A PROJECT REPORT

ON

# STRENGTH AND STIFFNESS RESPONSE OF SOIL REINFORCED WITH JUTE FIBER

UNDER GUIDANCE OF

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BY:

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## CERTIFICATE

I hereby declare that the work presented in the project entitled "Strength and Stiffness response of Chambaghat soil reinforced with jute fibre" submitted towards the completion of the project in eighth semester at Jaypee University of Information and Technology, Waknaghat, is an authentic record of my original work carried out under the guidance of Mr. Saurabh Rawat, Assistant Professor, Department Of Civil Engineering, JUIT, Waknaghat.

I have not submitted the matter embodied in this project for the award of any other degree.

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# ACKNOWLEDGEMENT

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We also take this opportunity to express a deep sense of gratitude to Dr Ashok Kumar Gupta, Head of Department of Civil Engineering JUIT, Waknaghat for his cordial support valuable information and guidance which helped us in completing the Task through various stages.

# INDEX

# 1.ABSTRACT

The main objective of this study is to investigate the use of waste fiber materials in geotechnical applications and to evaluate the effects of waste PLASTIC AND JUTE fibers on shear strength of unsaturated soil by carrying out direct shear tests and unconfined compression tests on two different soil samples. The results obtained are compared for the two samples and inferences are drawn towards the usability and effectiveness of fiber reinforcement, as a cost effective approach. For any land-based structure, the foundation is very important and has to be strong to support the entire structure. In order for the foundation to be strong, the soil around it plays a very critical role. So, to work with soils, we need to have proper knowledge about their properties and factors which affect their behavior. The process of soil stabilization helps to achieve the required properties in a soil needed for the construction work.

Keywords: *Plastic and jute waste, cost effective approach, shear strength, soil stabilization, fiber reinforcement.*

# 2.INTRODUCTION

Reinforced soil is a composite material which is formed by the association of frictional soil and tension resistant elements in the form of sheets, strips, nets or mats of metal, synthetic fabrics or fiber reinforced plastics and arranged in the soil mass in such a way as to reduce or suppress the tensile strain which might developed under gravity and boundary forces. The reinforcement in soil is placed more or less in the same way as steel in concrete and the end product is called reinforced soil. It is very effectively used for retaining structures, embankments, footings, sub grade etc. The incorporation of reinforcement in the earth mass, particularly in case of non-cohesive soils is not only for carrying the tensile stresses but instead meant for anisotropic suppression or reduction of one normal strain rate. Soil reinforcement technique with randomly distributed fiber is used in a variety of applications like, retaining structures, embankments, footings, pavement sub grade. During last 25 years, much work has been done on strength deformation behavior of fiber reinforced soil and it has been established beyond doubt that addition of fiber in soil improves the overall engineering performance of soil. Among the notable properties that improved are greater extensibility, small loss of post peak strength, isotropy in strength and absence of planes of weakness etc. Fiber reinforced soil has been used in many countries in the recent past and further research is in progress for many hidden aspects of it. Fiber reinforced soil is effective in all types of soils (i.e. sand, silt and clay). Use of natural material such as Jute, coir, sisal and bamboo, as reinforcing materials in soil is prevalent for a long time and they are abundantly used in many countries like India, Philippines, and Bangladesh etc. The main advantages of these materials are they are locallyavailable and are very cheap. They are biodegradable and hence do not create disposal problem in environment.

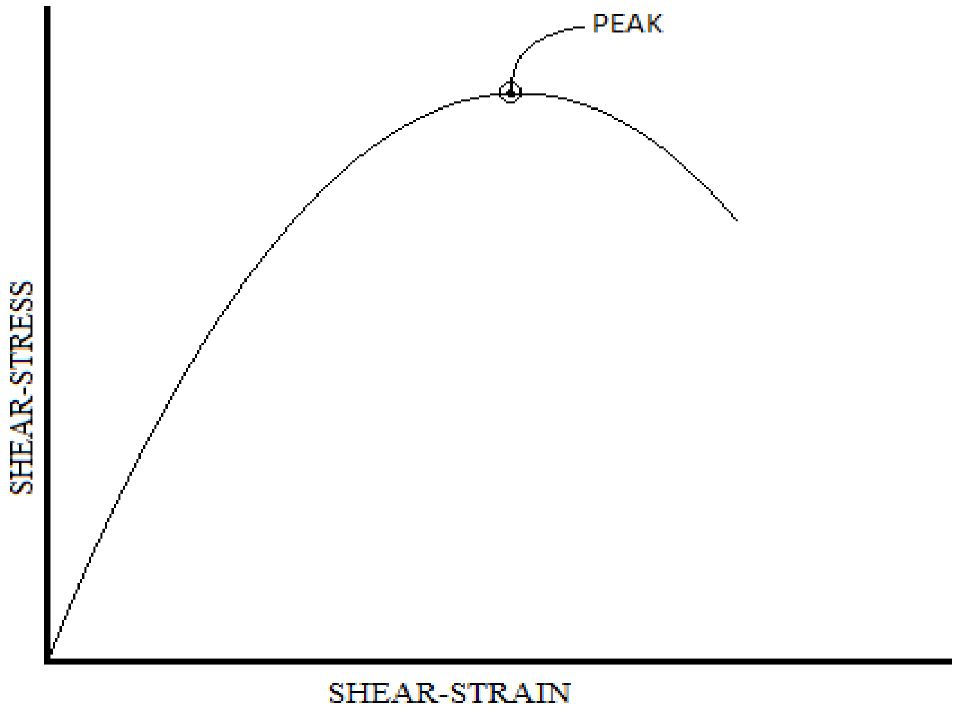
Shear strength is perhaps the most important property of soil. It represents the ability of soil to withstand shear stresses. Unlike normal stresses which, when they are compressive in nature tend to squeeze the soil. Shear stresses tend to displace in a particular direction, a portion of soil in relation to test. Knowledge of shear strength is necessary for solution of large number of problems that a consulting geotechnical engineer encounters. The stability of slopes, natural or man-made, the bearing capacity of foundations, the lateral pressure exerted by soil on retaining walls and similar structures are all dependent on shear strength of soil.

