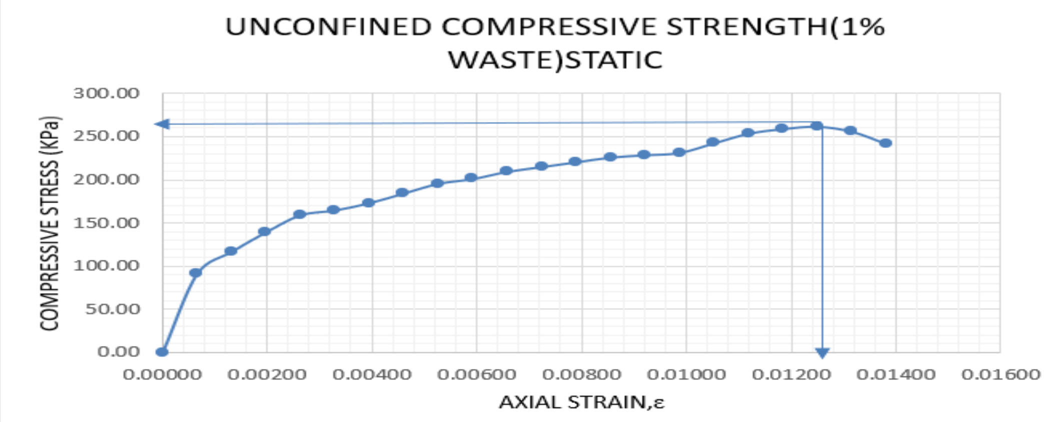


Fig 5.28 Unconfined compressive strength graph for 1% waste plastic(Static Compaction)

From Fig 5.28 The compressive stress for 1% waste mix was found to be 262.08 kPa which has increased from 0.5% waste mix. The increase in percentage of the 1% waste plastic mix soil is the maximum compressive stress attained compared to untreated soil as well as from the waste mix.

Table5.31 Unconfined compression test values for 1% waste plastic (Dynamic compaction)

DEFORMATION DIAL READING	AXIALDEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(Kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	32	10.56	91.26
100	0.1	0.00132	11.35	41	13.53	116.85
150	0.15	0.00197	11.36	49	16.17	139.56
200	0.20	0.00263	11.37	56	18.48	159.39
250	0.25	0.00329	11.38	58	19.14	164.98
300	0.30	0.00395	11.38	61	20.13	173.39
350	0.35	0.00461	11.39	65	21.45	184.64
400	0.40	0.00526	11.40	69	22.77	195.87
450	0.45	0.00592	11.41	71	23.43	201.42
500	0.50	0.00658	11.42	74	24.42	209.79
550	0.55	0.00724	11.42	76	25.08	215.32
600	0.60	0.00789	11.43	78	25.74	220.84
650	0.65	0.00855	11.44	80	26.4	226.35
700	0.70	0.00921	11.45	81	26.73	229.03
750	0.75	0.00987	11.45	82	27.06	231.70
800	0.80	0.01053	11.46	83	27.39	234.37
850	0.85	0.01118	11.47	91	30.03	**256.79**
900	0.90	0.01184	11.48	89	29.37	250.08
950	0.95	0.01250	11.48	87	28.71	245.18

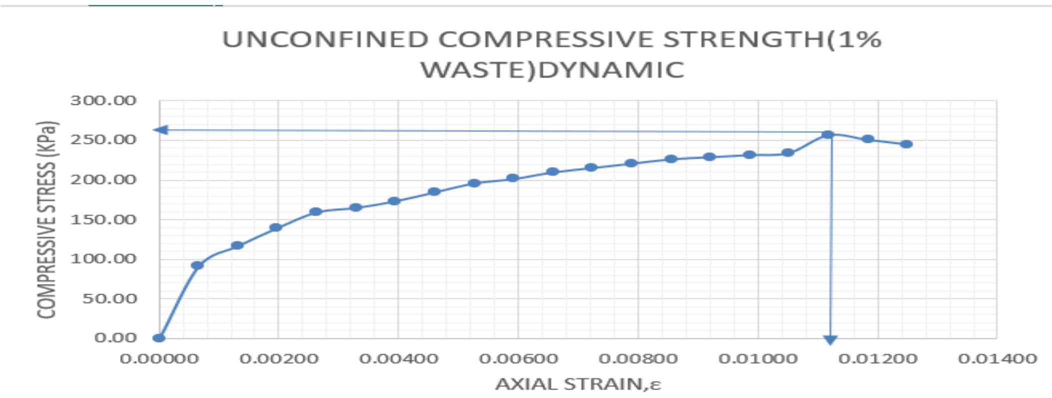


Fig 5.29 Unconfined compressive strength graph for 1% waste plastic(Dynamic Compaction)

From Fig 5.29 The compressive stress for 1% waste mix, dynamic compaction was found to be 256.79kPa which has increased from 0.5% waste mix.

Table5.32 Unconfined compression test values for 1.5% waste plastic (Static compaction)

DEFORMATION DIAL READING	AXIALDEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(Kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	29	9.57	82.71
100	0.1	0.00132	11.35	42	13.86	119.70
150	0.15	0.00197	11.36	49	16.17	139.56
200	0.20	0.00263	11.37	56	18.48	159.39
250	0.25	0.00329	11.38	58	19.14	164.98
300	0.30	0.00395	11.38	61	20.13	173.39
350	0.35	0.00461	11.39	63	20.79	178.96
400	0.40	0.00526	11.40	65	21.45	184.52
450	0.45	0.00592	11.41	68	22.44	192.91
500	0.50	0.00658	11.42	70	23.1	198.45
550	0.55	0.00724	11.42	73	24.09	206.82
600	0.60	0.00789	11.43	74	24.42	209.51
650	0.65	0.00855	11.44	75	24.75	212.20
700	0.70	0.00921	11.45	76	25.08	214.89
750	0.75	0.00987	11.45	78	25.74	220.40
800	0.80	0.01053	11.46	80	26.4	225.90
850	0.85	0.01118	11.47	84	27.72	237.04
900	0.90	0.01184	11.48	86	28.38	242.52
950	0.95	0.01250	11.48	88	29.19	**249.35**
1000	1.00	0.01316	11.49	83	27.39	233.75
1050	1.05	0.01382	11.50	81	26.73	227.96

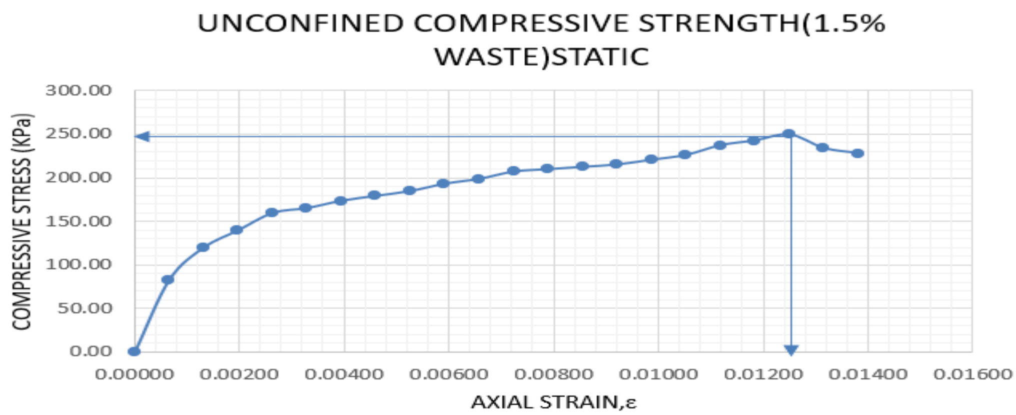


Fig 5.30 Unconfined compressive strength graph for 1.5% waste plastic(Static Compaction)

From Fig 5.30 The compressive stress for 1.5% waste mix, Static compaction was found to be 262.08 kPa which has decreased from 1% waste mix.

Table5.33 Unconfined compression test values for 1.5% waste plastic (Dynamic compaction)

DEFORMATION DIAL READING	AXIALDEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(Kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	25	8.25	71.30
100	0.1	0.00132	11.35	28	9.24	79.80
150	0.15	0.00197	11.36	39	12.87	111.08
200	0.20	0.00263	11.37	48	15.84	136.62
250	0.25	0.00329	11.38	57	18.81	162.13
300	0.30	0.00395	11.38	59	19.47	167.71
350	0.35	0.00461	11.39	61	20.13	173.28
400	0.40	0.00526	11.40	62	20.46	176.00
450	0.45	0.00592	11.41	64	21.12	181.56
500	0.50	0.00658	11.42	65	21.45	184.28
550	0.55	0.00724	11.42	67	22.11	189.82
600	0.60	0.00789	11.43	69	22.77	195.36
650	0.65	0.00855	11.44	74	24.42	209.37
700	0.70	0.00921	11.45	76	25.08	214.89
750	0.75	0.00987	11.45	79	26.07	223.22
800	0.80	0.01053	11.46	87	28.71	**245.67**
850	0.85	0.01118	11.47	85	28.05	239.86
900	0.90	0.01184	11.48	82	27.06	231.24
950	0.95	0.01250	11.48	78	25.74	219.81

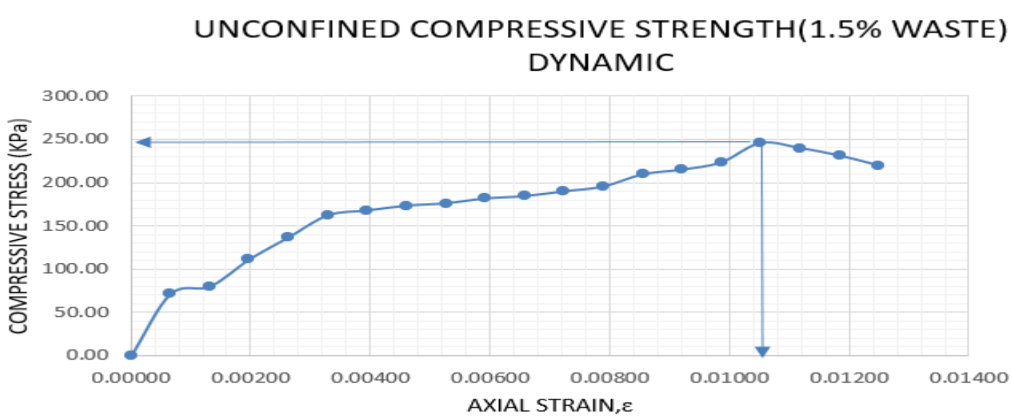


Fig 5.31 Unconfined compressive strength graph for 1.5% waste plastic (Dynamic Compaction)

From Fig 5.31 The compressive stress for 1.5% waste mix, Dynamic compaction was found to be 245.67 kPa which has decreased from 1% waste mix.

