STABILIZATION STUDIES THROUGH IMPROVED STRENGTH CHARACTERISTICS OF KAOLINITE CLAY USING GROUNDNUT SHELL ASH AND ONION PEEL POWDER

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Submitted by

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I, Sandeep Rajput, Roll No. 2K19/GTE/11 of M.Tech.(Geotechnical Engineering), hereby declare that the project Dissertation titled " Stabilization studies through improved strength characteristics of kaolinite clay using groundnut shell ash and onion peel powder" which is submitted by me to the Department of Civil Engineering , Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is original and not copied from any source without proper citation. This work has not previously formed the basis for the award of any Degree, Diploma Associateship, Fellowship or other similar title recognition.

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ABSTRACT

Industrialization and urbanization have polluted available land, and most agricultural land has recently been converted to development sites. When exposed to changes in water concentrations, a large section of agricultural land has expansive soil with significant inflammation and shrinkage properties. The procedure of treating or replacing soil with a suitable and effective substance to improve its mechanical qualities for building is known as soil stabilization. Valorization of waste from groundnut mills and onion peel waste in improving soil properties used for building and construction projects are cost-effective and economically feasible waste management methods. Other studies have found that using groundnut shell ash (G.S.A.) and onion peel powder (O.P.P.) on clay is beneficial. As a result, the purpose of this research is to examine the qualities of clayey soils containing varied percentages of G.S.A. and O.P.P. The atterberg limits-plastic limit, liquid limit, plasticity index, California bearing ratio, shear strength, and moisture content tests were used to evaluate the efficacy of G.S.A. and O.P.P. in improving soil parameters. Improved soil efficiency and geotechnical qualities have also been emphasized for long-term use in various commercial activities. With the addition of 2%, 4%, 6%, 8% of G.S.A and 10%, 20%, 30%, 40% of O.P.P., Unconfined Compressive Strength (UCS), Plastic Limit, Compaction, Liquid Limit, and California Bearing Ratio (CBR), significantly increase till certain limit.

Keywords: Stabilization, Kaolinite Clay, G.S.A., O.P.P.

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GLOSSARY OF TERMS

OPP	Onion Peel Powder
GSA	Groundnut Shell Ash
LL	Liquid Limit
PL	Plastic Limit
PI	Plasticity Index
CBR	California Bearing Ratio
UCS	Unconfined Compressive Strength
OMC	Optimum Moisture Content
MDD	Maximum Dry Density
I_f	Flow index
γ	Bulk density of soil
γ_d	Dry density of soil

CHAPTER 1

INTRODUCTION

1.1 INTRODUCTION

Kaolin deposits can be found in India's northeastern area. Deopani, in Assam's Karbi Anglong District, has a kaolinite deposit of roughly 1.0 million tones. The clay is still being extracted. It is linked to various black-colored, iron-bearing components, lowering the clay's value. A detailed inspection of the characterization and beneficiation of clay for various uses is required for its utilization. This research looks into the characteristics of the deposit's clay, as well as the feasibility of beneficiating it for diverse purposes utilizing well-known methods like size separation, magnetic separation, and organic acid leaching.

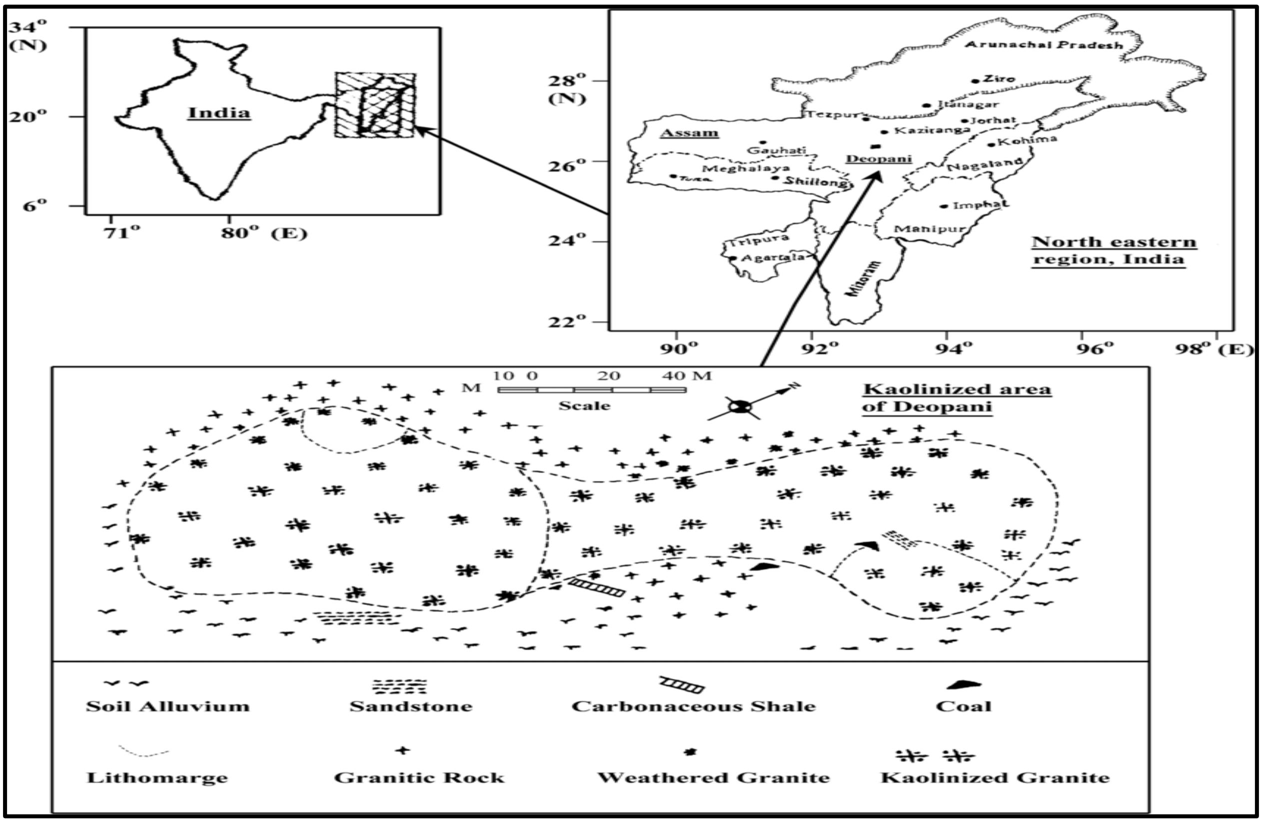

Fig.1.1 Location of Kaolinite Clay in Assam

Source-https://www.researchgate.net/profile/Rajib-Goswamee/publication/248536252/figure/fig1/AS:715700524023810@1547647710567/Kaolinized-area-of-Deopani.png

1.2 GENERAL

Kaolinite clay is a highly adaptable industrial material. Clay was used as a ceramic base material for the first time. Currently, the clay is used as a paper coating and filler pigment, as well as filler in paint, rubber, insecticide, pharmaceutical formulations, and cosmetics. The purity of the clay is the most important factor in determining its suitability for various applications. The chemical composition of pure kaolinite is 46.55% Sio_2, 39.60% Alo_3, and 13.97% H_2O. (Alaneme, 2016[1])

Revolution and urbanization are major turning points in evolutionary history. Since the development of machines and technology, the establishment of industrial centers and commercial structures has improved tremendously. A building's or road's foundation is necessary for efficient load transfer to the subsoil underneath the top layer. (K.D. Oyeyemi, 2017[9])**.** The quality of the soil used in building has a significant impact on structural and design aspects (Arslan,2010[2]). Large volume variations caused by soaking and drying seasons cause extensive damage to civil engineering projects, especially small buildings, shallow foundations, and heavily loaded concrete structures (Zhang, 2009[21]). This not only disrupts structures, but also degrades the environment by releasing huge hazardous chemicals.

1.3 EXPANSIVE SOILS

The solid waste created by many sectors in India is a source of growing waste management issues. However, by repurposing industrial waste for a variety of construction activities, this source of concern can be turned into a source of opportunity. Solid waste created can be used to improve soil properties and for long-term solid waste management (M.D. Meena, 2019[13]). Differential movements beneath a building's foundation are likely to occur due to volume changes in expansive soils, generating various forms of problems (L.D. Jones, 2012[12]). Stabilization of soil with various chemical agents is still the most widely known way for eliminating the destruction caused by expansive or weak soils (Vijayan, 2020[20]). Mixing coarse grain soil with fine grain soil to create a mixture with sufficient internal friction and cohesion is a more prevalent way to stabilization. Cement and lime have been utilized for soil stabilization for a long time (James, 2016[8]); nevertheless, their prices have raised significantly due to their high energy costs. However, due to their high energy costs, their prices have risen dramatically. Soil stabilization is an essential source of waste management for industries, allowing for more efficient building construction and less detrimental environmental effects from pollutants released from soil (Pacheo- Torgal, 2014[16]).

The following are some of the most common methods for improving soil:

1. Compaction

(a) Static compaction, which includes compaction piles;

(b) Dynamic compaction, which includes vibro flotation, explosions, and heavy tamping.

2. Consolidation

(a) Water or surcharge-based preloading

(b) Drainage: Sand drains, rope drains, sand wicks, cardboard drains, plastic drains, electro-osmosis, and plastic geotextile drains are all examples of drainage.

3. Grouting and Injection: Bentonite grouting, chemical grouting, lime slurry injection, and cement grouting are all examples of grouting and injection.

(a) **Soil stabilization**: mechanical, admixture, and lime heaps

(b) **Soil Reinforcement**: Granular piles, stone columns, soil nailing, geofabrics, Geogrids, geomembranes, and geocomposites, among others.