Shear strength is perhaps the most complex engineering property of soil. Unlike other civil engineering materials such as concrete and steel, the shear strength of a soil is not only a function of material but is a function of stresses applied to it, and of course the manner in which these stresses are applied. The shear strength of soil therefore can be tabulated in codes of practice since the same soil can exhibit markedly different shear strength under different field and engineering conditions.

Apart from inherent complexity, the subject of shear strength has had complexity thrust upon, since consulting geotechnical engineers, like other mortals, have mind sets and are often reluctant to revise beliefs when confronted with newer, conclusive but contrary evidence. To understand shear behavior of soil, it is useful to ignore complexity and to concentrate only on first principles so that we have some basic to hold on and to rely upon as we engage in the design process.

Practically soil consists of individual particles that can slide and roll relative to each other. Shear strength of a soil is equal to the maximum value of shear stress that can be mobilized within the soil mass without failure taking place. The failure may be in the form of sinking of footing or movement of wedge of soil behind a retaining wall forcing it to move out or slide in an earth embankment.

Shear strength is term used in soil mechanics to describe the magnitude of shear stress that a soil can sustain.




TYPICAL STRESS-STRAIN CURVE FOR A
DRAINED DILATANT SOIL

The term soil stabilization means the improvement of the stability or bearing power of the soil by the use of controlled compaction, proportioning and or the addition of suitable admixture or stabilizer. The basic principles in soil stabilization may be stated as follows:

1. Evaluating the properties of the given soil.
2. Deciding the method of supplementing the lacking property by the effective and economical method of stabilization.
3. Designing the stabilized soil mix for intended stability and durability values.
4. Considering the construction procedure by adequately compacting the stabilized layers.

For stabilizing soils we will use geosynthetics in our project :
The two main type of geosynthetics used are as follows:

## GEOTEXTILE

It is made of plastic threads that allow water to pass around them, but not very small particles of soil beneath them. Geotextiles separates and contains the base from the underlying soil sub grade . Jute is one such example of geo-textile which we will use in our project.

### GEOTEXTILES APPLICATION

1. Consolidation of soft soil.
2. Construction of roads.
3. Stabilization of embankments of roads & railways.
4. Preventing subsidence in roads & railway tracks.
5. Protection of all kinds of earthen slopes.
6. Control of erosion on river banks, canals.
7. Construction of concealed drains on hill roads.
8. Horticultural base in arid & semi-arid zones.
9. Watershed management and rain-water harvesting.

### TECHNICAL ADVANTAGES OF GEO TEXTILES

1. High Initial Strength.
2. High Roughness Co-efficient.
3. Low Extension at Break.
4. Best Drapability .
5. Good Spinnability .
6. Can be tailor-made to meet requirement.
7. Environmentally compatible.

### COMMERCIAL ADVANTAGES OF GEOTEXTILES

1. Easily Available
2. Competitive Pricing
3. Total Quality control in all stages from Crop to Fabric.
4. Long experience in production and machinery handling.

## GEOMEMBRANE

They are made from various types of polymers used to enhance, augment and make possible cost effective environmental, transportation and geotechnical engineering construction
projects . We are using plastic bags as geosynthetics .They are used to provide one or more of the following functions:

1. SEPERATION.
2. REINFORCEMENT..
3. DRAINAGE OR LIQUID BARRIER.

1. **SEPERATION:** It is achieved if the fabric prevents mixing of adjacent dissimilar soils which may occur during construction or may be caused by repeated external loading of a soil layer system Most fabrics can act as separators provided they have adequate strength.

2. **REINFORCEMENT:** Means the inclusion of the fabric to provide tensile strength, redistribution of stresses and / or confinement, thereby increasing the stability of a soil mass, reducing earth pressures, or decreasing deformation or susceptibility to cracking.

3. **DRAINAGE OR LIQUID BARRIER:** Collecting and redirecting seepage water within a soil mass or adjacent to retaining walls culverts and tunnel linings.

Ex - Non-woven fabrics or composites' have sufficient inflow capacity to fulfill this function
Geotextiles acts as a filter if it allows seepage from a water bearing layer while preventing most soil particles from being carried away by the water flow.

There are many techniques for soil stabilization, including compaction, dewatering and by adding material to the soil such as plastic bags and jute bags .