Table5.34 Unconfined compression test values for 2% waste plastic (Static compaction)

DEFORMATION DIAL READING	AXIALDEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(Kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	24	7.92	68.45
100	0.1	0.00132	11.35	31	10.23	88.35
150	0.15	0.00197	11.36	35	11.55	99.69
200	0.20	0.00263	11.37	38	12.54	108.16
250	0.25	0.00329	11.38	41	13.53	116.62
300	0.30	0.00395	11.38	45	14.85	127.91
350	0.35	0.00461	11.39	48	15.84	136.35
400	0.40	0.00526	11.40	51	16.83	144.78
450	0.45	0.00592	11.41	55	18.15	156.03
500	0.50	0.00658	11.42	58	19.14	164.43
550	0.55	0.00724	11.42	61	20.13	172.82
600	0.60	0.00789	11.43	65	21.45	184.03
650	0.65	0.00855	11.44	68	22.44	192.40
700	0.70	0.00921	11.45	70	23.1	197.93
750	0.75	0.00987	11.45	77	25.41	217.57
800	0.80	0.01053	11.46	81	26.73	228.72
850	0.85	0.01118	11.47	84	27.72	237.04
900	0.90	0.01184	11.48	86	28.38	242.52
950	0.95	0.01250	11.48	88	29.04	**247.99**
1000	1.00	0.01316	11.49	85	28.05	239.38
1050	1.05	0.01382	11.50	81	26.73	227.96

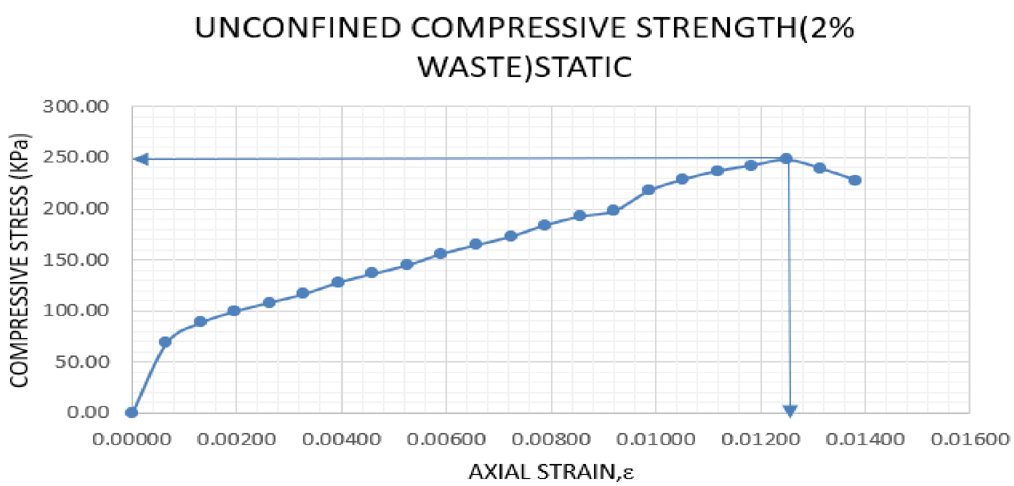


Fig 5.32 Unconfined compressive strength graph for 2% waste plastic(Static Compaction)

From Fig 5.32 The compressive stress for 2% waste mix, Static compaction was found to be 247.99 kPa which has decreased from 1.5% waste mix.

Table5.35 Unconfined compression test values for 2% waste plastic (Dynamic compaction)

DEFORMATION DIAL READING	AXIALDEFORMATION (cm)	AXIAL STRAIN ε	CORRECTED AREA(cm^2)	PROVING RING DIAL READING	AXIAL FORCE(Kg)	COMPRESSIVE STRESS(kPa)
0	0	0.00	11.34	0	0	0
50	0.05	0.00066	11.35	26	8.58	74.15
100	0.1	0.00132	11.35	31	10.23	88.35
150	0.15	0.00197	11.36	35	11.55	99.69
200	0.20	0.00263	11.37	38	12.54	108.16
250	0.25	0.00329	11.38	40	13.2	113.78
300	0.30	0.00395	11.38	42	13.86	119.39
350	0.35	0.00461	11.39	46	15.18	130.67
400	0.40	0.00526	11.40	48	15.84	136.26
450	0.45	0.00592	11.41	52	17.16	147.52
500	0.50	0.00658	11.42	54	17.82	153.09
550	0.55	0.00724	11.42	57	18.81	161.49
600	0.60	0.00789	11.43	65	21.45	184.03
650	0.65	0.00855	11.44	71	23.43	200.89
700	0.70	0.00921	11.45	73	24.09	206.41
750	0.75	0.00987	11.45	74	24.42	209.10
800	0.80	0.01053	11.46	86	28.38	**244.56**
850	0.85	0.01118	11.47	84	27.72	237.04
900	0.90	0.01184	11.48	82	27.06	231.24
950	0.95	0.01250	11.48	79	26.07	222.63

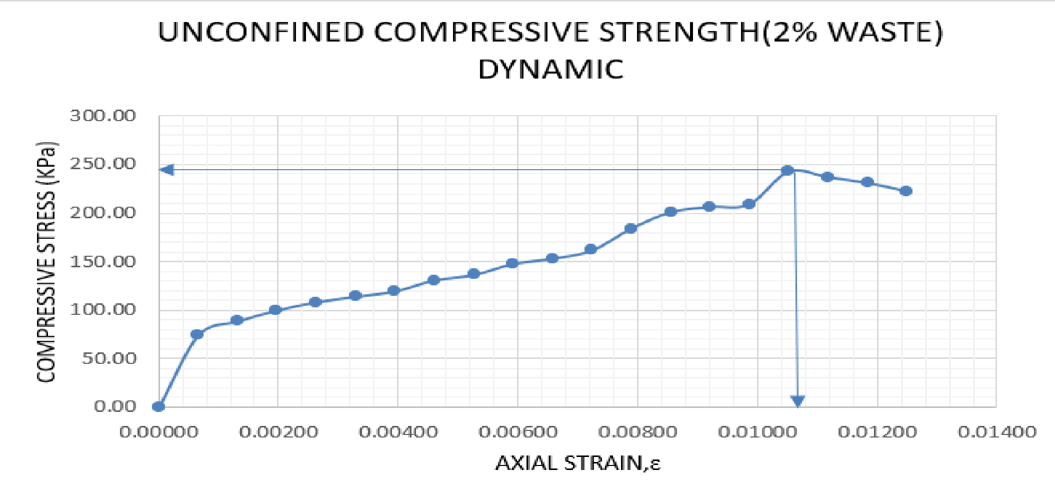


Fig 5.33 Unconfined compressive strength graph for 2% waste plastic(Dynamic Compaction)

From Fig 5.33 The compressive stress for 2% waste mix, Dynamic compaction was found to be 244.56 kPa which has decreased from 1.5% waste mix.

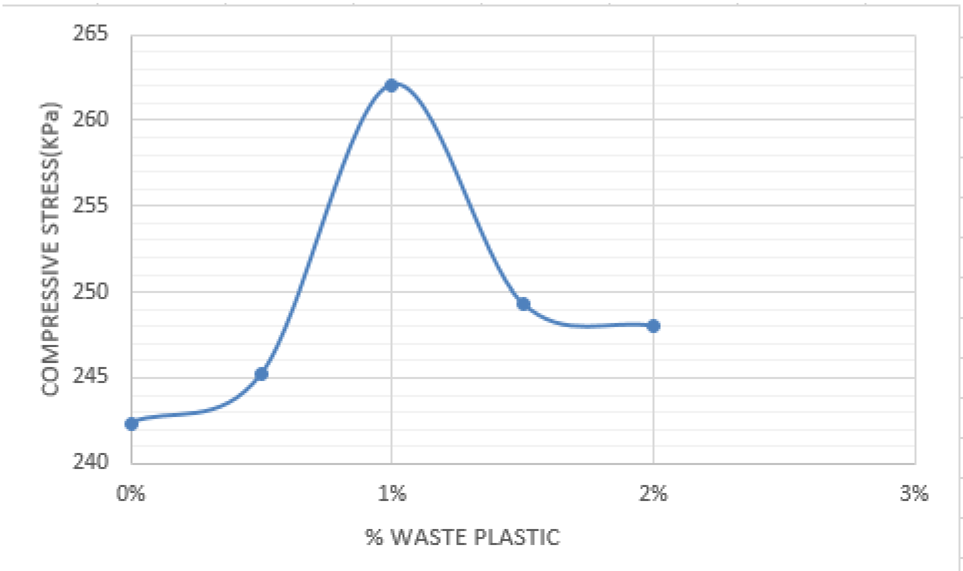


Fig 5.34 Compressive stress v/s % Waste plastic

From **Fig 5.34**, it can be seen that the compressive stress was found out to be maximum on 1% addition of waste plastic whereas after that the compressive stress decreases. So, optimum of 1% addition of waste plastic is necessary for attaining maximum strength.