CHAPTER 2
LITERATURE REVIEW

- **S. karthikeyan and B. Selvarani** investigated that Onion ash and powder have a greater specific gravity. The liquid limitation was reduced slightly using onion powder. The plastic limit was reduced in both onion ash and onion powder. The strength of UCC and Direct shear both increased significantly, although onion ash increased very slightly. When compared to onion powder, onion ash has a higher carrying capacity. Onion ash has a better shear strength and resistance than onion powder. Onion ash, unlike onion powder, can withstand liquefaction, collapse, and structural subsidence. (B.Selvarani, 2021[4])

- **Navanmi Chandra B and Veena Vijayan L** investigated that the addition of Bamboo fiber, Banana fiber and RHA, improved the clayey soil's characteristics. The optimum moisture content and maximum dry density increase when Bamboo fiber, Banana fiber and RHA are added. However, when the right amount of RHA is mixed in with the various percentages of Banana fiber and Bamboo fiber the optimum moisture content rises while the maximum dry density falls. The optimal percentage of RHA has been determined based on the test findings. By adding the optimal percentage of RHA to the various percentages of Banana fiber and Bamboo fiber in the soil, the ideal proportion is 1% and 0.5%. (B, 2017[3])

- **P.Ranga Ramesh, Dr D shrinivas and V.Subhasini** investigated that the plastic limit has been increased because groundnut shell ash was added to black cotton. This change in soil character is most likely due to the GSA providing bivalent calcium ions, which replaced less securely bound monovalent ions in the double layer enclosing the clay particles, increasing the plastic limit. The plastic limit increased with a higher dose of GSA, owing to an increase in the amount of particles in the mixture. Since the plasticity index is calculated using the plastic and liquid limits, no independent approach for lowering the plasticity index, which represents soil workability, has been proposed. With the addition of groundnut shell ash to the black cotton soil, the shrinkage limit continues to rise. This is due to the volume change caused by adding ash to the soil in various quantities. The shrinkage limit increased with a larger dose of GSA, which was related to an increase in the amount of fines in the sample. The CBR value of the soil increases slightly as the ash concentrations increase. It is possible that the minor increase in strength is due to a lack of calcium, it is necessary for the formation of calcium silicate hydrate, the most important component of strength growth. (P.Ranga Ramesh, 2019[15])

- **Folagbade O P Oriola and Moses George** investigated that Groundnut shell ash is used in natural soil resulted in a peak 7-day UCS value for WA compactive effort at SP of 526KN/m^2 at 6% GSA and 455KN/m^2 at 4% GSA content. TRRL (1977) specified 1710 KN/m^2 for base materials stabilization using OPC, but this rate fell short. They also failed to connect Ingles and Metcalf's requirement of 687–1373 KN/m^{2}for sub-base (1972). The peak soaking CBR values of 4% at SP and 4% at WA were achieved at 6 percent (GSA) and 0 percent (GSA), respectively. These values were low enough to meet the Nigerian General Specifications' requirements. Finally, the sample's durability tests failed to satisfy the required standard. (Falagbade O P orola, 2020[5])

- **Ebin S Wilson and Sudha A R** investigated that the addition of groundnut shell ash and glass powder to clayey soil improves its characteristics. The MDD of G.S.A. compacted soil increases significantly at 2% and then with the addition of glass powder, gradually reduces, the MDD of the soil-glass powder combination improved up to 6%, after which it dropped. At 6% groundnut shell ash as an addition, the highest UCC value of 0.756 Kg/cm^2 is reached. UCC values of 6 percent were obtained for both glass powder and GSA. For 6% groundnut shell ash, the maximum CBR value is 10.87%. (S Ebin, 2017[18]).

- **Pratik V. Shah, Pavasiya Dishant and Harshil R. bhavsar** investigated that The results show that when GSP + Soil is increased by 4%, the value of UCS and CBR increases as well. When the GSP is added, the value of Liquid Limit increases by 82.25 percent. The results of the Free Swell Index (FSI) demonstrate that the volume change has decreased dramatically from 90% to 25%. The FSI has been reduced by 72.22 percent. Because stabilized soil has the maximum UCS value yet the FSI is 63.63 percent, the ideal dose for the GSP was discovered to be 4% for using it as a foundation material in building. Because the soil has a value greater than 50%, it cannot be used. Because stabilized soil has the maximum CBR value, the ideal dose for the GSP was discovered to be 4% when using it as a foundation material for flexible pavement. At a 10% dose of GSP, the UCS and CBR values were observed to be lowered to 2.99 kg/cm2 and 8.45 percent, respectively. The values are lower than the black cotton soil's initial value; however the FSI is quite low, at 25%, when compared to the other dosages. The reduction in the value of UCS and CBR for a 10% dose of GSP is almost the same, i.e. 10.48 percent from the original value. (Shah P. D., 2020[19])

The formation of expansive soil is caused by the deposition of microscopic and submicroscopic particles. It has a high mineral content and can be turned to plastic by altering the water content. Organic clay contains organic materials, which is usually dark grey or black in appearance. Adsorbed water and particle attraction aid in the deformation of expanding soil's plasticity.

Aluminum silicates are found in expanding soil minerals in two fundamental units:

• Tetrahedron of silica
• Tetrahedron of aluminum

Kaolinite, Illite, and Montmorillonite are some of the other minerals found in clay. The expansive soil may be found predominantly around India's beaches. Thane Creek, Bombay Port, New Bombay City, Cochin, Kandla, and Haryana are among the clayey deposited districts, with Ambala, Faridabad, Kaithal, Hisar, Panipat, and Sirsa among them. Swelling damages in various construction buildings and structures occur when soil elements contain any of the following ingredients: montmorillonite in soil, water content around plastic content of soil, or soil is close to a source of water. (Zumrawi, 2017[22]) Differential movements beneath a building's foundation are likely to occur due to volume changes in expansive soils, generating various forms of problems. (L.D. Jones, 2012[12])

In general, expansive soils aren't a problem if moisture levels are kept consistent throughout the soil. Furthermore, water fluctuation from subsurface and surface water layers can be controlled using horizontal barriers in the form of membranes around a building (Hanse, 2020[7]), vertical moisture barriers installed around the perimeter of the building (Zheng, 2019[21]), and an adequate drainage system (Qi, 2019[17]). Stabilization of soil with various chemical agents is still the most widely known way for eliminating the destruction caused by expansive or weak soils (Vijayan, 2020[20]). As a result, replacing expansive and weak soil with non expansive soil or improving mechanical qualities through soil stabilization is critical.

2.1 NEED OF THE PRESENT STUDY

Soil stabilization is an efficient method for improving soil geotechnical qualities and stabilizes it. The interplay of soil-admixtures, sugarcane ash, and surrounding soil, as well as embankment fluctuations, determines the degree and amount of improvement in soil properties. In view of the foregoing, the proposed thesis demonstrates the effect of Groundnut shell ash and onion peel powder on soil stability restoration.

To explore the effectiveness of G.S.A. and O.P.P. as a stabilizer for Kaolinite soil, the following objectives are planned:

- To find out about the geotechnical properties of Kaolinite soil.
- To determine the change in geotechnical properties, upon addition of different percentage of G.S.A. and O.P.P.

- To demonstrate the effect of G.S.A. and O.P.P. on soil compaction characteristics and strength.
- To determine the usefulness of G.S.A. and O.P.P. as soil reinforcement.
- To arrive at the optimum dosage of G.S.A. and O.P.P.
- To evaluate the suitable blend that can be used in stabilization of Kaolinite soil.
- Utilize locally available materials to reduce construction costs.

CHAPTER 3

MATERIALS

Groundnut shell ash and onion peel powder were combined with Kaolinite clay in this study to improve the UCS value, compaction of kaolinite Clay and California bearing ratio (CBR).

3.1 SAMPLE COLLECTION

Experiments were conducted on Kaolinite clay obtained from Deepika Minerals and Chemicals in Siraspur, Delhi with latitude 28.76 and longitude 77.13. Approximately 35 kg kaolinite clay was used. The properties of kaolinite clays are accessible in Table 3.1

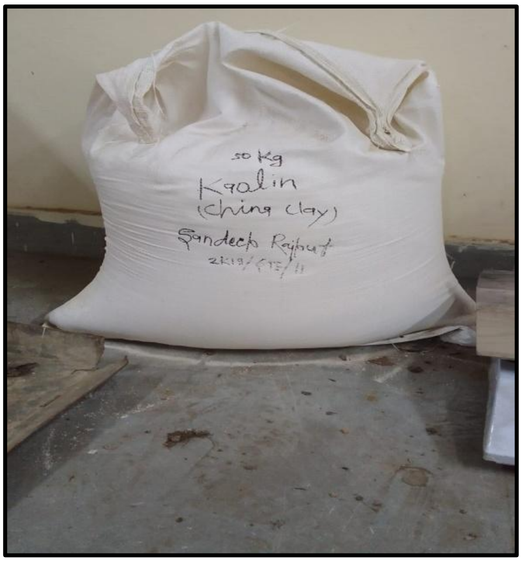

Fig.3.2 Kaolinite Sample (50kg)

Table 3.1: Properties of Kaolinite soil

Property	Standard Values
Specific Gravity	2.54%
Liquid Limit	47.9%
Plastic Limit	16.55%
Plasticity Index	31.35%
MDD	12.4KN/m^3
OMC	23%

3.2 GROUNDNUT SHELL ASH

A groundnut vendor in Rohini Sector 17, New Delhi, provided the groundnut shell. The ash was collected after the shells were burned on a metal sheet. A total of 35kg of shell was burned.

Groundnut shells compensate for around 21% of the weight of a dried peanut pod, showing that there is a large amount of shell leftover after groundnut processing. Groundnut shells accumulate as a result of increased production and are either burned or buried. Groundnut shell ash was obtained by burning of groundnut shell. This whole process is showing in Fig. 3.3, 3.4 and 3.5

Fig. 3.3 Groundnut shell in sun light

Fig. 3.4 Burning of groundnut Shell

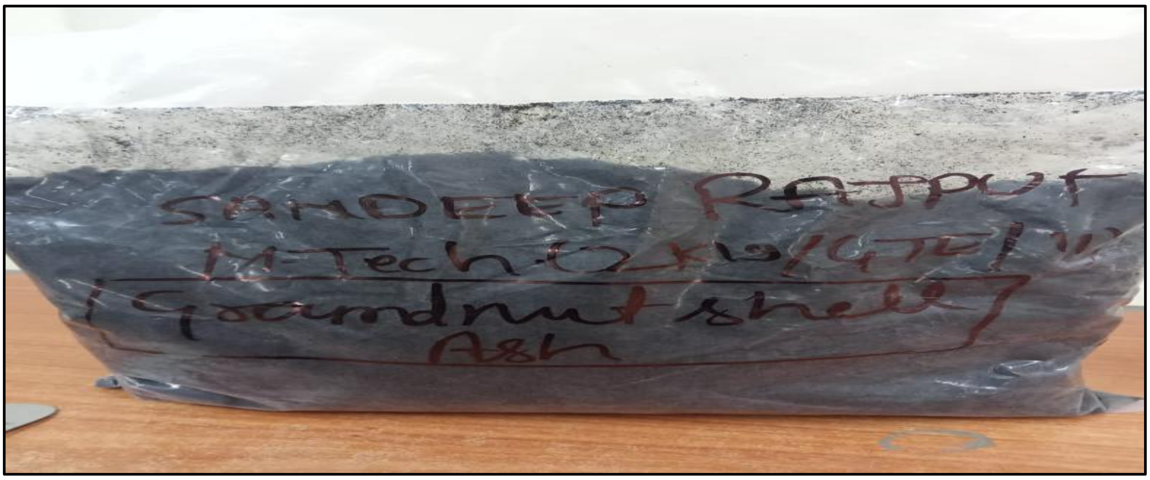

Fig. 3.5 Groundnut shell ash

Table3.2: Composition of Groundnut shell ash

Element	Weight (%)
SiO_2	34.2
Al_2O_3	12.42
Fe_2O_3	14
CaO	14.2
MgO	2.2
Na_2O_3	0.047
K_2O	15.47
P_2O_3	2.3
MnO	0.36
SO_3	0.65

(Alaneme, 2016[1])

3.3 ONION PEEL POWDER

Onion is a widely available substance, and onion waste is generated in a variety of ways every day, all of which were successfully utilized in this experimental investigation. In both black cotton and expansive soil, onion peel ash improves strength. In this research onion peel was collected from local Delhi's vegetables mandi from Rohini sector 17 New Delhi.