But the stabilization using waste plastic strips is an economic method since the stabilizer used here is waste plastic materials, which are easily and cheaply available. Jute is made of natural fibers of a bamboo-like plant and is rated as one of the most dependable and proven soil stabilization products available. Jute stabilizes surface soil on slopes with thousands of tiny check dams per square yard.

# 3.LITERATURE REVIEW:

In the recent years, several researchers are trying to develop solutions for the reuse of different types of wastes generated which has become one of the major challenges for the environmental issues in many countries. Wastes such as plastic waste tire shreds mixed with soil behave similar to fiber reinforced soils and several researchers presented technique of using discrete fibers to enhance the strength of soil. Most of them used different types of fibers as reinforcing materials, such as natural fibres , glass fibres, plastic fibres, polypropylene and polyester fibres. Experimental results reported by various researchers
(Shewbridge and Sitar, 1989, Maher & Gray, 1990; Maher and Ho, 1994, Li et al. 2001, Rao & Balan, 2000, Consoli et al., 2002, 2003, 2004, 2009, Sivakumar Babu & Vasudevan 2008 a, b; Sivakumar Babu & Chouksey 2010) showed that the fibre reinforced soil is a potential composite material which can be advantageously employed in improving the structural behaviour of soils. The tests were carried with different types of fibres in different proportions and the effects of fibre in improving strength and stability of soil were identified.

Shewbridge and Sitar (1989) conducted experiments with sand reinforced with fibres to observe the deformation pattern and to quantify the width of shear zone in sand. The results showed that deformation pattern of reinforcement was found curve- linear and symmetric about the centre of the shear zone. Maher and Gray (1990) carried out triaxial compression tests on sand reinforced with discrete, randomly distributed fibres and observed the influence of various fibre properties on soil behaviour. Using the experimental results they have proposed a force equilibrium model based on statistical analysis for randomly distributed discrete fibre reinforced sand. Maher and Ho (1994) reported that the fibre reinforcement increased the shear strength and ductility of clay. Li et al. (2001) carried out centrifuge model test to study the behaviour of fibre reinforced cohesive steep slope using poly propylene fibres. It was found that critical height of slope can be increased due to reinforcing. Consoli et al. (2002) carried out an experimental study of the utilization of the polyethylene fibres derived from plastic wastes in the reinforcement of uncemented and artificially cemented sand and showed that the plastic waste improved the stress strain response of uncemented and cemented sands. This is perhaps one of the earliest attempts advocating the
use of plastic waste. Consoli et al. (2003) proposed a field application for such materials designed for increasing the bearing capacity of spread foundations when placed on a layer of

fibre-reinforced cemented sand built over a weak residual soil stratum. Consoli et al. (2004) carried out triaxial compression test on cemented and uncemented sand reinforced with various types of fibres to study the effect of fibres on mode of failure, ultimate deviator stress, ductility and energy absorption capacity. They observed that the inclusion of fibres changed the mode of failure from brittle to ductile. Studies were also conducted on tire shreds as reinforcing material (Hataf and Rahimi 2006, Yoon et al. 2008). Hataf and Rahimi (2006) carried series of laboratory tests on the model of shallow footing resting on reinforced sand. Tire shreds were used as reinforcement elements. It was found that addition of 10% shreds by volume contributed to improvement of bearing capacity, expressed in terms of the bearing capacity ratio (BCR) in the range of 1.17 to 1.83 where as use of 50% tire shreds increased BCR to values in the range of 2.95 to 3.9 for different sizes of shreds. Yoon et al. (2008) presented a method for the reuse of waste tires called 'tirecell' for soil improvement. The results indicated that tirecell reinforced sand produced higher bearing capacities and lower settlements. Sivakumar Babu et al. (2007) presented the results based on numerical analysis of stress strain response of fibre reinforced sand. Numerical simulation results indicate that the presence of random reinforcing material in soils make the stress concentration more diffused and restricts the shear band formation. Numerical simulation results also indicate that pull-out resistance of fibres governs the stress strain response of random-reinforced soil. Sivakumar Babu and Vasudevan (2008a, 2008b and 2008c) presented comprehensive experimental results using compacted soil-fibre specimens, with coir fibres randomly distributed in the soil specimen. Experiments were carried out for various fibre parameters such as fibre content, fibre length and fibre diameter. Results showed that the improvement in strength and stiffness response, reduction in compression indices, reduction in swelling behaviour of soil. It is also observed that fibres reduce the seepage velocity of plain soil considerably and thus increase the piping resistance of soil. Based on critical state concepts, Sivakumar Babu and Sandeep Chouskey (2010) proposed a constitutive model to obtain stress strain response of coir fibre reinforced soil as a function of fibre content. The above literature review clearly indicates that studies are available on the use of wastes from plastic water bottles are limited. The soil mixed with plastic waste is expected to behave as a fibre reinforced soil. The patented procedures for the use of fibre-reinforced soil in the field are also available (Freed 1988)). To promote the recycling of plastic wastes on a large-scale in geotechnical applications where bulk utilization of waste materials is possible, work is carried out and presented in this report.

Soil stabilization is the process of altering some soil properties by different methods, mechanical or chemical in order to produce an improved soil material which has all the desired engineering properties .