Table 5.36 Load penetration data for CBR test of Untreated soil (Soaked)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	21.3
1	26.5
1.5	29.7
2	33.1
2.5	36.3
3	39.5
4	41.7
5	44.6
7.5	51.3
10	58.7
12.5	65.8

2.5mm CBR value = 2.65%
5.0mm CBR value = 2.17%

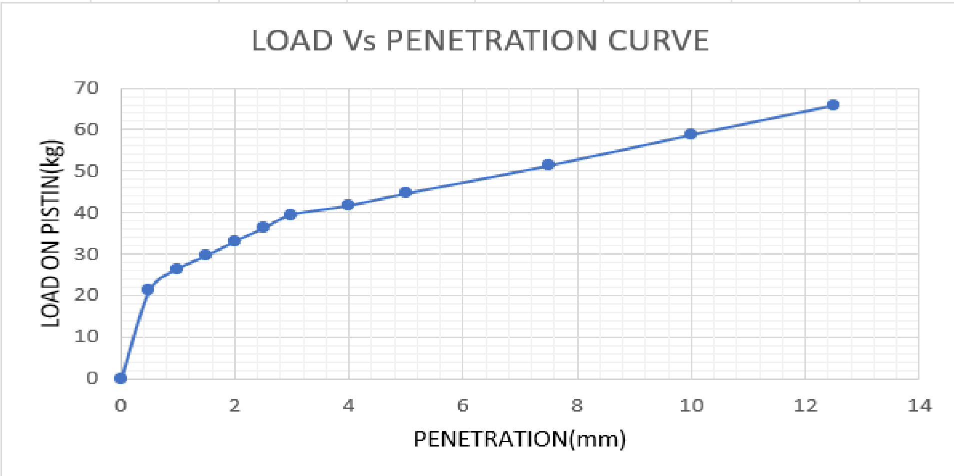


Fig 5.35 Load penetration curve for Untreated soil (Soaked)

It can be seen from the **Fig 5.35** the CBR value at 2.5mm was found to be more compared to the 5mm CBR value. Compared to Unsoaked CBR, value of Soaked CBR found out to be less.

Table 5.37 Load penetration data for CBR test for 0.5% Waste plastic (Unsoaked)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	24.3
1	36.5
1.5	52.3
2	62.7
2.5	69.3
3	76.5
4	87.1
5	99.3
7.5	115.2
10	121.5
12.5	127.3

2.5mm CBR value = 5.06%
5.0mm CBR value = 4.83%

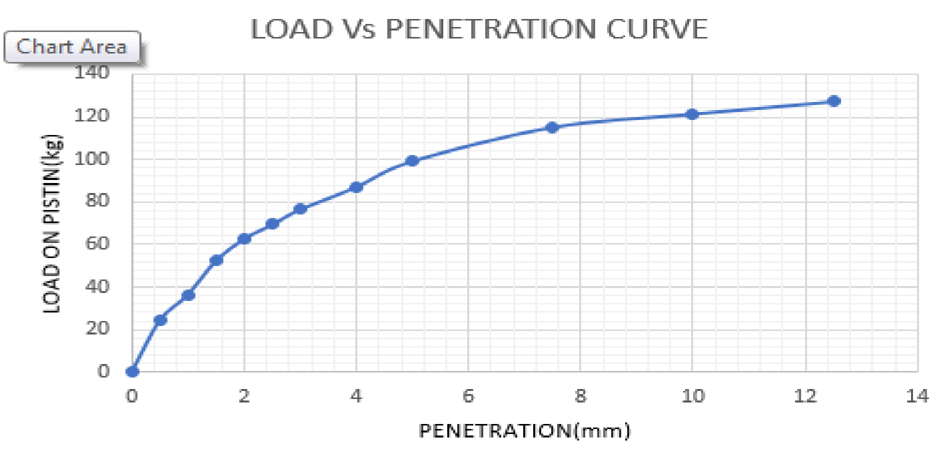


Fig 5.36 Load penetration curve for Treated soil with 0.5% Waste Plastic(Unsoaked)

It can be seen from the **Fig 5.36** the CBR value at 2.5mm was found to be more compared to the 5mm CBR value and it increases from untreated soil mix.

Table 5.38 Load penetration data for CBR test for 0.5% Waste plastic (Soaked)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	24.3
1	36.5
1.5	37.3
2	39.7
2.5	42.6
3	53.1
4	57.6
5	61.0
7.5	75.2
10	82.5
12.5	87.3

2.5mm CBR value = 3.11%

5.0mm CBR value = 2.98%

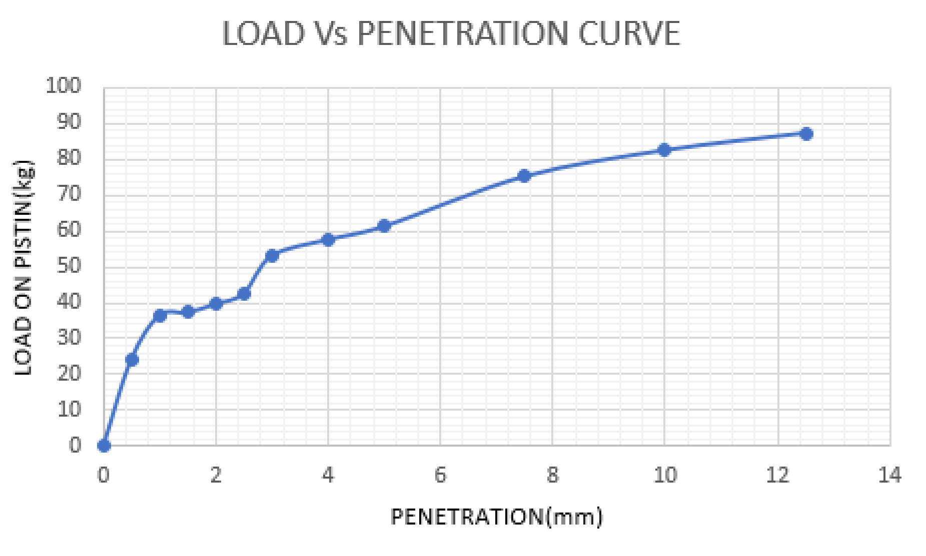


Fig 5.37 Load penetration curve for Treated soil with 0.5% Waste Plastic(Soaked)

It can be seen from the **Fig 5.37** the CBR value at 2.5mm was found to be more compared to the 5mm CBR value. Compared to 0.5% waste plastic Unsoaked CBR, the value of 0.5% waste plastic Soaked CBR found out to be less.

Table 5.39 Load penetration data for CBR test for 1% Waste plastic (Unsoaked)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	21.3
1	32.5
1.5	44.3
2	62.7
2.5	74.9
3	82.5
4	94.1
5	105.0
7.5	112.7
10	115.5
12.5	119.3

2.5mm CBR value = 5.47%

5.0mm CBR value = 5.11%

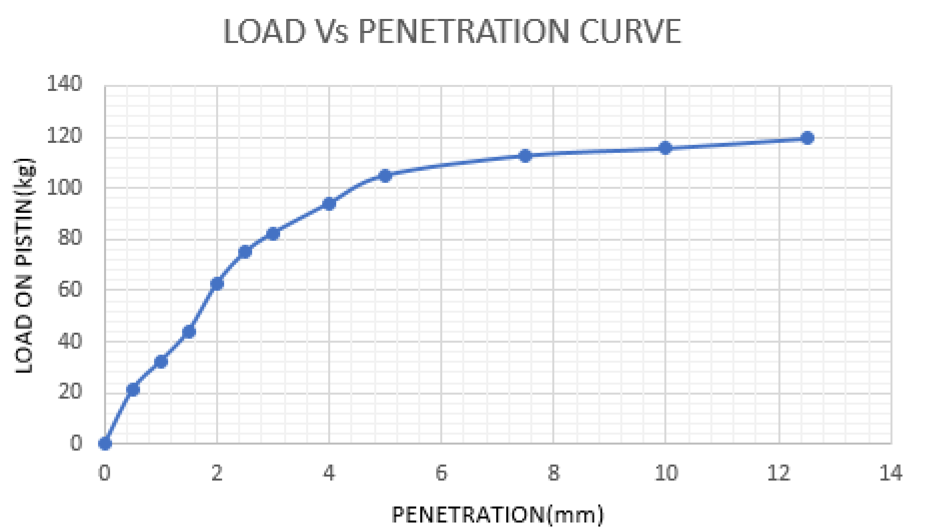


Fig 5.38 Load penetration curve for Treated soil with 1% Waste Plastic(Unsoaked)

It can be seen from the **Fig 5.38** the CBR value at 2.5mm was found to be more compared to the 5mm CBR value and it increases from 0.5% waste plastic mix.

Table 5.40 Load penetration data for CBR test for 1% Waste plastic (Soaked)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	25.3
1	29.6
1.5	41.5
2	47.2
2.5	50.0
3	56.2
4	58.4
5	63.5
7.5	74.2
10	79.5
12.5	83.1

2.5mm CBR value = 3.65%
5.0mm CBR value = 3.09%

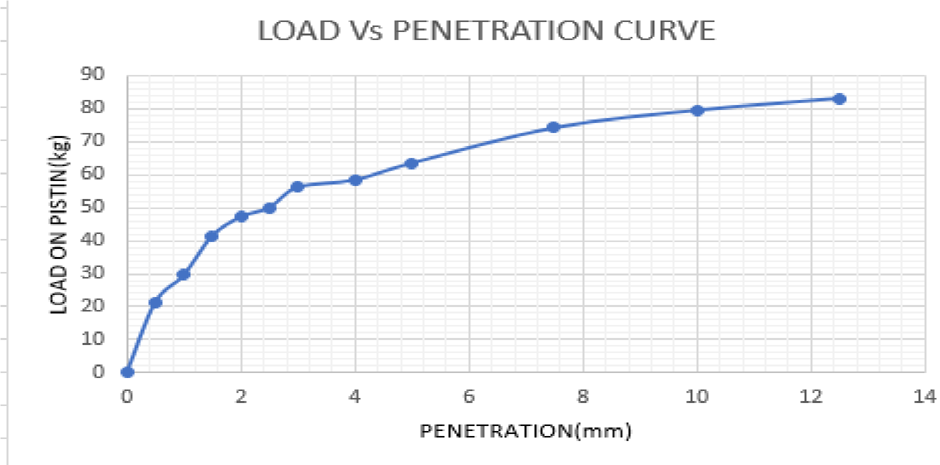


Fig 5.39 Load penetration curve for Treated soil with 1% Waste Plastic(Soaked)

It can be seen from the **Fig 5.39** the CBR value at 2.5mm was found to be more compared to the 5mm CBR value and it increases from 0.5% waste mix. Compared to 1% waste plastic Unsoaked CBR, the value of 1% waste plastic Soaked CBR found out to be less.