Fig. 3.6 Onion peel

Fig. 3.7 Grinding of onion peel for making powder

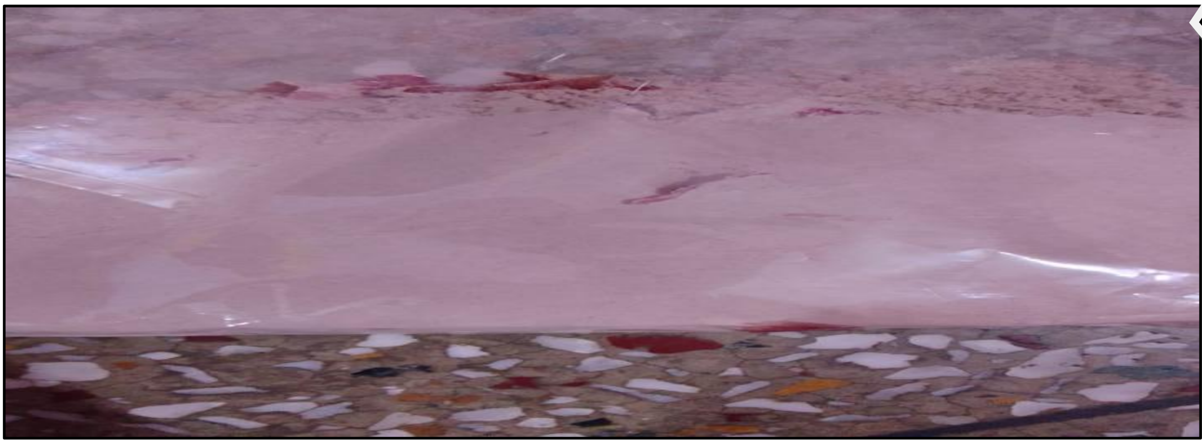

Fig. 3.8 Onion peel powder

CHAPTER 4

METHODOLOGY

For evaluating several laboratory tests, different concentrations of groundnut shell ash and onion peel powder mixture were utilized in the experimental study. This research delivers reliable measurements of the experimental study, due to the fact that they are avoided owing to the time and money they generate. To replace them, modern techniques and instruments are used to conduct quantitative and qualitative tests. Furthermore, laboratory-based testing techniques enable a deeper knowledge of the mechanistic aspects and controlled parameters required to achieve the desired outcomes. The results of multiple tests are analyzed to measure the various qualities of the soil.

4.1 LIQUID LIMIT TEST

The liquid limit test can be used to compute the compression index, which is important in settlement analysis. It is soft if the natural moisture content of the soil exceeds the liquid limit; it is brittle and hard if it falls below the liquid limit. To classify soils and evaluate their flexibility, the liquid limit value is used.

The liquid limit is defined as the moisture content at which a groove cut by a standard tool into a sample of soil obtained in a standard cup closes for 12 mm after 25 blows in a standard manner. The shear strength of the soil has reached its limit. I am doing liquid limit test in DTU laboratory showing in Fig. 4.9

Fig.4.9: Liquid limit determination (DTU)

Procedure:

- Approximately 130gm of air-dried soil must be taken from a completely mixed material that passes the 425 micron I.S sieve.
- In a mixing disc, distilled water is mixed with the soil sample to make uniform paste. With 30 to 35 drops of cup, the paste should be thick enough to close a standard groove of 12 mm length.
- Allow at least 24 hours before testing clayey soil, to ensure even moisture distribution.
- Before the test, the soil sample should be thoroughly mixed with as few strokes of the spatula as much possible. A portion of the uniform paste is placed in the cup of the Mechanical Liquid Limit device and spread.
- Trim the thickest area to a depth of 1 cm and return the surplus soil to the dish at the same time.
- Strong strokes with the grooving tool (ASTM Grooving Tool for Sandy soils and Casagrande's Grooving Tool for Clayey soils) along the diameter at the follower's centre line determine the soil in the cup, resulting in a clean pointed groove of proper dimension.
- To lift and lower the cup, turn the crank at a pace of two revolutions per second until the two parts of the soil come into contact for around 12 mm by flow alone. Keep count of how many strokes it takes to seal the groove for about 12 mm.
- Repeat the test 3-4 times more with different moisture concentrations each time for blows should be between 15 and 35.

Calculation

A flow curve indicating water content on an algorithmic scale and the quantity of drops on a logarithmic scale must be displayed on a semi logarithmic graph. The flow curve is a straight line drawn as near to the four or more depicted points as possible. The moisture content of 25 drops must be rounded to the nearest whole number and reported as the soil's liquid limit.

Flow index, $I_f = \frac{W1-W2}{\log_{10}\left(\frac{N2}{N1}\right)}$

Where, I_f = Flow Index

W1= Moisture content as a function of N1 drop.

W2= Moisture content as a function of N2 drop.

4.2 PLASTIC LIMIT TEST

The moisture content at which soil starts to behave like a plastic material is known as the plastic limit. When it is rolled into 3.2mm (1/8inch) diameter threads, the soil will crumble at this water content. In Fig. 4.10 I am performing plastic limit test.

Fig.4.10: Plastic limit determination (DTU)

Apparatus:

- For rolling plastic limit threads, use a 1 cm thick glass plate that is at least 30 cm (12 inch.) square.
- Spatula with 2 cm wide blade and a length of 10 to 13 cm.
- Oven for drying.
- Water Content Containers.
- 3.2mm diameter, 100mm long metallic rod

Procedure:

- Form an ellipsoidal ball out of a 1.5 to 2.0 g chunk of the plastic-limit specimen. Roll the material between your palms and the glass plate with just enough pressure to create a consistent-diameter thread all the way down as shown in Fig.4.11. Each stroke should gradually compress the thread until its diameter reaches 3.2 mm, which should take no more than 2 minutes. It is suggested that strike rates of 80 to 90 per minute should be used. A stroke is a single forwards and backwards movement of the hand from its starting position.

Fig.4.11: Performing plastic limit (DTU)

- When the thread reaches a diameter of 3.2 mm, cut it into several pieces. Squeeze the pieces together and knead with the thumbs and first fingers of both hands to form an ellipsoidal ball. Collect, knead, and re-roll the thread until it collapses under the rolling pressure and the soil can no longer be formed into a 3.2-mm thread.
- Collect the fractured thread fragments in a container with a known weight.
- Repeat this process with another 1.5 to 2.0 g of soil from the plastic limit specimen until the container is full.
- Repeat this process to make another container with at least 6 g of soil.
- Determine the amount of water in the soil in the containers.

Calculation:

The average of the two water contents is rounded to the nearest whole number. This result is the plastic limit, (PL) unless the difference between the two trail plastic limits is less than one, then the test should be repeated. Repeat the test if the difference between the two trial plastic limitations exceeds the allowable range of 1.4% points for two results.

4.3 COMPACTION TEST

This portion of the research is utilized to demonstrate the compaction characteristics of expansive soil blended in a mixture of groundnut shell ash and onion peel powder concentrations. Compaction testing aids in determining the dry density of soil and moisture content.

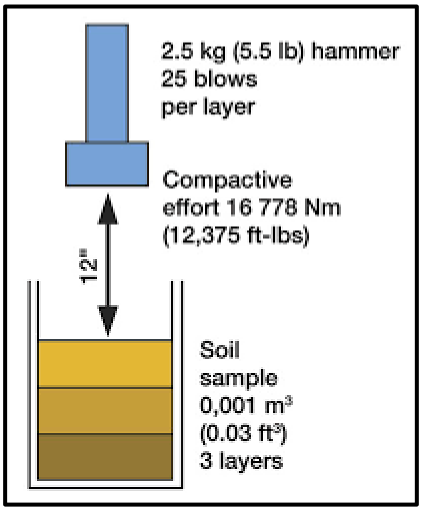

Fig.4.12: Compaction test apparatus

Source-https://civilblog.org/wp-content/uploads/2013/05/Testing-Standard-Proctor.jpg

Fig.4.13: Performing Proctor Test in Lab (DTU)

Test procedure:

The steps in the Proctor Compaction Test process are as follows:

- Collect approximately 3.5 kg of soil.
- Weight the mould and soil mass (W_m) without the collar.
- Pour the soil into the mixer and gradually add water until it reaches the desired moisture level (w).
- Lubricate the collar with lubricant.
- Remove the soil from the mixer and put it in the mould in three or five layers, depending on the method employed (Standard Proctor or Modified Proctor). To begin the compaction process, blow each layer 25 times. Whether automatically or manually, drops are applied at a steady rate. The soil mass should completely fill the mould and extend 1 cm above the collar.
- Remove the collar with care and use a sharpened straight edge to trim the soil that rises above the mould.
- Weight the mould and the soil in which it is spreading (W).
- The soil is extruded from the mould using a metallic extruder, which must be lined up with the mould.
- Take water readings from the bottom, middle and top of the sample.
- To produce higher water content, add water to the soil in the mixer.
- The bulk and dry density of expansive soil can be calculated using the formula below.
- Bulk density of soil, $\gamma = \frac{M}{V}$(gm/cc)

Dry Density of soil, $\gamma_d = \gamma / (1+w)$

Where,

γ = Bulk density of soil (gm/cc)

γ_d = Dry density of soil (gm/cc)

M = Mass of wet compacted mould

V = Volume of the mould (1000 cc)

w = Moisture content present in soil

4.4 CALIFORNIA BEARING RATIO (CBR TEST)

CBR is the force per unit area required to penetrate a soil mass at 1.25 mm/min with a 50 mm diameter circular plunger compared to the force required to pierce a standard material at the same rate. The ratio is commonly estimated for penetrations of 2.5 and 5 mm. It's utilized when the 5mm ratio is consistently higher than the 2.5mm ratio. The Table: 4.3 shows the standard loads for various penetrations for standard material with a C.B.R. value of 100%.

Table: 4.3 Standard Loads

Penetration of plunger (mm)	Standard load (Kg)
2.5	1370
5	2050

Fig.4.14: CBR Apparatus (DTU)

Apparatus:

- **CBR Apparatus:** A loading machine having a capacity of at least 5000 kg and base and a movable head that allows a 50 mm plunger to pierce the specimen at a rate of 1.25mm/min.
- **Cylindrical Mould:** Height is 175mm and the interior diameter is 150mm, with a detachable perforated base plate of 10mm thickness and 235mm dia.
- **Collar:** An extension collar with a height of 60 mm that can be removed.
- **Spacer Disc:** It has a height of 47.7 mm and diameter of 148 mm, including the handle.

Procedure:

- Place the mould assembly with the test specimen on the lower plate of the penetration testing machine. To prevent soil upheaval into the hole of the surcharge weights, a 2.5 kg annular weight should be placed on the soil surface before placing the rest of the surcharge weights and seating the penetration plunger
- To ensure that the penetration piston makes full contact with the sample, seat it at the centre of the specimen with the smallest possible load, but not more than 4 kg.
- Set the load gauge and deformation gauge to zero. Apply enough pressure to the piston that it penetrates at a rate of 1.25 mm/min.
- Take load values at 0.5, 1.0, 1.5, 2.0, 2.5, 4.0, 5.0, 7.5, 10, and 12.5 mm penetrations.
- To separate the mould from the loading equipment, lift the plunger. Determine the moisture content of around 20 to 50 g of soil from the top 30 mm layer.