Stated that Soils are generally stabilized to increase their strength and durability or to prevent erosion and dust formation in soils. The main aim is the creation of a soil material or system that will hold under the design use conditions and for the designed life of the engineering project. The properties of soil vary a great deal at different places or in certain cases even at one place; the success of soil stabilization depends on soil testing. Various methods are employed to stabilize soil and the method should be verified in the lab with the soil material before applying it on the field.

Principles of Soil Stabilization: (Singh, H.P., BAGRA, M. (2013) "Strength and Stiffness Response of Itanagar Soil Reinforced with Jute Fiber". )

• Evaluating the soil properties of the area under consideration.
• Deciding the property of soil which needs to be altered to get the design value and
choose the effective and economical method for stabilization.
• Designing the Stabilized soil mix sample and testing it in the lab for intended stability and durability values.

Soil properties vary a great deal and construction of structures depends a lot on the bearing capacity of the soil, hence, we need to stabilize the soil which makes it easier to predict the load bearing capacity of the soil and even improve the load bearing capacity. The gradation of the soil is also a very important property to keep in mind while working with soils. The soils may be well-graded which is desirable as it has less number of voids or uniformly graded which though sounds stable but has more voids. Thus, it is better to mix different types of soils together to improve the soil strength properties. It is very expensive to replace the inferior soil entirely soil and hence, soil stabilization is the thing to look for in these cases.

1. It improves the strength of the soil, thus, increasing the soil bearing capacity.
2. It is more economical both in terms of cost and energy to increase the bearing capacity of the soil rather than going for deep foundation or raft foundation.

3.It is also used to provide more stability to the soil in slopes or other such places.
4. Sometimes soil stabilization is also used to prevent soil erosion or formation of
dust, which is very useful especially in dry and arid weather.
5. Stabilization is also done for soil water-proofing; this prevents water from entering
into the soil and hence helps the soil from losing its strength.
6. It helps in reducing the soil volume change due to change in temperature or moisture content.
7.Stabilization improves the workability and the durability of the soil.

## SUMMARY OF LITERATURE REVIEW

- Many investigators have conducted the studies on fiber-reinforced materials. The results of direct shear tests performed on sand specimens indicated increased shear strength.
- These results were supported by a number of researchers. Investigations were also conducted to determine the behavior of material properties of fiber-reinforced sands. The failure envelopes for fiber-sand composites were bilinear.
- The critical confining stress was a function of surface friction properties of the fibers and soil. The inclusion of discrete fibers increased both the cohesion and angle of internal friction of the specimens.
- The improvement of the engineering properties due to the inclusion of discrete fibers was determined to be a function of a variety of parameters including fiber type, fiber length, aspect ratios,, fiber content, orientation, and soil properties.
- The peak strength reportedly increased with increasing fiber content and length up to a limiting amount of each beyond which no additional benefits were observed.
- The results showed an increase of more than 20% increase in angle of internal friction, which would consequently result in significant increases in shear strength and soil bearing capacity.
- These results further suggest that the use of this type of reinforcement may prove beneficial with embankments and other foundation/geotechnical works.

# 4.OBJECTIVE OF THE PROJECT

Literature concerning the strength and stiffness of plastic waste reinforced soils clearly indicates there are limited studies in this area and there is need for detailed studies in this area.

Based on this observation the following are the objectives of our project:

1. Study the change in strength of the Chambaghat soil using geosynthetics which are available from daily use (i.e plastic bags and jute bags).
2. Study of various properties such as gsd, atterberg limits, compaction, shear parameters using various combinations of geosynthetics (plastic bags and jute bags) with soil.
3. Recommending the most suitable geosynthetics (plastic or jute) based on the comparative study done.

# 5.MATERIAL AND METHODS:

## ESSENTIAL PROPERTIES OF PLASTIC BAGS AND JUTE BAGS WHICH HELP IN SOIL STABILISATION

### 1. PLASTIC BAGS

1. ***Strength***--can withstand considerable pressure without stretching or breaking.
**2. *Durability***--will last for hopefully centuries without degradation, especially when protected by a covering of plaster, and is not adversely affected by moisture or normal temperatures.
***3. Low cost***--not too expensive for common use.
***4. Availability--***readily available in a form that can be used. I suggest that

you check with the manufacturer of the material in question and see how it compares to polypropylene, which rates very high in each of the categories.

**2. JUTE BAGS**

1. High moisture absorption capacity, flexibility , drainage properties .
2. Erosion control, separation, filtration .
3. Lower costs compared to synthetic geotextiles.
4. Ease of installation and bio-degradable properties.

**WHY PLASTIC BAGS AND JUTE BAGS??**

**1. PLASTIC BAGS**

1. Plastic bags are commonly used for shopping , storage and marketing for various purposes due to its most advantage character of less volume and weight.
2. Most of these plastic are specifically made for spot use, having short life span and are being discarded immediately after use.
3. Though, at many places waste plastics are being collected for recycling or reuse, however; the secondary markets for reclaimed plastics have not developed as recycling program.
4. Therefore, the quantity of plastics that is being currently reused or recycled is only a fraction of the total volume produced every year. The estimated municipal solid waste production in India up to the year 2000 was of the order of 39 million tons per year. From this plastics constitute around 4 % of the total waste.

**2. JUTE BAGS**

1. Worldwide, the natural fiber with the largest use after cotton is jute.
2. India is the world's largest producer of raw jute and the biggest manufacturer of jute goods

1. Mechanical method of Stabilization

In this procedure, soils of different gradations are mixed together to obtain the desired property in the soil. This may be done at the site or at some other place from where it can be transported easily. The final mixture is then compacted by the usual methods to get the required density.