Table 5.41 Load penetration data for CBR test for 1.5% Waste plastic (Unsoaked)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	21.3
1	42.5
1.5	54.3
2	65.7
2.5	79.6
3	84.5
4	97.1
5	115.0
7.5	120.3
10	125.4
12.5	127.3

2.5mm CBR value = 5.81%
5.0mm CBR value = 5.63%

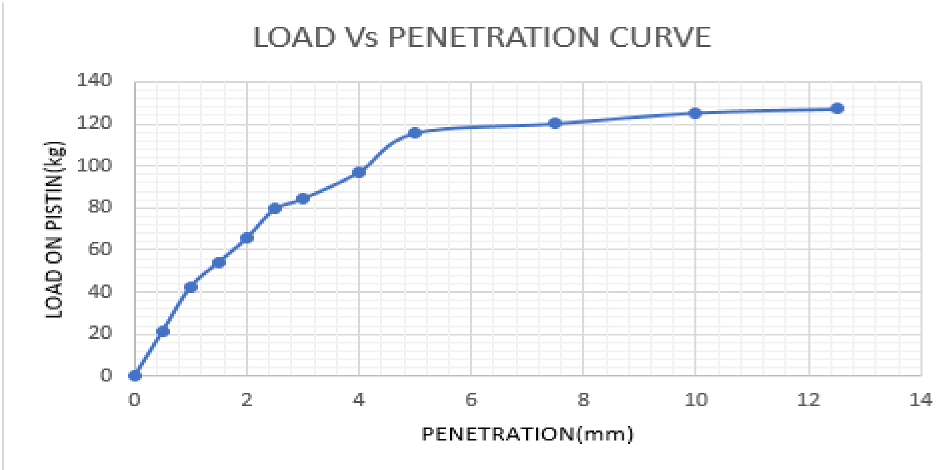


Fig 5.40 Load penetration curve for Treated soil with 1.5% Waste Plastic(Unsoaked)

It can be seen from the **Fig 5.40** the CBR value at 2.5mm was found to be more compared to the 5mm CBR value and it increases from 1% waste mix.

Table 5.42 Load penetration data for CBR test for 1.5% Waste plastic (Soaked)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	21.3
1	22.6
1.5	41.5
2	47.2
2.5	56.4
3	63.5
4	74.1
5	81.2
7.5	90.7
10	95.2
12.5	102.5

2.5mm CBR value = 4.12%
5.0mm CBR value = 3.95%

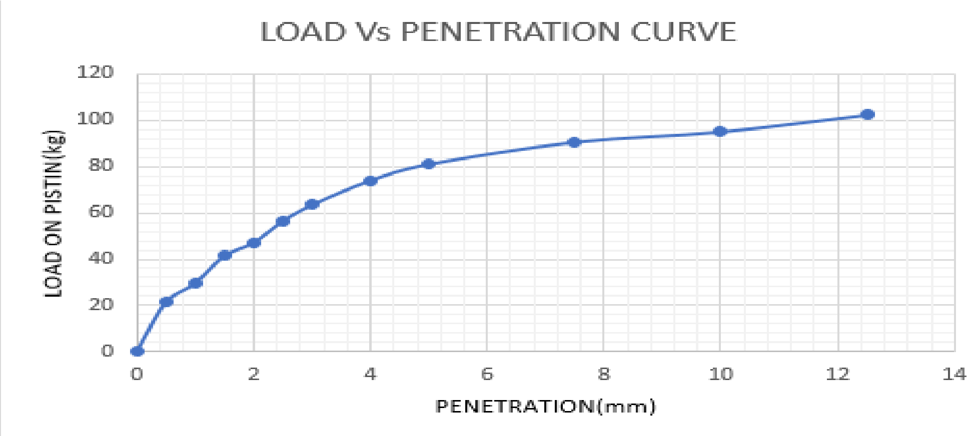


Fig 5.41 Load penetration curve for Treated soil with 1.5% Waste Plastic(Soaked)

It can be seen from the **Fig 5.41** the CBR value at 2.5mm was found to be more compared to the 5mm CBR value and it increases from 1% waste mix. Compared to 1.5% waste plastic Unsoaked CBR, the value of 1.5% waste plastic Soaked CBR found out to be less.

Table 5.43 Load penetration data for CBR test for 2% Waste plastic (Unsoaked)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	23.5
1	42.7
1.5	53.3
2	66.7
2.5	74.1
3	82.5
4	97.1
5	103.4
7.5	115.2
10	121.4
12.5	126.3

2.5mm CBR value = 5.41%
5.0mm CBR value = 5.03%

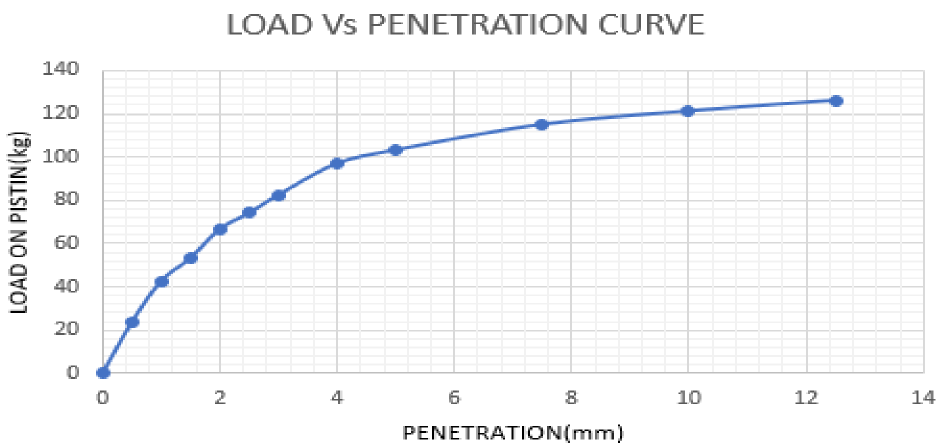


Fig 5.42 Load penetration curve for Treated soil with 2% Waste Plastic(Unsoaked)

It can be seen from the **Fig 5.42** the CBR value at 2.5mm was found to be more compared to the 5mm CBR value and it decreases from 1.5% waste mix.

Table 5.44 Load penetration data for CBR test for 2% Waste plastic (Soaked)

Penetration(mm)	Load on Piston(kg)
0	0
0.5	24.3
1	32.6
1.5	40.7
2	47.2
2.5	53.0
3	59.4
4	67.2
5	74.4
7.5	81.6
10	87.2
12.5	91.6

2.5mm CBR value = 3.87%
5.0mm CBR value = 3.62%

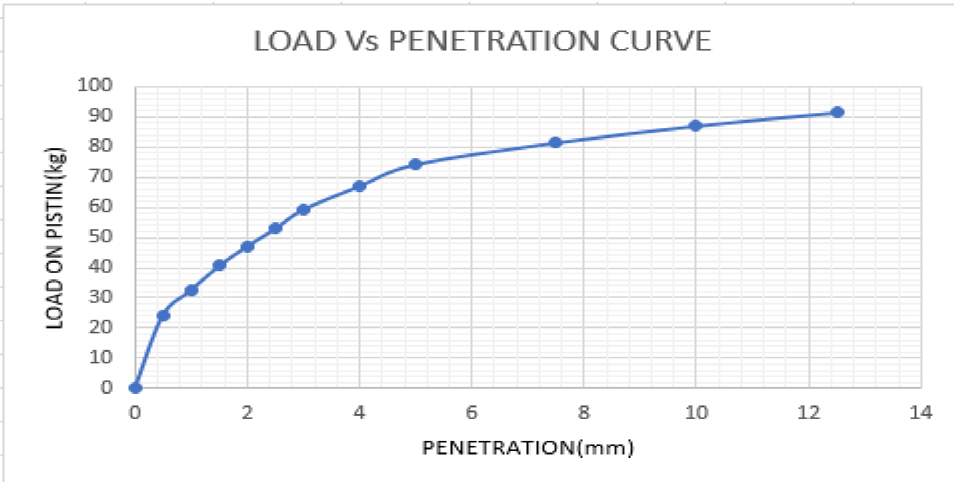


Fig 5.43 Load penetration curve for Treated soil with 2% Waste Plastic(Soaked)

It can be seen from the **Fig 5.43** the CBR value at 2.5mm was found to be more compared to the 5mm CBR value and it decreases from 1.5% waste mix. Compared to 2% waste plastic Unsoaked CBR, the value of 2% waste plastic Soaked CBR found out to be less.

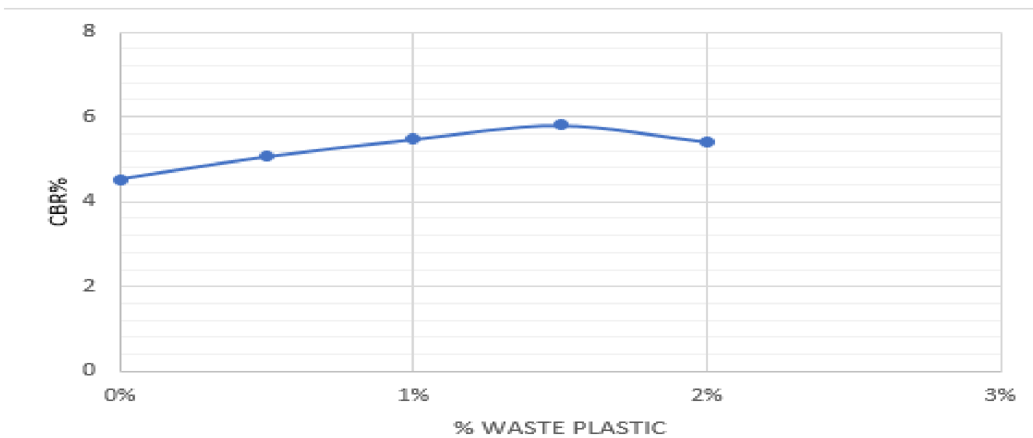


Fig 5.44 %Waste Plastic v/s CBR value(Unsoaked)

It can be seen from the above **Fig 5.44**, that CBR value at 1.5% found out to be maximum compared to the other values and the least value found out to be on untreated soil.