Calculation: If the curve's initial segment is concave upwards, change the origin and draw a tangent to it at the point of maximum slope. Find and record the proper load reading for each penetration.

$$\textbf{C.B.R.} = (\mathbf{P_T}/\mathbf{P_S}) \times \mathbf{100}$$

Where:

$\mathbf{P_T}$= Corrected test load based on the chosen penetration of the load penetration curve.

$\mathbf{P_S}$= Standard load calculated from the table above for the same penetration.

C.B.R. values are commonly estimated for penetration depths of 5 mm and 2.5 mm. The C.B.R. value at 2.5 mm is usually always higher than at 5 mm penetration, hence the former will be used for design purposes. The test should be repeated if the C.B.R. for 5 mm is higher than that for 2.5 mm. If the findings are the same, design for 5 mm penetration using the C.B.R.

4.5 UNCONFINED COMPRESSIVE STRENGTH

The Unconfined Compression Test is also known as the Uniaxial Compression Test. The Unconfined Compression Test is a laboratory test used to measure the Unconfirmed Compressive Strength of a rock specimen (UCS). Unconfined compressive strength is the maximum axial compressive stress that a specimen may withstand without restricting strain. Because the stress is delivered along the longitudinal axis

Fig.4.15: Performing UCS test

Fig.4.16: Failure of sample

Procedure:

- The soil specimen is formed in the big mould at the required density and water content.
- The soil-filled sample tube is removed and the sampling tube is inserted into the large mould.
- A suitable procedure is used to soak the soil sample in the sampling tube.
- A layer of grease is applied on the split mould. The mould has been weighed.
- The sample is taken from the sampling tube and inserted in the split mould using an appropriate procedure using the sample extractor and knife.
- In the split mould, the specimen's two ends are clipped off.
- In the mould, the specimen is weighed.
- The specimen is extracted after splitting the split mould into two halves.
- Vernier callipers are used to measure the specimen's length and diameter.

- The specimen is put on the compression machine's bottom plate.
- The specimen is brought into touch with the upper plate.
- Set the dial gauges and the proving ring gauge to zero.
- The compression load is delivered at rate of 0.5 to 2% per minute to create axial strain.
- The dial gauge reading is recorded every 60 seconds for a strain of 6 percent to 12 percent, and the proving ring is recorded every 2 minutes or so beyond 12 percent.
- The test is repeated until failure surfaces are clearly visible or an axial strain of 20% is achieved.
- If feasible, measure the angle between the failure surface and the horizontal.
- For the water content determination, a sample from the specimen's failure zone is collected.

CHAPTER 5

RESULTS AND DISCUSSION

5.1 ATTERBERG TEST LIMITS (LL, PL & PI)

Table 5.4: Variation in groundnut shell ash and onion peel powder to determine LL, PL, and PI

Soil + % of O.P.P. + % of G.S.A.	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
0	47.9	16.55	31.35
Soil + 10% O.P.P. + 2% G.S.A.	46	14.3	31.7
Soil + 20% O.P.P.+ 4% G.S.A.	58	25.5	32.5
Soil + 30% O.P.P. + 6% G.S.A.	49	22	27
Soil + 40% O.P.P. + 8% G.S.A.	43	18	25

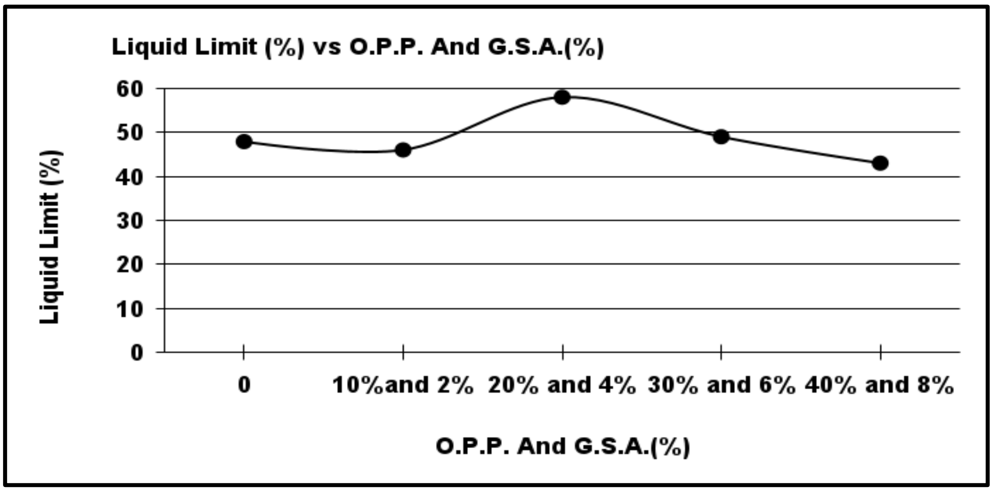

Fig: 5.17 Variation in OPP and GSA to determine LL

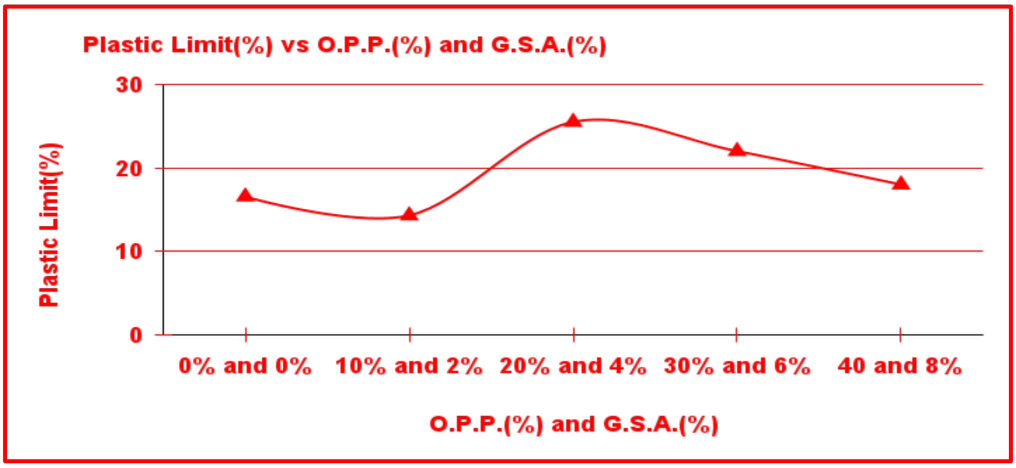

Fig: 5.18 Variation in OPP and GSA to determine PL

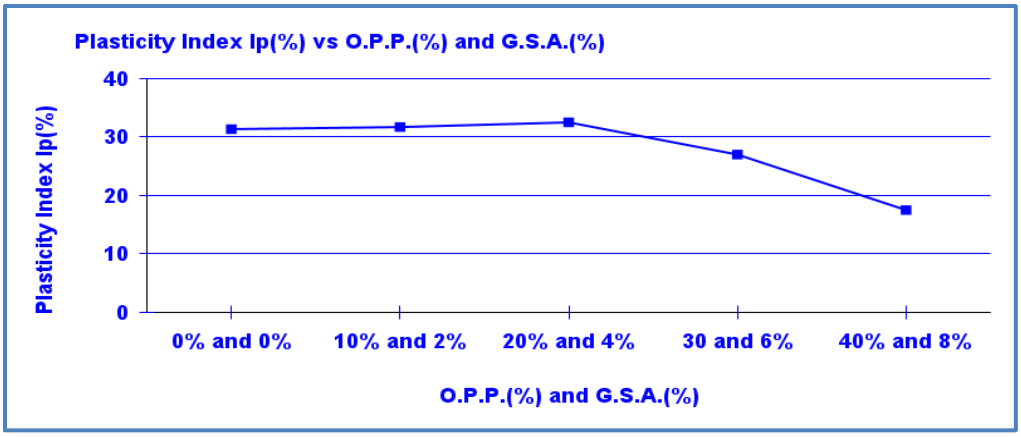

Fig: 5.19 Variation in OPP and GSA to determine PI

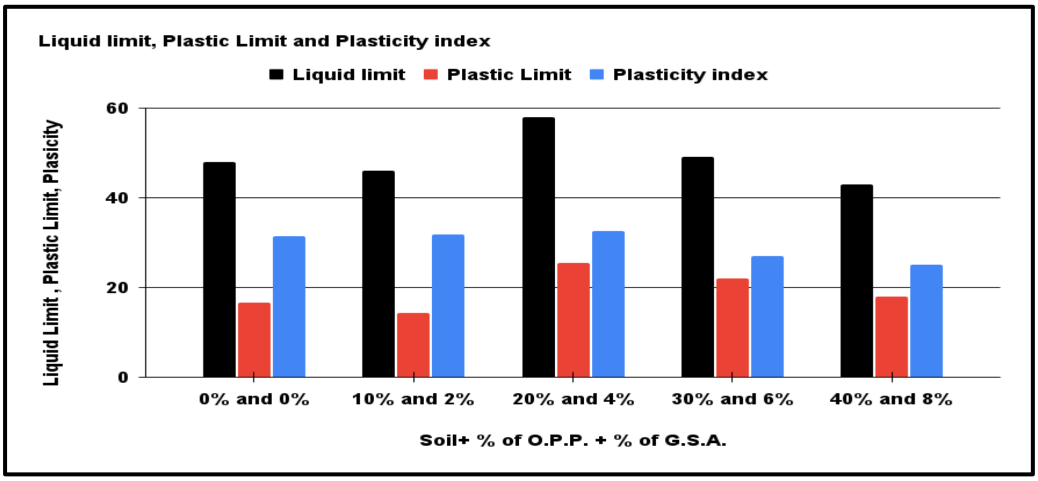

Fig: 5.20 Variations in LL, PL and PI

Calculation:

Liquid Limit (LL) = 47.9%, Plastic Limit (PL) = 16.55%,

Plasticity Index (PI) = (LL – PL) = (47.9-16.55) % = 31.35%

5.2 COMPACTION TESTS

Table 5.5: Variation in Compaction Characteristics

Soil + % of O.P.P. + % of G.S.A.	Water Content (%)	Dry Density (KN/m^3)
0	13	11.4
	16	11.6
	18	11.7
	23	12.4
	25	12.1
	28	11.8
Soil + 10% O.P.P. + 2% G.S.A.	16	11.02
	20	11.5
	27	12.5
	35	13.4
	35.5	12.3
	40.5	11.7
Soil + 20% O.P.P. + 4% G.S.A.	19	11.8
	24	11.2
	32	12.6
	28	11.4
	27	9.9
Soil +30% O.P.P. + 6% G.S.A.	21	11.3
	23	11.5
	24	11.8
	27	12.4
	24	11.9
	22	11.1
Soil + 40% O.P.P. + 8% G.S.A.	16	11.5
	19	12
	22	13
	27	11.5
	30	10.5

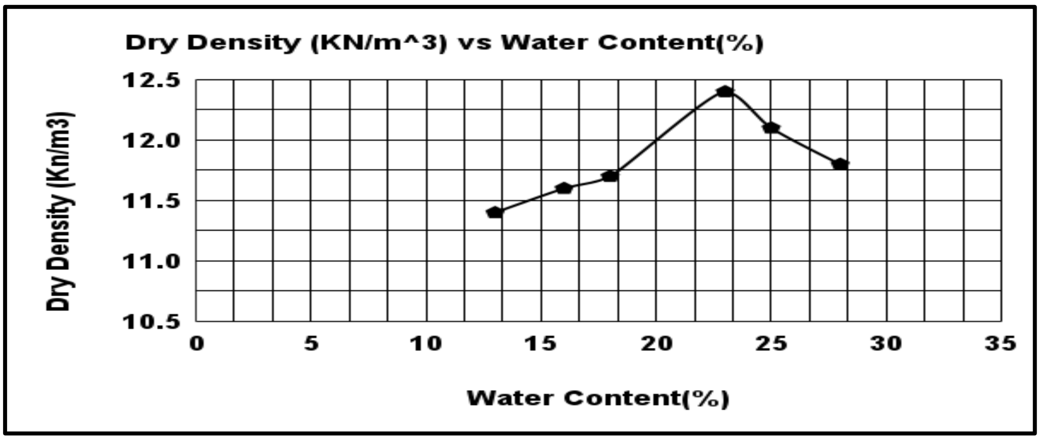

Fig: 5.21 0% O.P.P. +0% G.S.A.