2. Additive method of stabilization

It refers to the addition of manufactured products into the soil, which in proper quantities enhances the quality of the soil. Materials such as cement, lime, bitumen, fly ash etc. are used as chemical additives. Sometimes different fibers are also used as reinforcements in the soil. The addition of these fibers takes place by two methods;

**a)** Oriented fiber reinforcement-

The fibers are arranged in some order and all the fibers are placed in the same orientation. The fibers are laid layer by layer in this type of orientation. Continuous fibers in the form of sheets, strips or bars etc. are used systematically in this type of arrangement.

**b)** Random fiber reinforcement-

This arrangement has discrete fibers distributed randomly in the soil mass. The mixing is done until the soil and the reinforcement form a more or less homogeneous mixture. Materials used in this type of reinforcements are generally derived from paper, nylon, metals or other materials having varied physical properties. Randomly distributed fibers have some advantages over the systematically distributed fibers. Somehow this way of reinforcement is similar to addition of admixtures such as cement, lime etc. Besides being easy to add and mix, this method also offers strength isotropy, decreases chance of potential weak planes which occur in the other case and provides ductility to the soil , between void ratio (e) versus normal pressure (p) was obtained.
The results in the form of e

versus log p for plain sand and sand mixed with different percentages of plastic wastes .

**Soil properties**

**Atterberg Limits**

1) Liquid Limit:

It is the water content of the soil between the liquid state and plastic state of the soil. It can be defined as the minimum water content at which the soil, though in liquid state, shows small shearing strength against flowing. It is measured by the Casagrande's apparatus and is denoted by $\mathbf{W_1}$.

CASSAGRANDE,S APPARATUS

## 2) Plastic Limit

This limit lies between the plastic and semi-solid state of the soil. It is determined by rolling out a thread of the soil on a flat surface which is non-porous. It is the minimum water content at which the soil just begins to crumble while rolling into a thread of approximately 3mm diameter. Plastic limit is denoted by $w_P$. This is determined by rolling out soil till its diameter reaches approximately 3 mm and measuring water content for the soil which crumbles on reaching this diameter. Plasticity index ($I_p$) was also calculated with the help of liquid limit and plastic limit.


PLASTIC LIMIT TEST

3) Shrinkage Limit

This limit is achieved when further loss of water from the soil does not reduce the volume of the soil. It can be more accurately defined as the lowest water content at which the soil can still be completely saturated. It is denoted by $\mathbf{w_s}$.

$I_p = w_L - w_P$
$w_L$- Liquid limit
$w_P$- Plastic limit

## Particle Size Distribution

Soil at any place is composed of particles of a variety of sizes and shapes, sizes ranging from a few microns to a few centimeters are present sometimes in the same soil sample. The distribution of particles of different sizes determines many physical properties of the soil such as its strength, permeability, density etc. Particle size distribution is found out by two methods, first is sieve analysis which is done for coarse grained soils only and the other method is sedimentation analysis used for fine grained soil sample. Both are followed by plotting the results on a semi-log graph. The percentage finer N as the ordinate and the particle diameter i.e. sieve size as the abscissa on a logarithmic scale. The curve generated from the result gives us an idea of the type and gradation of the soil. If the curve is higher up or is more towards the left, it means that the soil has more representation from the finer particles; if it is towards the right, we can deduce that the soil has more of the coarse grained particles. The soil may be of two types- well graded or poorly graded (uniformly graded). Well graded soils have particles from all the size ranges in a good amount. On the other hand, it is said to be poorly or uniformly graded if it has particles of some sizes in excess and deficiency of particles of other sizes. Sometimes the curve has a flat portion also which means there is an absence of particles of intermediate size, these soils are also known as gap graded or skip graded. For analysis of the particle distribution, we sometimes use D10, D30, and D60 etc. terms which represents a size in mm such that 10%, 30% and 60% of particles respectively are finer than that size. The size of D10 also called the effective size or diameter is a very useful data. There is a term called uniformity coefficient Cu which comes from the ratio of D60 and D10, it gives a measure of the range of the particle size of the soil sample.

# Specific gravity

Substance of a definite volume divided by mass of equal volume of water. In case of soils, specific gravity is the number of times the soil solids are heavier than equal volume of water. Different types of soil have different specific gravities, general range for specific gravity of soils:

| | |
|---|---|
| Sand | 2.63-2.67 |
| Silt | 2.65-2.7 |
| Clay and Silty clay | 2.67-2.9 |
| Organic soil | <2.0 |

# Proctor compaction test

This experiment gives a clear relationship between the dry density of the soil and the moisture content of the soil. The experimental setup consists of (i) cylindrical metal mould (internal diameter- 10.15 cm and internal height-11.7 cm), (ii) detachable base plate, (iii) collar (5 cm effective height), (iv) rammer (2.5 kg). Compaction process helps in increasing the bulk density by driving out the air from the voids. The theory used in the experiment is that for any compactive effort, the dry density depends upon the moisture content in the soil. The maximum dry density (MDD) is achieved when the soil is compacted at relatively high moisture content and almost all the air is driven out, this moisture content is called optimum moisture content (OMC). After plotting the data from the experiment with water content as the abscissa and dry density as the ordinate, we can obtain the OMC and MDD.