Table 5.45 Test values of Waste plastic addition to soil

% Waste Plastic	OMC(%)	MDD(KN/m^3)	UCS(Static)(kPa)	UCS(Dynamic)(kPa)	CBR %(Unsoaked)	CBR %(Soaked)
0%	19.1	15.88	242.36	231.09	4.52	2.65
0.5%	18.2	19.4	245.18	237.04	5.06	3.11
1%	18.4	18.78	262.08	256.79	5.47	3.65
1.5%	18.9	18.5	249.35	245.67	5.81	4.12
2%	19.3	18.4	247.99	244.56	5.41	3.87

5.4 Combine graphs of Marble dust and Waste plastic

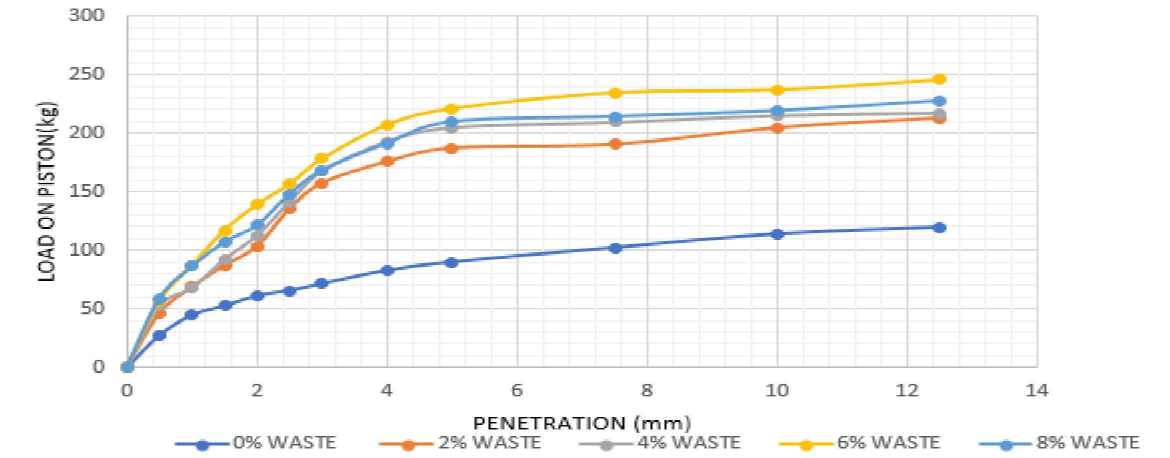


Fig 5.45 CBR values for various percentage of Marble Dust addition(Static Compaction)

From the **Fig 5.45**, the maximum value of CBR was found to be on 6% addition of Marble dust.

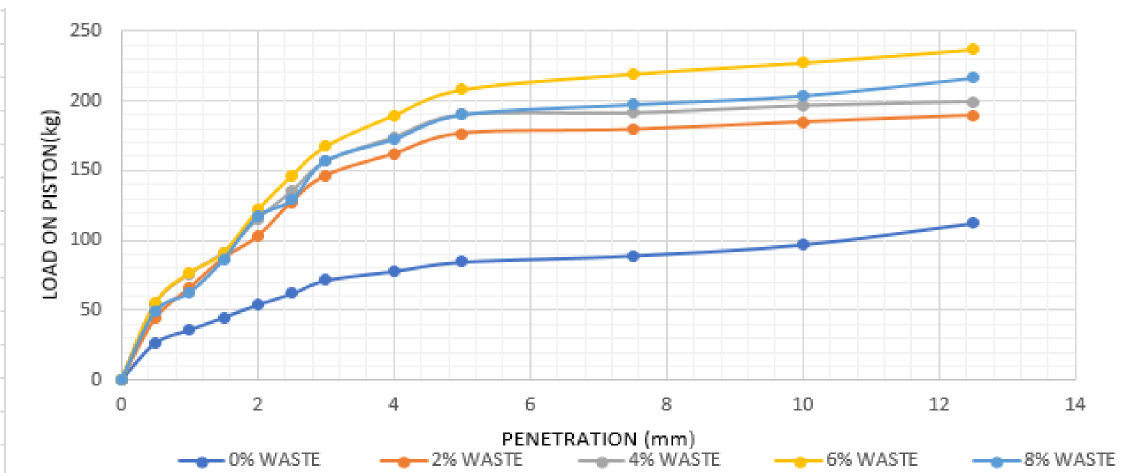


Fig 5.46 CBR values for various percentage of Marble Dust addition(Dynamic Compaction)

From **Fig 5.46**, the maximum value of CBR was found to be on 6% addition of Marble Dust after that it decreases.

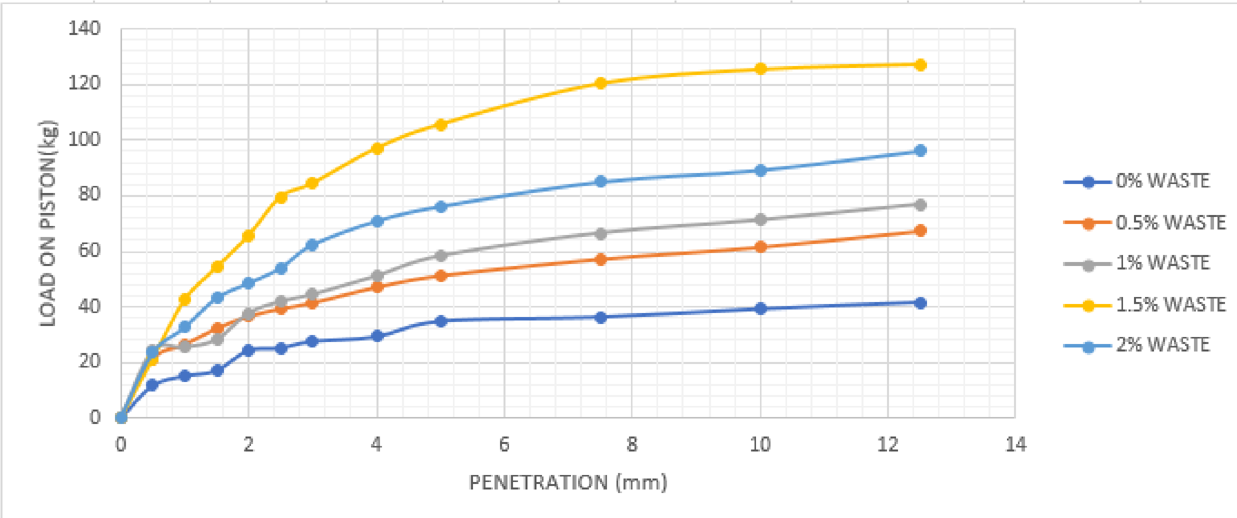


Fig 5.47 CBR values for various percentage of Waste Plastic addition(Unsoaked)

From **Fig 5.47**, the maximum value of CBR was found to be on 1.5% addition of Waste Plastic after that it decreases.

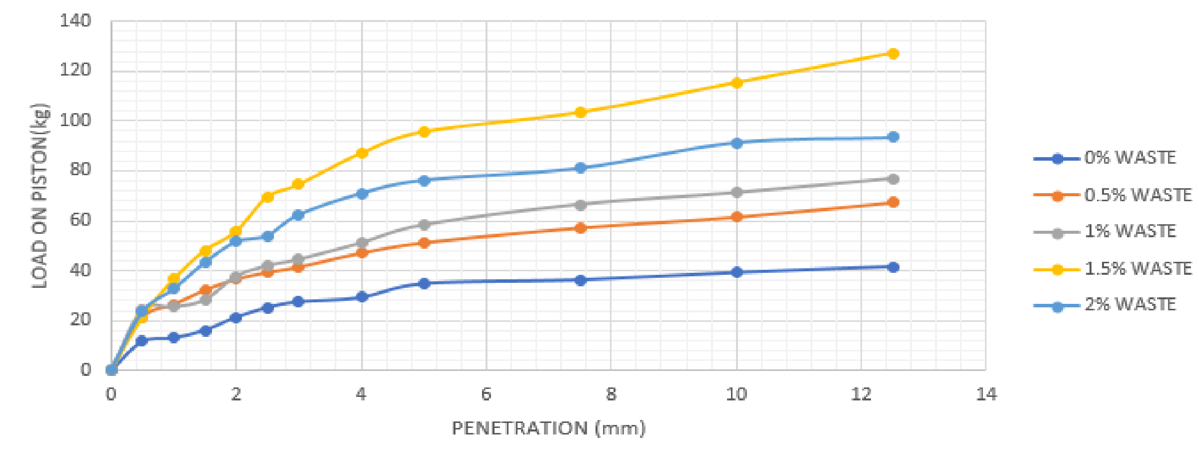


Fig 5.48 CBR values for various percentage of Waste Plastic addition(Soaked)

From **Fig 5.48**, the maximum value of CBR was found to be on 1.5% addition of Waste Plastic after that it decreases.

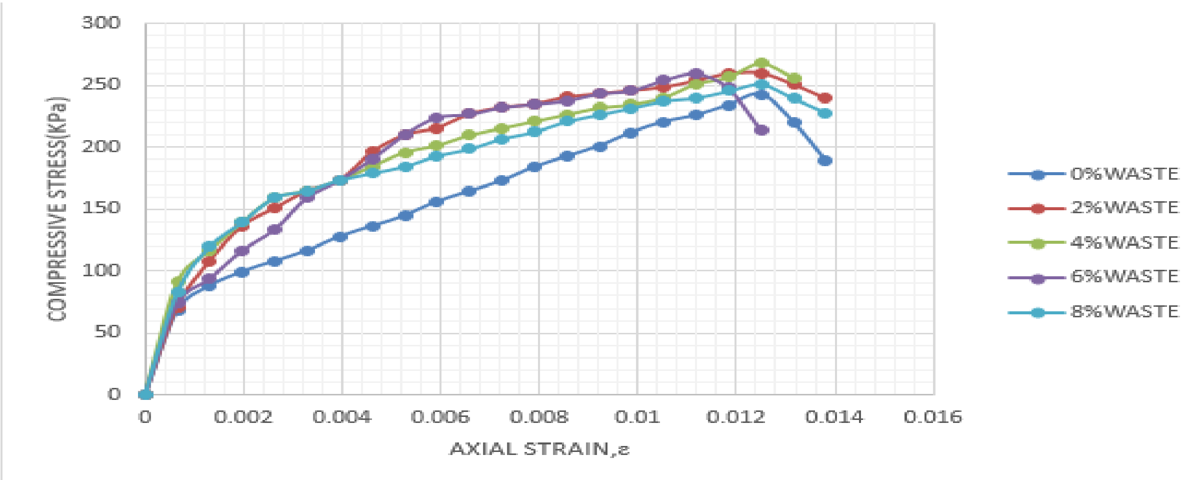


Fig 5.49 Graph for UCS test for various percentage of Marble Dust(Statically compacted)

From **Fig 5.49**, the maximum value of UCS was found to be 4% addition of Marble Dust by static compaction.