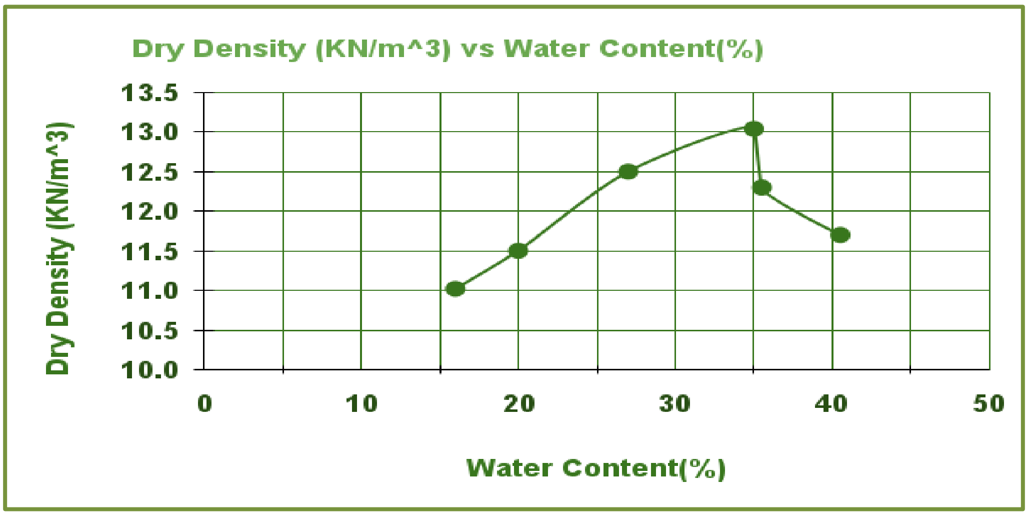

Fig: 5.22 10% O.P.P. +2% G.S.A.

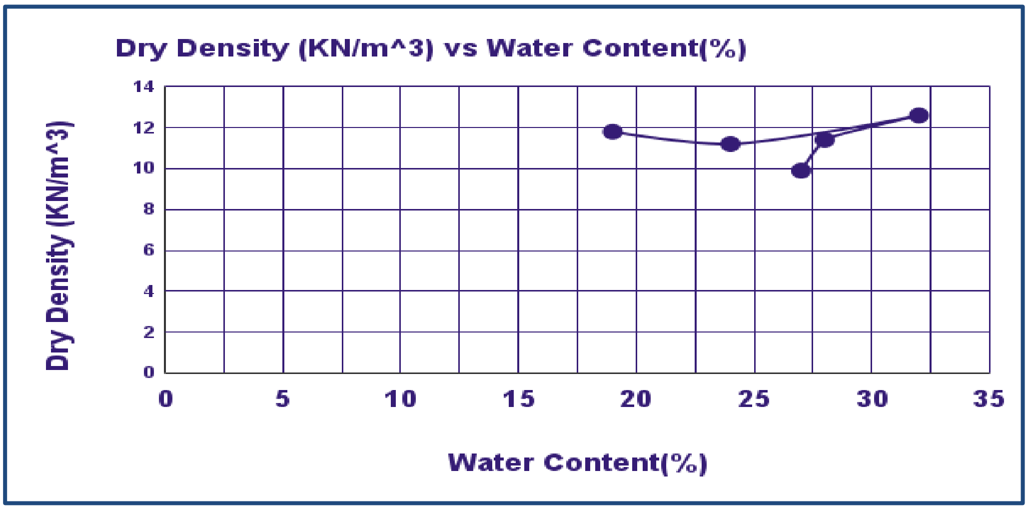

Fig: 5.23 20% O.P.P. +4% G.S.A.

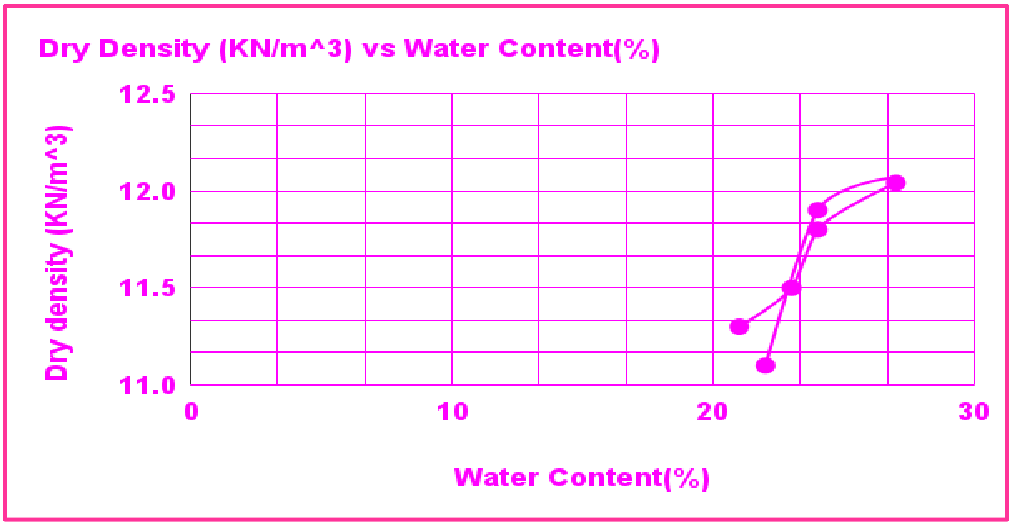

Fig: 5.24 30%O.P.P. +6% G.S.A.

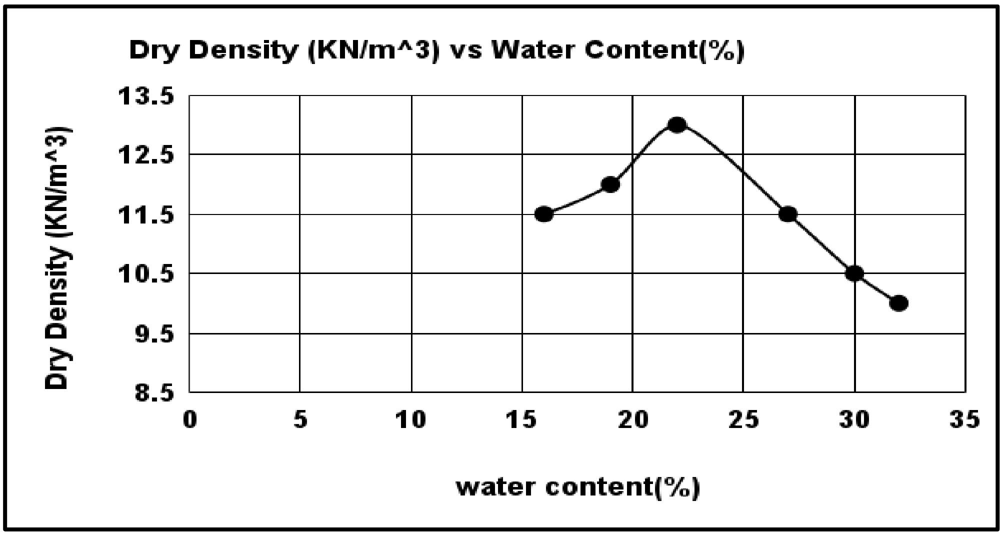

Fig: 5.25 40% O.P.P+8% G.S.A.

Table: 5.6 Variation in Compaction Characteristics

Soil + % of O.P.P.+ % of G.S.A.	OMC (%)	MDD (KN/m^3)
0	23	12.4
Soil + 10% O.P.P. + 2% G.S.A.	35	13.4
Soil + 20% O.P.P.+ 4% G.S.A.	32	12.6
Soil + 30% O.P.P. + 6% G.S.A.	27	12.04
Soil + 40% O.P.P. + 8% G.S.A.	22	13

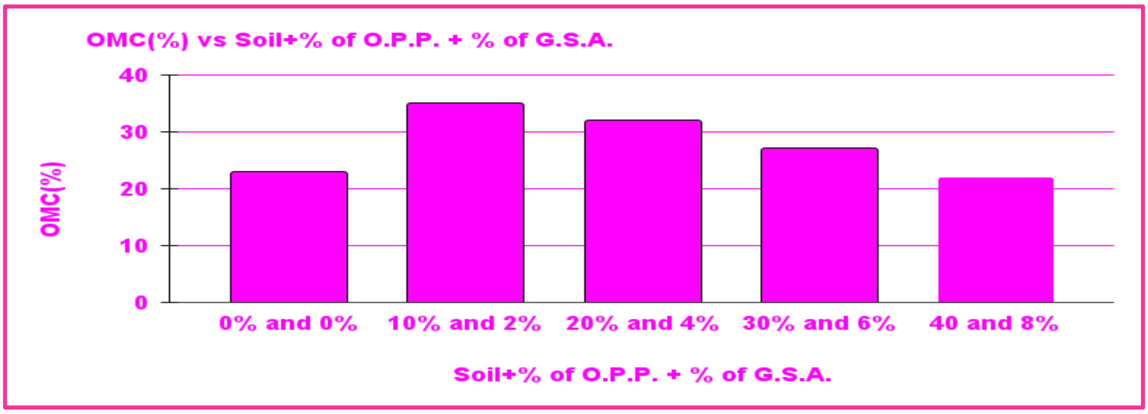

Fig: 5.26 Variations in O.P.P. and G.S.A. (O.M.C.)