# Shear strength

Shearing stresses are induced in a loaded soil and when these stresses reach their
limiting value, deformation starts in the soil which leads to failure of the soil mass. The
shear strength of a soil is its resistance to the deformation caused by the shear stresses
acting on the loaded soil. The shear strength of a soil is one of the most important
characteristics. There are several experiments which are used to determine shear strength
such as DST or UCS etc. The shear resistance offered is made up of three parts:
i) The structural resistance to the soil displacement caused due to the soil particles getting interlocked,
ii) The frictional resistance at the contact point of various particles, and
iii) Cohesion or adhesion between the surface of the particles.
In case of cohesionless soils, the shear strength is entirely dependent upon the frictional resistance, while in others it comes from the internal friction as well as the
cohesion.
Methods for measuring shear strength:

## a) Direct Shear Test (DST)

This is the most common test used to determine the shear strength of the soil. In this experiment the soil is put inside a shear box closed from all sides and force is
applied from one side until the soil fails. The shear stress is calculated by dividing
this force with the area of the soil mass. This test can be performed in three conditions- undrained, drained and consolidated undrained depending upon the setup of the experiment.
This test is used to find out the cohesion (c) and the angle of internal friction (φ) of the soil, these are the soil shear strength parameters. The shear strength is one of the most important soil properties and it is required whenever any structure depends on the soil shearing resistance.

The test is conducted by putting the soil at OMC and MDD inside the shear box which is made up of two independent parts. A constant normal load (ς) is applied to obtain one value of c and φ. Horizontal load (shearing load) is increased at a constant rate and is applied till the failure point is reached. This load when divided with the area gives the shear strength 'τ' for that particular normal load. The equation goes as follows:
After repeating the experiment for different normal loads (ς) we obtain a plot which is a straight line with slope equal to angle of internal friction (φ) and intercept equal to the cohesion (c). Direct shear test is the easiest and the quickest way to determine the shear strength parameters of a soil sample. The preparation of the sample is also very easy in this experiment.

## b) Unconfined Compression Test (UCS test)

This test is a specific case of triaxial test where the horizontal forces acting are zero. There is no confining pressure in this test and the soil sample tested is subjected to vertical loading only. The specimen used is cylindrical and is loaded till
it fails due to shear.
$q_u$= load/corrected area (A')
$q_u$- compressive stress
A'= cross-sectional area/ (1- ε)
This experiment is used to determine the unconfined compressive strength of the soil sample which in turn is used to calculate the unconsolidated, undrained shear strength of unconfined soil. The unconfined compressive strength (qu) is the compressive stress at which the unconfined cylindrical soil sample fails under simple compressive test. The experimental setup constitutes of the compression device and dial gauges for load and deformation. The load was taken for different readings of strain dial gauge starting from ε = 0.005 and increasing by 0.005 at each step. The corrected cross-sectional area was calculated by dividing the area by (1- ε) and then the compressive stress for each step was calculated by dividing the load with the corrected area.

C) **TRIAXIAL TEST:**
Casagrande developed the triaxial test in the course of his research aimed at removing the disadvantages of Direct Shear Test.

DST has following disadvantages:

1. Failure plane is always horizontal in the test (this may not be the weakest plane in the sample)
2. No provision for measuring pore water pressure in the shear box. So it is not possible to determine the effective stress from undrained test.
3. This test cannot give reliable undrained strengths.

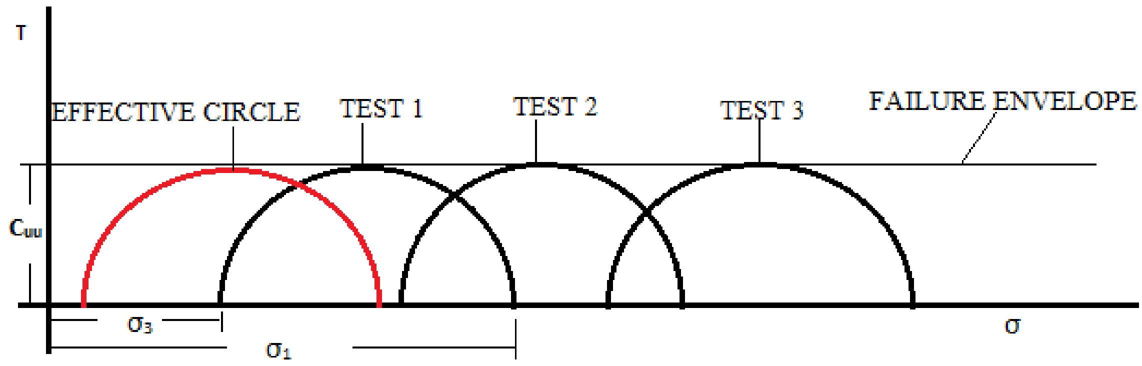
It is most versatile of all shear testing methods, even if it is a bit complicated. Drainage conditions can be controlled whatsoever be the type of soil. Sands can be tested under undrained conditions and saturated soils of low permeability can be tested under drained conditions.