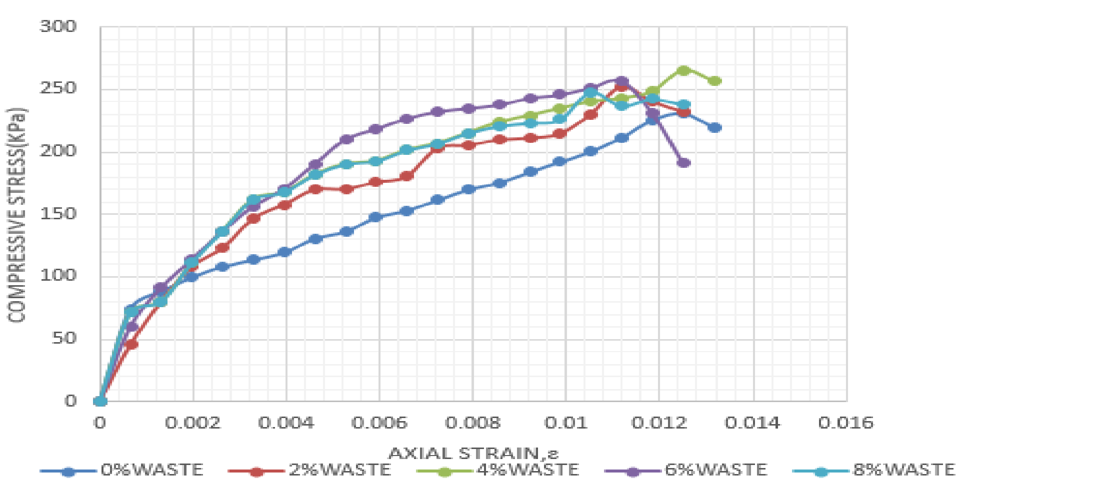


Fig 5.50 Graph for UCS test for various percentage of Marble Dust(Dynamically compacted)

From **Fig 5.50**, the maximum value of UCS was found to be 4% addition of Marble Dust by dynamic compaction. The dynamic compaction values show less UCS than static compaction.

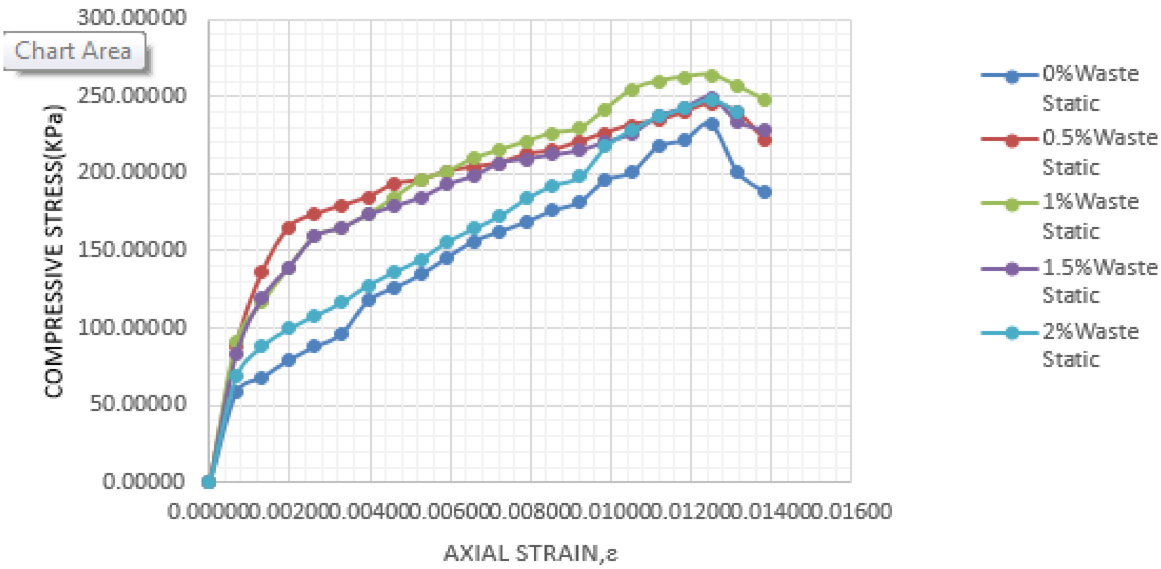


Fig 5.51 Graph for UCS test for various percentage of Waste Plastic(Statically compacted)

From **Fig 5.51**, the maximum value of UCS was found to be 1% addition of Waste Plastic by Static compaction.

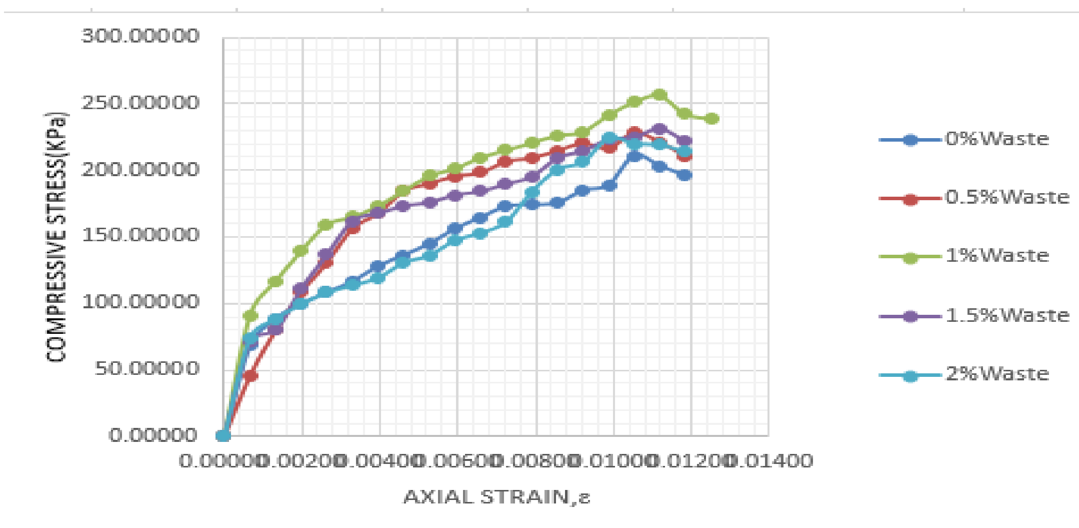


Fig 5.52 Graph for UCS test for various percentage of Waste Plastic(Dynamically compacted)

From **Fig 5.52**, the maximum value of UCS was found to be 1% addition of Waste Plastic by Dynamic compaction. The dynamic compaction values show less UCS than static compaction.

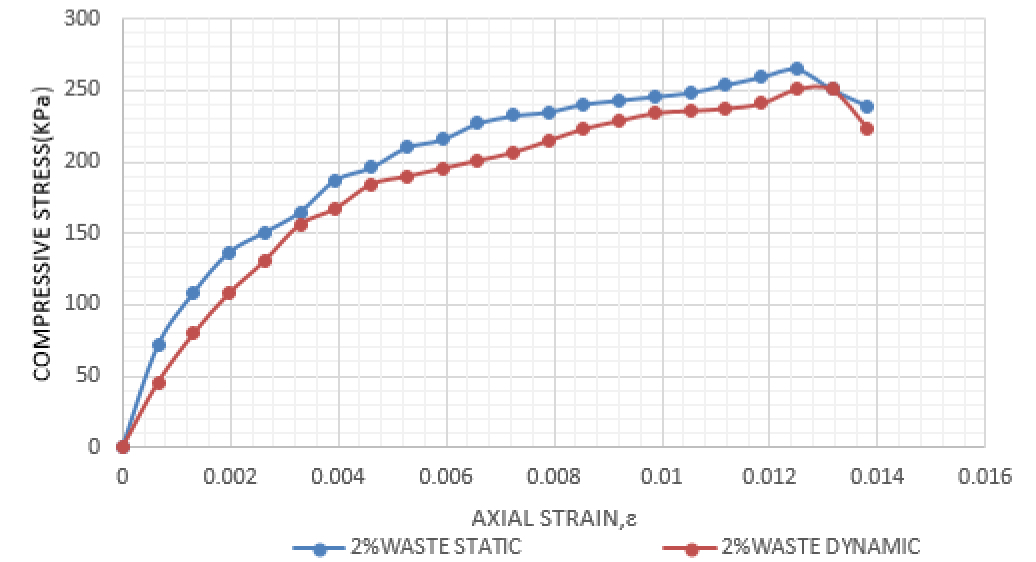


Fig 5.53 Superimposed Graph for UCS test for 2% of Marble Dust(Statically and Dynamically compacted)

From **Fig 5.53**, the static compaction value of UCS shows higher results than Dynamic compaction of UCS.