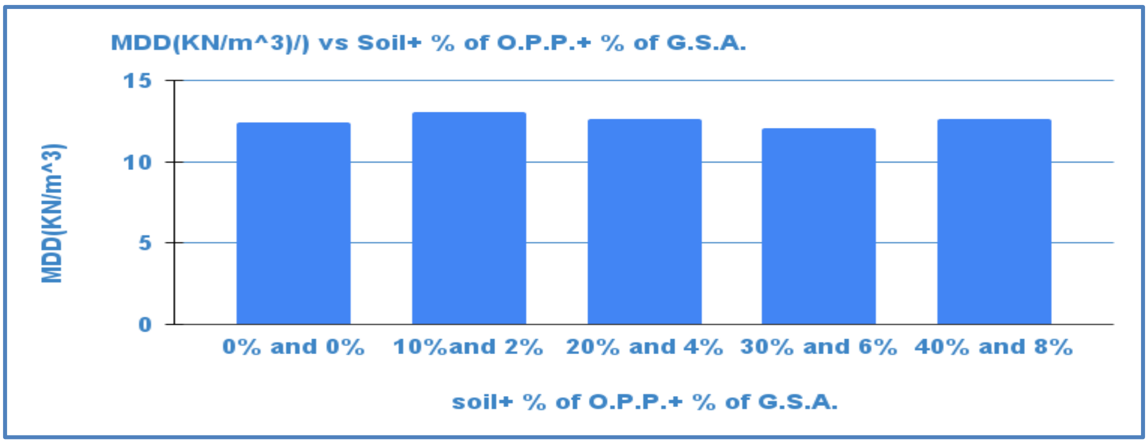

Fig: 5.27 Variations in O.P.P. and G.S.A. (MDD)

5.3 UNCONFINED COMPRESSION STRENGTH

Table: 5.7 Unconfined compressive strength at soil + 0% O.P.P. + 0%G.S.A.

Sample details:

D=4cm, H or L=8cm=80mm

A_o=12.56cm^2

Proving Ring Factor=0.247

Elapsed Time(Min.)	Compressive Dial Gauge Reading (No. of Div.) L_c=0.01	Deformation(mm) ΔL	Strain $\epsilon = \frac{\Delta L}{L}$	Corrected Area (cm^2)	Proving Ring Reading 1Div.=1Kg	Axial load(P) kg	Compressive Stress ($\frac{Kg}{cm^2}$)	Compressive Stress $\frac{KN}{(m^2}$)
0	0	0	0.0000	12.56	0	0	0	0
0.5	21	0.21	0.0026	12.59	2.4	2.964	0.235	23.08
1	38	0.38	0.0048	12.62	2.9	3.582	0.284	27.83
1.5	58	0.58	0.0073	12.65	3.3	4.076	0.322	31.59
2	88	0.88	0.0110	12.70	4	4.940	0.389	38.15
2.5	95	0.95	0.0119	12.71	4.5	5.558	0.437	42.88
3	112	1.12	0.0140	12.74	4.9	6.052	0.475	46.59
3.5	150	1.5	0.0188	12.80	5.6	6.916	0.540	52.99
4	175	1.75	0.0219	12.84	6	7.410	0.577	56.59
4.5	185	1.85	0.0231	12.86	6.6	8.151	0.634	62.17
5	200	2	0.0250	12.88	7.2	8.892	0.690	67.69
5.5	230	2.3	0.0288	12.93	7.6	9.386	0.726	71.18
6	250	2.5	0.0313	12.97	8.5	10.498	0.810	79.40
6.5	265	2.65	0.0331	12.99	8.9	10.992	0.846	82.98
7	275	2.75	0.0344	13.01	8.7	10.745	0.826	81.01

Table5.8: Unconfined Compressive strength at soil + 10% OPP+ 2%GSA

Sample details:

D=4cm, H or L=8cm=80mm

A_o=12.56cm^2

Proving ring factor=0.247

Elapsed Time(Min.)	Compressive Dial Gauge Reading (No. of Div.) L_c=0.01	Deformation(mm) ΔL	Strain $\epsilon = \frac{\Delta L}{L}$	Corrected Area (cm^2)	Proving Ring Reading 1Div.=1Kg	Axial load(P) kg	Compressive Stress ($\frac{Kg}{cm^2}$)	Compressive Stress $\frac{KN}{(m^2)}$)
0	0	0	0	12.56	0	0	0	0
0.5	25	0.25	0.0031	12.60	2.9	3.582	0.284	27.876
1	40	0.4	0.0050	12.62	3.2	3.952	0.313	30.702
1.5	65	0.65	0.0081	12.66	3.9	4.817	0.380	37.301
2	95	0.95	0.0119	12.71	4.5	5.558	0.437	42.877
2.5	102	1.02	0.0128	12.72	5.5	6.793	0.534	52.358
3	116	1.16	0.0145	12.74	6.2	7.657	0.601	58.917
3.5	160	1.6	0.0200	12.82	7.8	9.633	0.752	73.708
4	185	1.85	0.0231	12.86	8.9	10.992	0.855	83.835
4.5	200	2	0.0250	12.88	9.5	11.733	0.911	89.315
5	240	2.4	0.0300	12.95	10.2	12.597	0.973	95.404
5.5	260	2.6	0.0325	12.98	11.9	14.697	1.132	111.018
6	300	3	0.0375	13.05	12.5	15.438	1.183	116.013
6.5	320	3.2	0.0400	13.08	13.6	16.796	1.284	125.894
7	360	3.6	0.0450	13.15	11.6	14.326	1.089	106.821

Table5.9: Unconfined compressive strength at soil + 20% OPP+ 4%GSA

Sample details:

D=4cm, H or L=8cm=80mm

A_o=12.56cm^2

Proving ring factor=0.247

Elapsed Time(Min.)	Compressive Dial Gauge Reading (No. of Div.) L_c=0.01	Deformation(mm) ΔL	Strain $\epsilon = \frac{\Delta L}{L}$	Corrected Area (cm^2)	Proving Ring Reading 1Div.=1Kg	Axial load(P) kg	Compressive Stress ($\frac{Kg}{cm^2}$)	Compressive Stress $\frac{KN}{(m^2}$)
0	0	0	0	12.56	0	0	0	0
0.5	30	0.3	0.0038	12.607	3.9	4.817	0.382	37.465
1	47	0.47	0.0059	12.634	4.5	5.558	0.440	43.137
1.5	72	0.72	0.0090	12.674	5.2	6.422	0.507	49.690
2	99	0.99	0.0124	12.717	6.2	7.657	0.602	59.045
2.5	108	1.08	0.0135	12.732	7.5	9.263	0.728	71.343
3	122	1.22	0.0153	12.755	8.4	10.374	0.813	79.763
3.5	170	1.7	0.0213	12.833	9.5	11.733	0.914	89.658
4	199	1.99	0.0249	12.880	10.6	13.091	1.016	99.669
4.5	210	2.1	0.0263	12.899	12.9	15.932	1.235	121.125
5	260	2.6	0.0325	12.982	14.6	18.031	1.389	136.207
5.5	280	2.8	0.0350	13.016	16.8	20.748	1.594	156.326
6	320	3.2	0.0400	13.083	18.4	22.724	1.737	170.328
6.5	340	3.4	0.0425	13.117	20.4	25.194	1.921	188.350
7	360	3.6	0.0450	13.152	15.6	19.266	1.465	143.656

Table: 5.10 Unconfined compressive strength at soil + 30% O.P.P. + 6%G.S.A.

Sample details:

D=4cm, H or L=8cm=80mm

A_o**=12.56cm^2**

Proving ring factor=0.247

Elapsed Time(Min.)	Compressive Dial Gauge Reading (No. of Div.) L_c=0.01	Deformation(mm) ΔL	Strain $\epsilon = \frac{\Delta L}{L}$	Corrected Area (cm^2)	Proving Ring Reading 1Div.=1Kg	Axial load(P) kg	Compressive Stress ($\frac{Kg}{cm^2}$)	Compressive Stress $\frac{KN}{(m^2)}$
0	0	0	0	12.560	0	0	0.000	0
0.5	30	0.3	0.0038	12.607	2.9	3.582	0.284	27.859
1	47	0.47	0.0059	12.634	4.1	5.064	0.401	39.303
1.5	72	0.72	0.0090	12.674	5	6.175	0.487	47.779
2	99	0.99	0.0124	12.717	6	7.410	0.583	57.140
2.5	108	1.08	0.0135	12.732	7.1	8.769	0.689	67.538
3	122	1.22	0.0153	12.755	7.9	9.757	0.765	75.015
3.5	170	1.7	0.0213	12.833	8.7	10.745	0.837	82.108
4	199	1.99	0.0249	12.880	9.2	11.362	0.882	86.506
4.5	210	2.1	0.0263	12.899	10.2	12.597	0.977	95.773
5	260	2.6	0.0325	12.982	11.8	14.573	1.123	110.085
5.5	280	2.8	0.0350	13.016	13.5	16.673	1.281	125.619
6	320	3.2	0.0400	13.083	15.7	19.390	1.482	145.334
6.5	340	3.4	0.0425	13.117	16.1	19.884	1.516	148.648
7	360	3.6	0.0450	13.152	16.9	20.872	1.587	155.627
7.5	375	3.75	0.0469	13.178	17.1	21.119	1.603	157.160
8	399	3.99	0.0499	13.219	17.8	21.983	1.663	163.078
8.5	410	4.1	0.0513	13.238	17	20.995	1.586	155.524

Table: 5.11 Unconfined Compressive strength at soil + 40% OPP+ 8%GSA

Sample details:

D=4cm, H or L=8cm=80mm

A_o=12.56cm^2

Proving ring factor=0.247

Elapsed Time(Min.)	Compressive Dial Gauge Reading (No. of Div.) L_c=0.01	Deformation(mm) ΔL	Strain $\epsilon = \frac{\Delta L}{L}$	Corrected Area (cm^2)	Proving Ring Reading 1Div.=1Kg	Axial load(P) kg	Compressive Stress ($\frac{Kg}{cm^2}$)	Compressive Stress $\frac{KN}{(m^2)}$)
0	0	0	0	12.560	0	0	0	0
0.5	38	0.38	0.0048	12.620	1.5	1.853	0.147	14.395
1	57	0.57	0.0071	12.650	1.9	2.347	0.185	18.190
1.5	85	0.85	0.0106	12.695	2.5	3.088	0.243	23.850
2	110	1.1	0.0138	12.735	2.8	3.458	0.272	26.628
2.5	120	1.2	0.0150	12.751	5.5	6.793	0.533	52.239
3	132	1.32	0.0165	12.771	5.9	7.287	0.571	55.953
3.5	180	1.8	0.0225	12.849	7.6	9.386	0.730	71.635
4	200	2	0.0250	12.882	8.5	10.498	0.815	79.913
4.5	215	2.15	0.0269	12.907	8.9	10.992	0.852	83.513
5	250	2.5	0.0313	12.965	10.2	12.597	0.972	95.281
5.5	290	2.9	0.0363	13.032	12.5	15.438	1.185	116.164
6	325	3.25	0.0406	13.092	13.9	17.167	1.311	128.588
6.5	345	3.45	0.0431	13.126	14.8	18.278	1.392	136.557
7	375	3.75	0.0469	13.178	13.4	16.549	1.256	123.155
7.5	398	3.98	0.0498	13.218	13.9	17.167	1.299	127.365
8	450	4.5	0.0563	13.309	14	17.290	1.299	127.403
8.5	480	4.8	0.0600	13.362	14.5	17.908	1.340	131.429
9	510	5.1	0.0638	13.415	14.9	18.402	1.372	134.516
10	230	2.3	0.0288	12.932	12.9	15.932	1.232	120.814