In triaxial test pore water pressure can be measurement can be made accurately. Volume changes can also be measured. There is no rotation of the principal stresses during test. The failure plane is not forced. The specimen can fail on any weak plane or can simply bulge. The stress distribution on failure plane is fairly uniform.

TYPES OF TRI-AXIAL TESTS:

| S.N.O | STAGE: 1 BEFORE SHEAR(Only confining stress) | STAGE: 2 AFTER SHEAR(Confining and deviator stress) | TEST SYMBOL | REMARKS |
|---|---|---|---|---|
| 1. | Unconsolidated (No drainage) | Undrained (No drainage) | UU | Determination of total shear parameters $\mathbf{C}_u, \phi_u$. |
| 2. | Consolidated (drainage allowed) | Undrained (No drainage) | CU | Determination of total stress parameters $\mathbf{C}_{cu}$, $\phi_{cu}$ as well as effective stress parameters |
| 3. | Consolidated (drainage allowed) | Drained ( drainage allowed) | CD | Very slow test, effective stress parameters are obtained are $\mathbf{C'_{cd}}$, $\phi_{cd}$. |

1. UNCONSOLIDATED UNDRAINED (UU): The undrained test is carried on undisturbed sample of clay, silt and peat to determine the strength of the natural ground. It is also carried out on remoulded samples of clay to measure its sensitivity.
   *The results of unconsolidated undrained tests are as shown:*




UNCONSOLIDATED UNDRAINED TEST FOR TRI-AXIAL

It may be concluded that in an undrained test on saturated clays , both the major principal effective stress $\sigma'_1$ and the minor principal effective stress $\sigma'_3$ are independent of magnitude of the cell pressure applied.

All Mohr's circles for UU test are plotted in terms of either total stresses or effective stresses have same diameter.

$$\sigma'_3 = \sigma_3 - u$$
$$\sigma'_1 = \sigma_1 - u$$

Deviator stress $\sigma'_1 - \sigma'_3 = \sigma_1 - \sigma_3 = \sigma_d$ = diameter .

Failure envelope is a horizontal straight line, $\phi_{uu}=0$ , it can be represented by equation,

Failure envelope = $\tau_f = C_{uu} = \dfrac{\sigma_1 - \sigma_3}{2}$

$C_{uu}$= Cohesion for unconsolidated undrained.

2.CONSOLIDATED –UNDRAINED TRI-AXIAL TESTS(CU): Unlike the consolidated-drained test the total and effective principal stresses are not the same in the consolidated-undrained test. Because the pore water pressure at failure is measured in this test, *the principal stresses may be analyzed as follows*

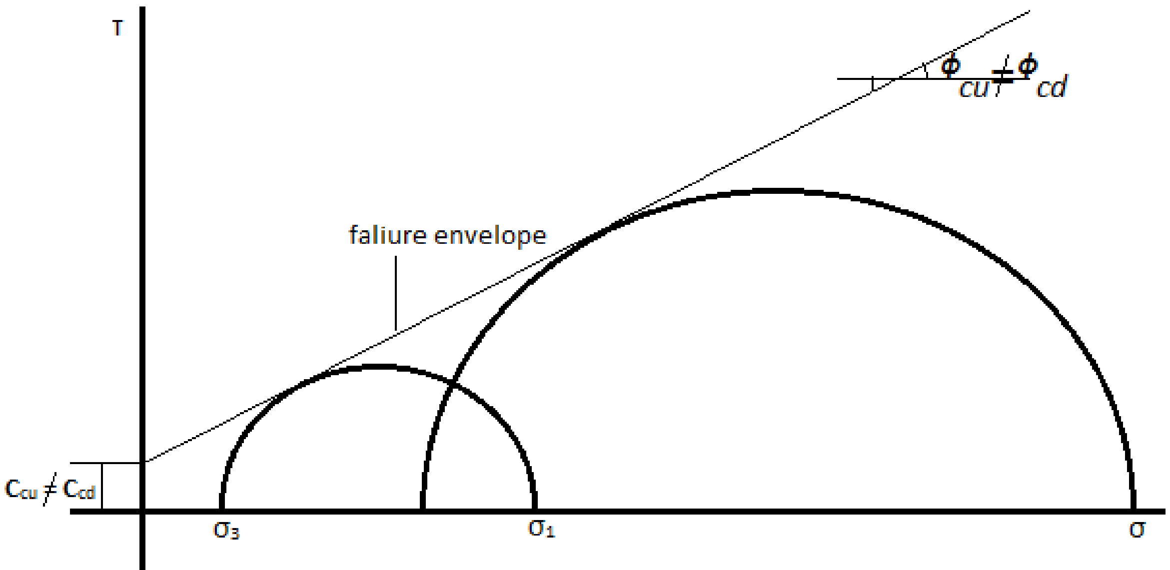
Major Principal stress at failure (total) = $\sigma_3 + (\Delta\sigma_d)_f = \sigma_1$

Major Principal stress at failure (effective) = $\sigma_1 - (\Delta u_d)_f = \sigma'_1$

Minor Principal stress at failure (total) = $\sigma_3$

Minor principal stress at failure (effective) = $\sigma_3 - (\Delta u_d)_f = \sigma'_3$

$(\Delta u_d)_f$ = Pore Water Pressure At Failure.