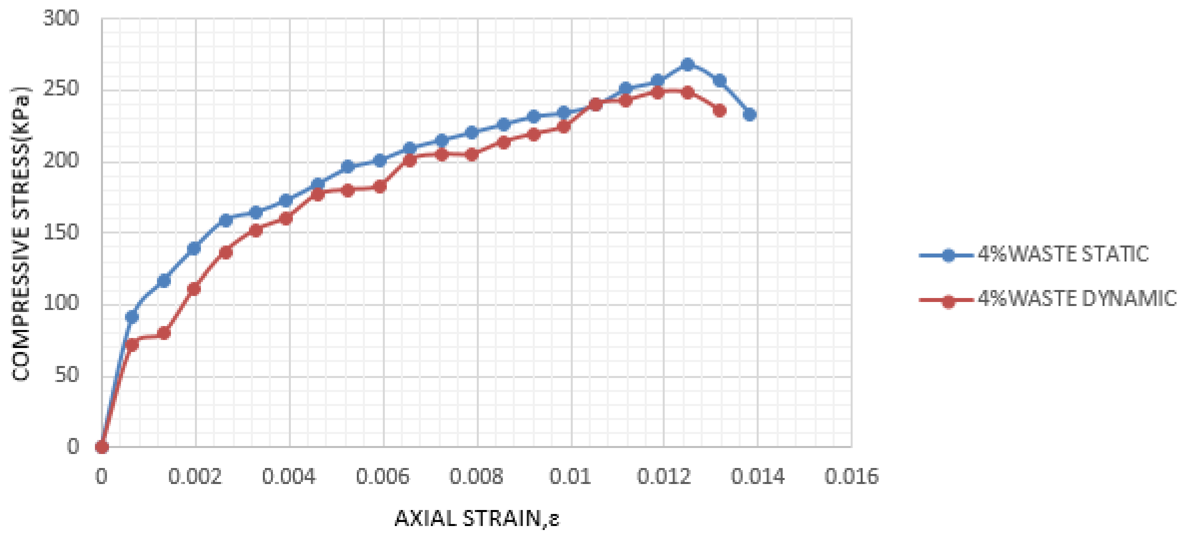


Fig 5.54 Superimposed Graph for UCS test for 4% of Marble Dust(Statically and Dynamically compacted)

From **Fig 5.54**, the static compaction value of UCS shows higher results than Dynamic compaction of UCS.

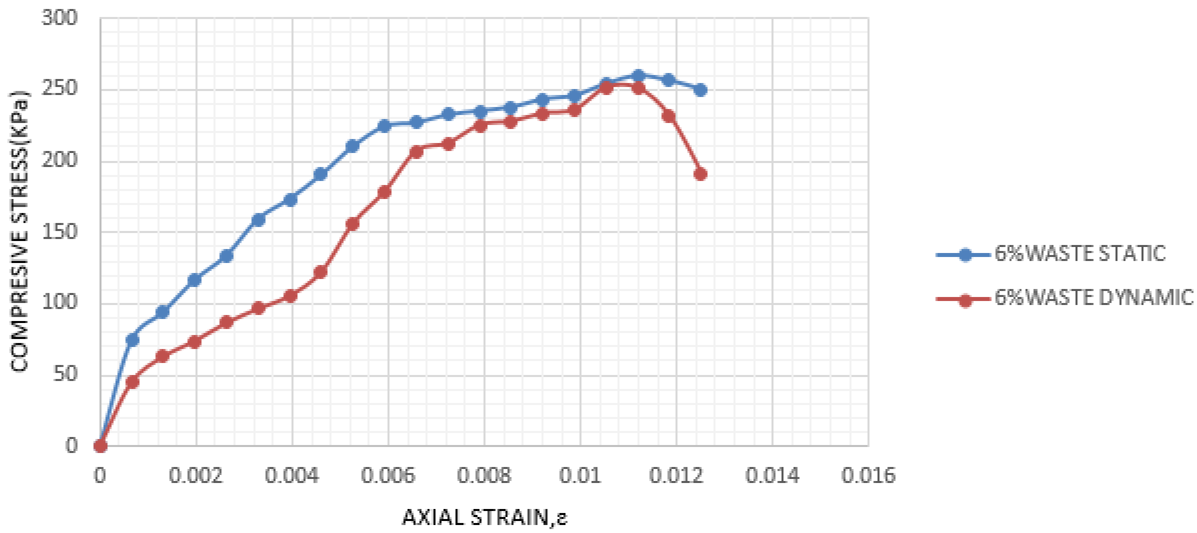


Fig 5.55 Superimposed Graph for UCS test for 6% of Marble Dust(Statically and Dynamically compacted)

From **Fig 5.55**, the static compaction value of UCS shows higher results than Dynamic compaction of UCS.

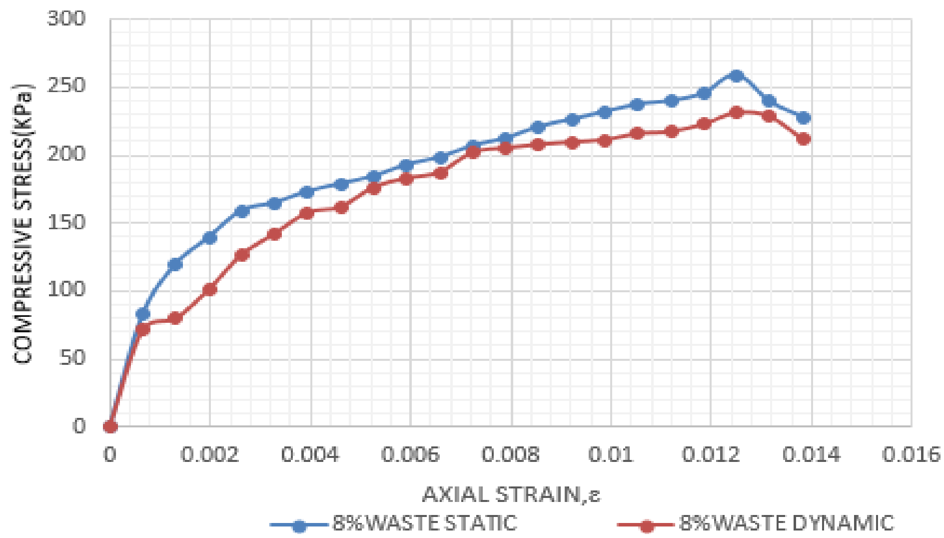


Fig 5.56 Superimposed Graph for UCS test for 8% of Marble Dust(Statically and Dynamically compacted)

From **Fig 5.56**, the static compaction value of UCS shows higher results than Dynamic compaction of UCS.

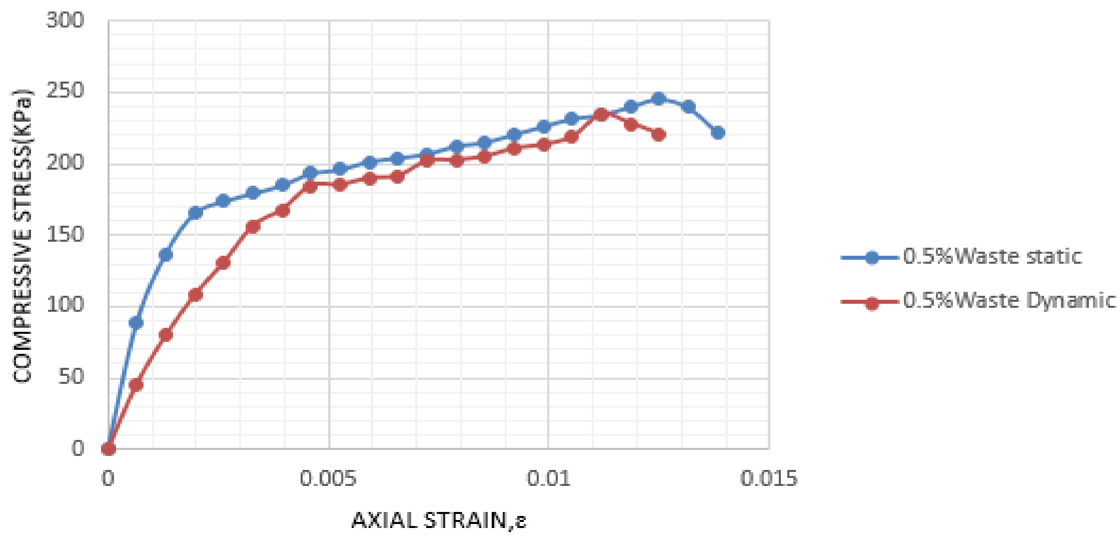


Fig 5.57 Superimposed Graph for UCS test for 0.5% of Waste Plastic(Statically and Dynamically compacted)

From **Fig 5.57**, the static compaction value of UCS shows higher results than Dynamic compaction of UCS.

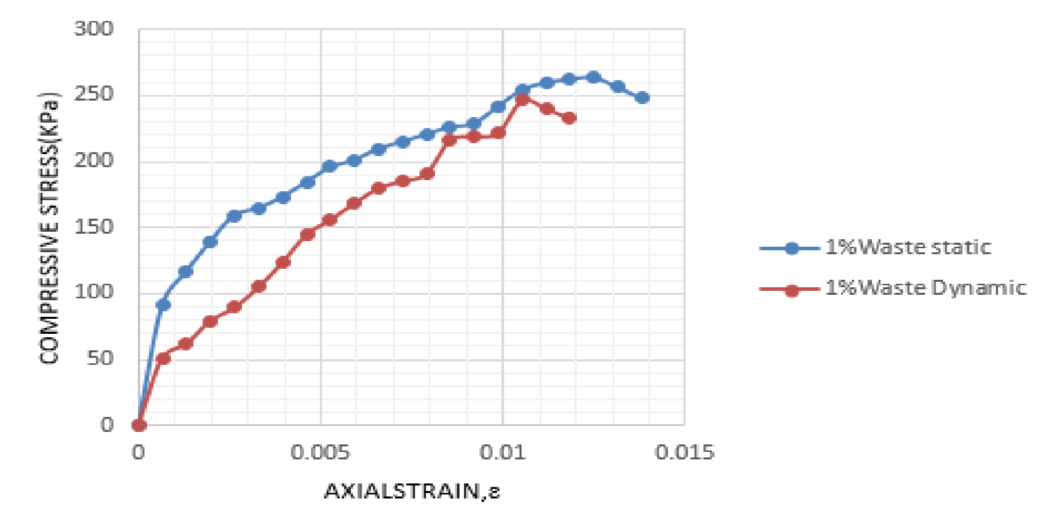


Fig 5.58 Superimposed Graph for UCS test for 1% of Waste Plastic(Statically and Dynamically compacted)

From **Fig 5.58**, the static compaction value of UCS shows higher results than Dynamic compaction of UCS.