Table: 5.12 Variation in unconfined compression strength (UCS)

Soil + % of O.P.P.+ % of G.S.A.	UCS (KN/m^2)
0	82.98
Soil + 10% O.P.P. + 2% G.S.A.	125.894
Soil + 20% O.P.P. + 4% G.S.A.	188.350
Soil + 30% O.P.P.+ 6% G.S.A.	163.078
Soil + 40% O.P.P. + 8% G.S.A.	134.516

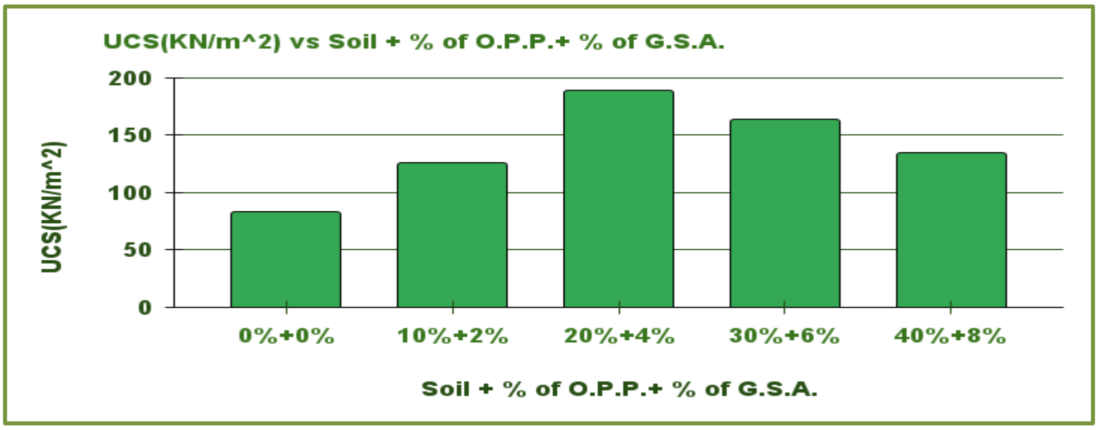

Fig: 5.28 variations in UCS

5.4 CALIFORNIA BEARING RATIO (CBR TEST)

Table: 5.13 Soils + 0% of O.P.P. + 0% of G.S.A.

Area of plunger = 19.63cm^2, Diameter of Plunger = 50mm, Proving Ring factor = 0.247

Penetration(mm)	Proving Ring Reading(A) 1Div.=1Kg	Load Intensity = $\frac{(A)\times 0.247}{19.63}$ (Kg/cm^2)	Standard load Intensity(Kg/cm^2)	CBR (%)
0	0	0		
0.5	55	0.692		
1	62	0.780		
1.5	76	0.956		
2	89	1.120		
2.5	120	1.510	70	2.16
3	128	1.610		
4	135	1.699		
5	170	2.139	105	2.04
7.5	220	2.768		
10	260	3.272		
12.5	294	3.699		

CBR at 5 mm penetration=2.04%

CBR at 2.5mm penetration=2.16%

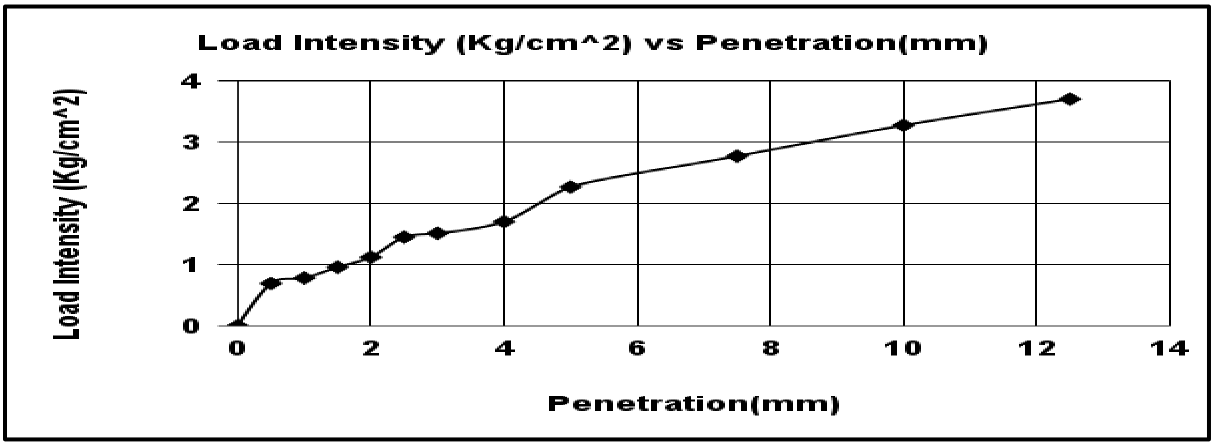

Fig: 5.29 Variations in CBR (Penetration vs. Load Intensity)

Table: 5.14 soil + 10% of O.P.P. + 2% Of G.S.A.

Area of plunger = 19.63cm^2, Diameter of Plunger = 50mm, Proving Ring factor = 0.247

Penetration(mm)	Proving Ring Reading(A) 1Div.=1Kg	Load Intensity= $\frac{(A) \times 0.247}{19.63}$ (Kg/cm^2)	Standard load Intensity(Kg/cm^2)	CBR (%)
0	0	0		
0.5	67	0.843		
1	78	0.981		
1.5	97	1.221		
2	115	1.447		
2.5	157	1.975	70	2.82
3	165	2.076		
4	173	2.177		
5	210	2.642	105	2.51
7.5	270	3.397		
10	297	3.737		
12.5	310	3.901		

CBR at 5mm penetration=2.51%

CBR at 2.5mm penetration=2.82%

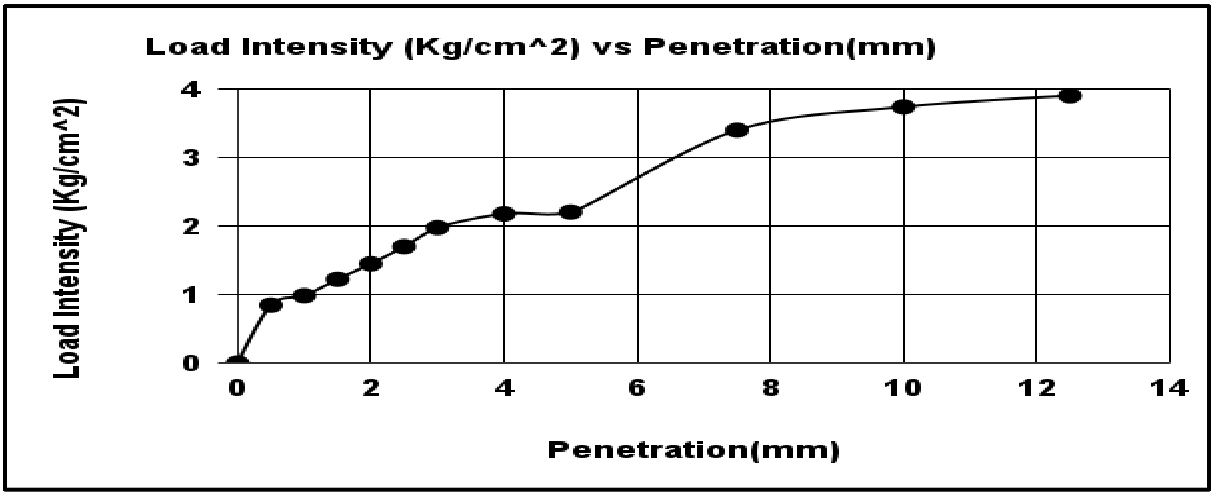

Fig: 5.30 Variations in CBR (Penetration vs. Load)

Table: 5.15 soil + 30% of O.P.P. + 6% Of G.S.A.

Area of plunger = 19.63cm^2, Diameter of Plunger = 50mm, Proving Ring factor = 0.247

Penetration(mm)	Proving Ring Reading(A) 1Div.=1Kg	Load Intensity= $\frac{(A)\times 0.247}{19.63}$ (Kg/cm^2)	Standard load Intensity(Kg/cm^2)	CBR (%)
0	0	0		
0.5	60	0.755		
1	95	1.195		
1.5	120	1.510		
2	180	2.265		
2.5	210	2.642	70	3.77
3	230	2.894		
4	255	3.209		
5	295	3.712	105	3.53
7.5	310	3.901		
10	340	4.278		
12.5	365	4.593		

CBR at 5mm penetration=3.53%

CBR at 2.5mm penetration=3.77%

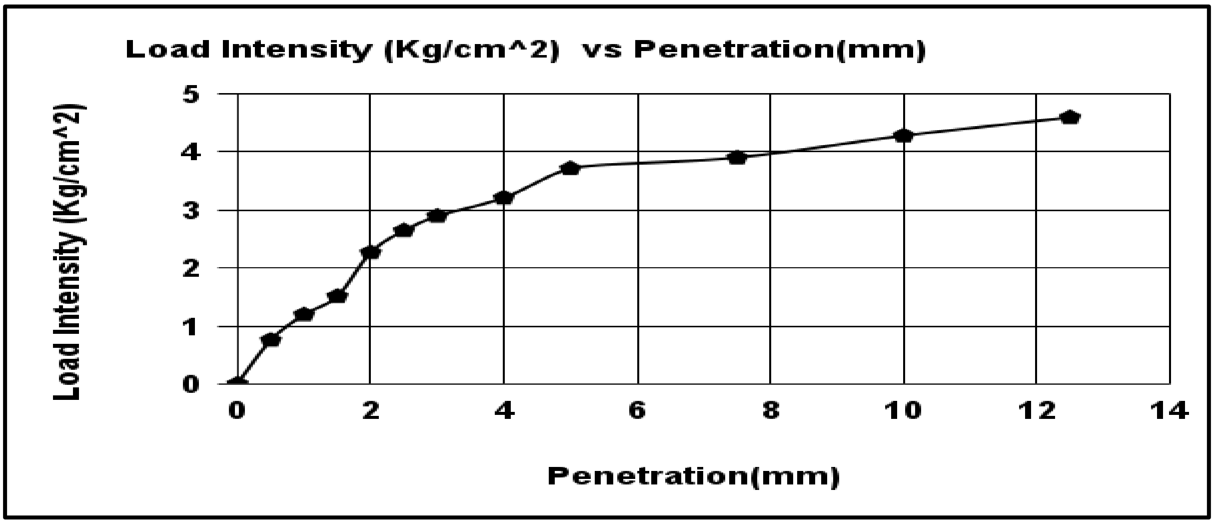

Fig: 5.31 Variations in CBR (Penetration vs. Load)

Table: 5.16 soil + 20% of O.P.P. + 4% of G.S.A.

Area of plunger = 19.63cm^2, Diameter of Plunger = 50mm, Proving Ring factor = 0.247

Penetration(mm)	Proving Ring Reading(A) 1Div.=1Kg	Load Intensity $=\frac{(A)\times 0.247}{19.63}$ (Kg/cm^2)	Standard load Intensity(Kg/cm^2)	CBR (%)
0	0	0		
0.5	65	0.818		
1	105	1.321		
1.5	150	1.887		
2	250	3.146		
2.5	360	4.530	70	6.47
3	385	4.844		
4	410	5.159		
5	499	6.279	105	5.98
7.5	520	6.543		
10	560	7.046		
12.5	600	7.550		

CBR at 5mm penetration=5.98%

CBR at 2.5mm penetration=6.47%

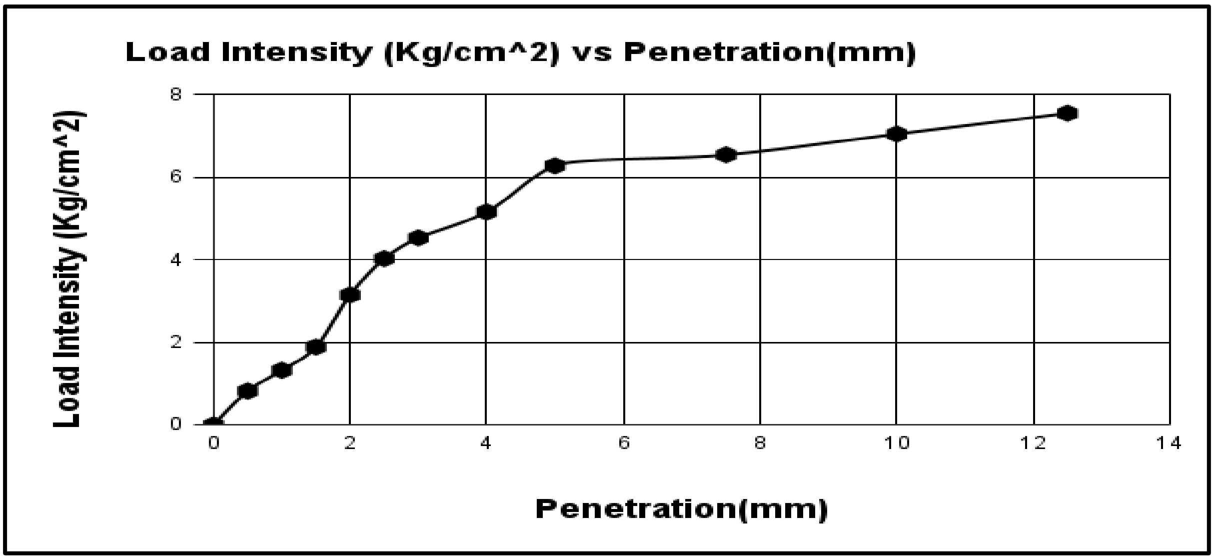

Fig: 5.32 Variations in CBR (Penetration vs. Load)

Table: 5.17 soil + 40% of O.P.P. + 8% of G.S.A.

Area of plunger = 19.63cm^2, Diameter of Plunger = 50mm, Proving Ring factor = 0.247

Penetration(mm)	Proving Ring Reading(A) 1Div.=1Kg	Load Intensity $=\frac{(A)\times 0.247}{19.63}$ (Kg/cm^2)	Standard load Intensity(Kg/cm^2)	CBR (%)
0	0	0		
0.5	55	0.692		
1	85	1.070		
1.5	98	1.233		
2	120	1.510		
2.5	185	2.328	70	3.32
3	191	2.403		
4	200	2.517		
5	250	3.146	105	2.99
7.5	275	3.460		
10	296	3.725		
12.5	310	3.901		

CBR at 5mm penetration=2.99%

CBR at 2.5mm penetration=3.32%

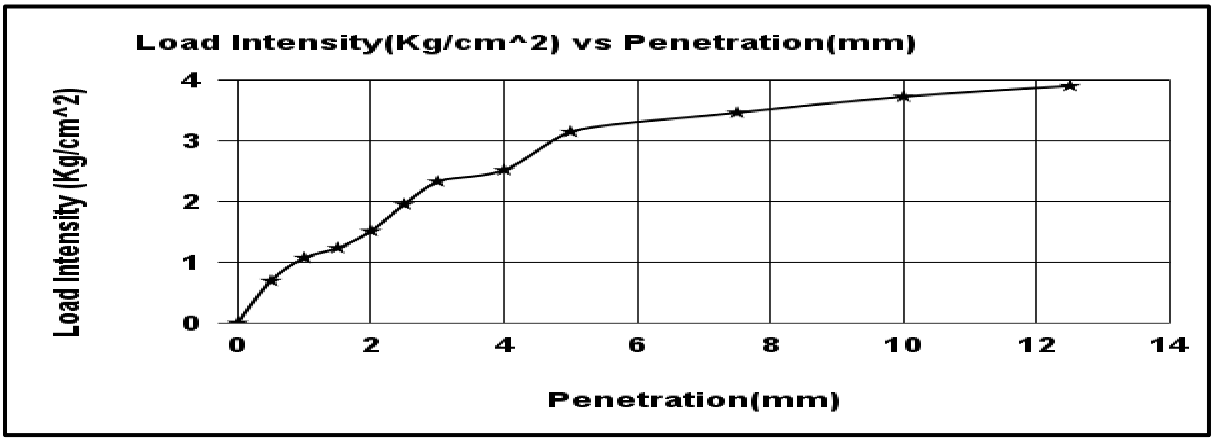

Fig: 5.33 Variations in CBR (Penetration vs. Load)

Table: 5.18 Variations in CBR

soil+% OPP +%GSA	CBR(%) at 2.5mm	CBR(%) at 5mm
0	2.16	2.04
10%+2%	2.82	2.51
20%+4%	6.47	5.98
30%+6%	3.77	3.53
40%+8%	3.32	2.99

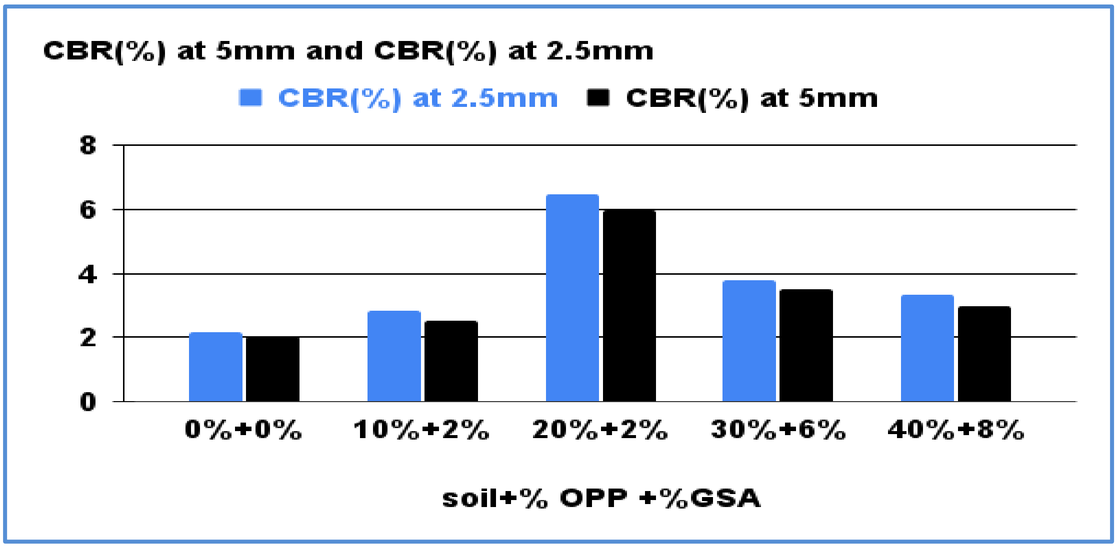

Fig: 5.34 CBR values at 2.5mm penetration and 5mm penetration

Results obtained from California Bearing Test (CBR) ratio with varying percentage Groundnut shell ash and onion peel powder is demonstrated to observe its effect on kaolinite clay. A concentration of 20% O.P.P. + 4% G.S.A. soil could provide a CBR of 6.47%.

Table: 5.19 Analysis of test results

Parameter	Soil + 0% O.P.P. + 0% G.S.A	Soil + 10% O.P.P.+ 2% G.S.A.	Soil+ 20% O.P.P.+4% G.S.A.	Soil + 30% O.P.P.+ 6% G.S.A.	Soil + 40% O.P.P.+ 8% G.S.A.
Liquid Limit (%)	47.9	46	58	49	43
Plastic Limit (%)	16.55	14.3	25.5	22	18
Plasticity Index (%)	31.35	31.7	32.5	27	25
OMC (%)	23	35	32	27	22
MDD (KN/ m^3)	12.4	13.05	12.6	13.04	13
CBR (%)	2.16	2.82	6.47	3.77	3.32
UCS (KN/m^2)	82.98	125.89	188.35	163.08	134.52

CHAPTER 6

CONCLUSION

The first conclusion reached after analyzing the data is that stabilizing soil using waste products in combination with onion peel powder and groundnut shell ash produced better results. They are also the most efficient source of soil amendment for improving geotechnical properties and effective strength.

The objective of this experiment was to stabilize the geotechnical qualities of expansive soil. The following observations can be obtained from this research:

- The liquid limit was found to decrease from 47.9% at 0% GSA and 0% OPP to 46% at (10 %OPP and 2% GSA). After then, the liquid limit increases by 12% at (20% OPP and 4 %GSA). And continuously decreasing beyond (30% OPP and 6% GSA). So optimum dosage for liquid limit is 58% at (20% OPP and 4 %GSA).

- At 10% OPP and 2% GSA content, the plastic limit decreases by 2.25%. With the increase in GSA and OPP content since then, the plastic limit has increased up to 20%OPP and 4% GSA and then decreases. So optimum dosage for plastic limit is 25.5% at (20% OPP and 4 %GSA).

- It was observed that the plasticity index increased from 31.35 % to 32.5 at 20% OPP and 4 % GSA. With the addition of the OPP and GSA content, it gradually decreases. The soil's plasticity index indicates its workability; lower the plasticity, the more workable the soil.

- At 10% OPP and 2% GSA, the highest variation in the OMC and MDD was seen, indicating that this was the optimum dosage.

- The CBR test values for various percentages of onion peel powder and groundnut shell ash were observed to increase from 2.16% at 0% OPP and 0% GSA to 6.47% at 20% OPP and 4% GSA, then decrease at 30% OPP and 4% GSA. It was found that 20% OPP and 4% GSA is the optimum dosage for CBR.

- The compressive stress increases as the onion peel powder and groundnut shell ash content increases, according to the UCS test results for varied percentages of OPP and GSA. At 30% OPP and 6% GSA, the compressive stress reached its maximum.

- According to the tests results, we can say that the optimal percentage of OPP and GSA is 20% of OPP and 4% of GSA.

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