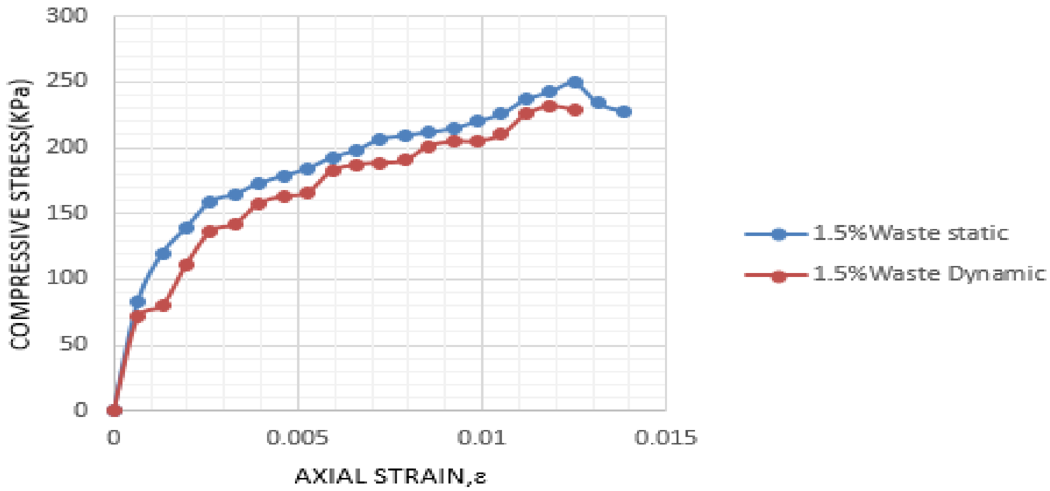


Fig 5.59 Superimposed Graph for UCS test for 1.5% of Waste Plastic(Statically and Dynamically compacted)

From **Fig 5.59**, the static compaction value of UCS shows higher results than Dynamic compaction of UCS.

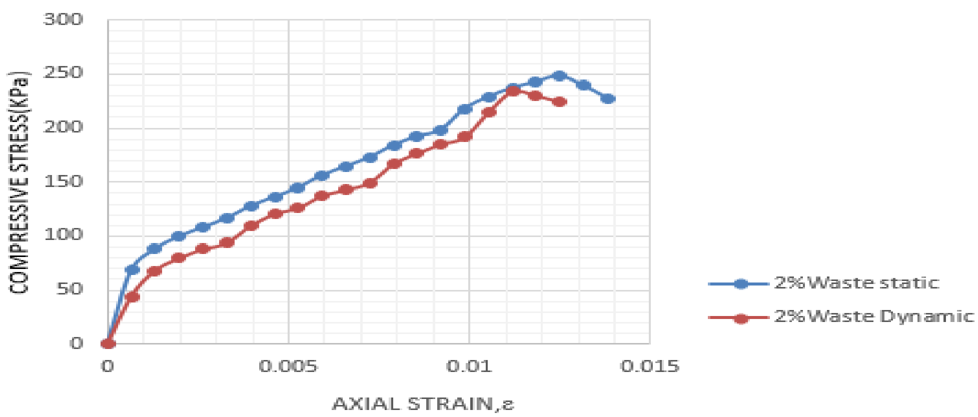


Fig 5.60 Superimposed Graph for UCS test for 2% of Waste Plastic(Statically and Dynamically compacted)

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

CHAPTER 6
DISCUSSION OF TEST RESULTS

Laboratory tests were carried out considering both the waste i.e., marble dust and waste plastic. One is considered to be an industrial 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 marble dust(2%,4%,6%,8% addition), in the Standard Proctor test, the optimum waste was found out to be on addition of 4% waste and after that its maximum dry density decreases. The marble dust is a material which contain calcium oxide(more than 50%), silicon oxide, magnesium oxide, aluminium oxide, Sulphur oxide, iron oxide(Bibekananda Naik 2019) etc.which are best known to bind soil particles together to gain maximum strength by reducing void ratio. At 4% addition of waste, the OMC of the soil also decreases compared to the untreated soil which means that at 4% 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 4% waste addition in case of dynamic compaction whereas in case of static compaction the maximum strength was found out to be on 4% 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 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 4% 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. Marble Dust alone can give strength to the soil but studies also suggest that Marble Dust along with other additive like mixing the Cement, Polypropylene Fibre with durability as well.

In CBR test, the percentage of waste addition were 2%, 4%, 6% and 8% in which the optimum value of CBR was found out to be on 6% addition of waste in both static and dynamic compaction and after that it decreases. Marble dust can show even better results with addition of other materials as mentioned above as Cement, Polypropylene Fibre (Ranjit Singh,2021) etc.

Another waste that was considered in this study is waste plastic (plastic shopping bags) is non-biodegradable and easily available waste material. In this study plastic shopping bags are cut into small pieces of sizes 15mmX40mm. Using this waste, Standard Proctor test, Unconfined Compressive strength test(static and dynamic) and CBR(Soaked and Unsoaked) test were done.

In Standard Proctor test, it has been seen that 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 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 is 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 waste plastic upto 0.5% the OMC decreases and MDD increases after that OMC increases and MDD decreases. At 0.5% addition of waste plastic OMC decreases from 19.1% to 18.2% and MDD increases from 15.88 KN/m^3 to 19.4KN/m^3 which is maximum.

In the Unconfined Compressive strength test by dynamic compaction, the maximum strength was found out to be on 1% addition of waste which is 256.70 KPa which falls under the category of very stiff soil and after that it decreases. This indicates that the quantity of waste plastic. But in static compaction, maximum strength was found out to be 262.08KPa on 1% 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 doesnot 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 5.81% and after that it decreases. The maximum value of soaked CBR was found to be on 1.5% addition of waste which is 4.12% and after that it decreases. The value of unsoaked CBR is found to be higher than soaked CBR value.

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

Test performed	Untreated soil(values)	Test performed	Treated soil(values)	
			Marble dust	Waste Plastic
Liquid limit	41.08%	OMC	18.3%(2% Addition)	18.2%(0.5%Addition)
			18.1%(4% Addition)	18.4%(1%Addition)
Plastic limit	23.17%		18.6%(6% Addition)	18.9%(1.5%Addition)
			19.3%(8% Addition)	19.3%(2%Addition)
Plasticity index	17.91%	MDD	16.26kN/m^3(2% Addition)	19.4kN/m^3(0.5% Addition)
			21.00kN/m^3(4% Addition)	18.78kN/m^3(1% Addition)
Specific gravity	2.662		19.2kN/m^3(6% Addition)	18.5kN/m^3(1.5% Addition)
			18.4kN/m^3(8% Addition)	18.4kN/m^3(2% Addition)
OMC(Standard Proctor test)	19.1%	UCS(Static)	264.90kPa(2% Addition)	245.18kPa(0.5% Addition)
			267.72kPa(4% Addition)	262.08kPa(1% Addition)
MDD	15.88KN/m^3		259.61kPa(6% Addition)	249.35kPa(1.5% Addition)
			250.61kPa(8% Addition)	247.99kPa(2% Addition)
UCS(Static)	242.36KPa	UCS(Dynamic)	262.08kPa(2% Addition)	237.04kPa(0.5% Addition)
			264.90kPa(4% Addition)	256.79kPa(1% Addition)
UCS(Dynamic)	231.09KPa		256.79kPa(6% Addition)	245.67kPa(1.5% Addition)
			247.99kPa(8% Addition)	244.56kPa(2% Addition)
CBR(Static)	4.79%	CBR	7.82%(Static)	5.06%(Unsoaked)
			10.32%(Static)	5.47%(Unsoaked)
			11.43%(Static)	5.81%(Unsoaked)
			10.79%(Static)	5.41%(Unsoaked)
CBR(Dynamic)	4.52%	CBR	7.23%(Dynamic)	3.11%(Soaked)
			9.85%(Dynamic)	3.65%(Soaked)
CBR(Soaked)	2.65%		10.68%(Dynamic)	4.12%(Soaked)
			9.41%(Dynamic)	3.87%(Soaked)

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. In Standard Proctor test, the maximum dry density was found to be maximum on 4% addition of Marble dust which means that the soil particles got more compacted during 4% waste addition. But after that on increasing addition of waste the OMC increases on 6% and 8% addition of Marble dust compared to the 2% and 4% waste.
2. By performing the Unconfined Compressive Strength test by dynamic compaction, the compressive strength of each sample was determined. From the different percentages of marble dust used, the maximum compressive stress was found to be at 4% waste addition. After that the compressive stress gradually decreases and the minimum compressive stress was found to be on untreated soil.
3. But in static compaction of Unconfined Compressive Strength test, the maximum strength of the soil was found out to be on 4% addition of marble dust and after that it decreases but the results of static compaction are more compared to the dynamic compaction.
4. By performing the CBR test by dynamic compaction, the CBR value of each sample was determined. From the different percentages of marble dust used, the maximum improvement in CBR value is observed when 6% of marble dust is mixed with soil. It is concluded that proportion of 6% marble dust in soil giving maximum CBR value.
5. In static compaction of CBR test, the maximum CBR value was found out to be on 6% addition of marble dust, the results of static compaction are more compared to the dynamic compaction.
6. In the Standard Proctor test on addition of waste plastic, the maximum dry density was found to be maximum on 0.5% addition of waste plastic. But after that on increasing addition of waste the OMC increases on 1%,1.5% and 2% addition of waste plastic compared to the 0.5% waste.
7. In UCS test by dynamic compaction, the maximum strength attained at 1% addition of waste plastic. Similarly, by static compaction maximum strength attained at 1% addition of waste plastic. Static compaction values are more compared to the dynamic compaction values of UCS.
8. The Unconfined Compressive Strength of statically compacted soil using marble dust exhibit higher values compared to the waste plastic.
9. CBR value of soil increases when waste plastic is added to the soil in different layers for both unsoaked and soaked condition.

10. Maximum improvement in CBR value is observed when 1.5% of waste plastic is mixed with the soil and it again decrease when 2% of waste plastic is mixed with the soil. It is concluded that proportion of 1.5% of waste plastic in soil giving maximum unsoaked and soaked CBR value
11. .In CBR test, results using marble dust, the peak value is more compared to the waste plastic.
12. Comparing both the waste, marble dust shows more prominent result compared to the waste plastic.
13. In comparison of static and dynamic compaction of UCS, results using marble dust are more compared to the waste plastic.

7.2 Scope for future study

1. Study can be carried out using waste percentage of waste plastic beyond 2% and using plastic bags strips of dimensions; 6 x 15mm, 6 x 30 mm, 6 x 45 mm, 12 x 15 mm and 18 x 15 mm.
2. Study can be carried out by using marble dust can be extended beyond 8%.
3. Rather the study with marble dust can be carried out by using it with other chemically active waste like lime, cement etc.
4. 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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