Mohr's Circle of Failure envelope

For test involving drainage in the first stage, when Mohr's circles are plotted in terms of total stresses, the diameter increases with the confining pressure. The resulting failure envelope is an inclined line with an intercept on vertical axis.

Here C , $\phi$ are the stress parameters.

2.CONSOLIDATED – DRAINED (CD): In the CD test, saturated specimen is first subjected to all around confining pressure $\sigma_3$ , by compression of the chamber fluid. As confining pressure is applied the pore water pressure of the specimen increases by $U_c$ *(if drainage is prevented).* This increase in the pore water pressure can be expressed as a non-dimensional parameter in the form.

$$\boxed{B = \frac{U_c}{\sigma_3}}$$

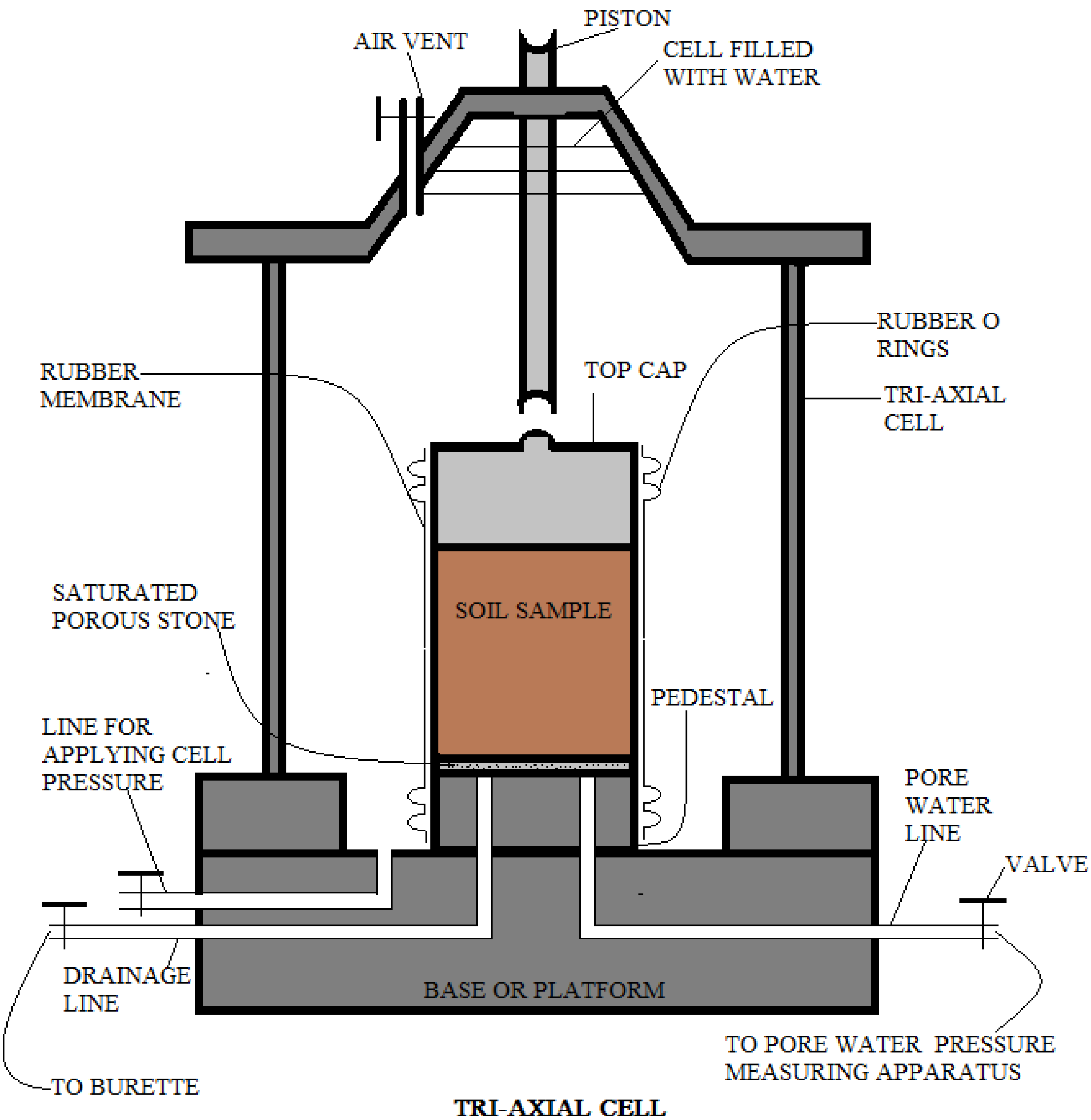
Where, B= skempton’s pore pressure parameter.

In this test, the pore water pressure developed during the test is completely dissipated, we have total and effective confining stress = $\sigma_3 = \sigma'_3$.

And the total and effective axial stress at failure = $\sigma_3 + (\Delta\sigma_d)_f$

$$= \sigma_1 = \sigma'_1$$

PROCEDURE TO CONDUCT THE TRIAXIAL TEST-




**TRI-AXIAL CELL**

1. Placing the cylindrical soil sample to be tested on the pedestal of *a triaxial cell* after placing on the pedestal a saturated porous stone.
2. Isolating the soil sample from the water which will surround it when the cell is filled with water by enveloping the sample with rubber membrane and sealing it at the bottom with the pedestal and at the top with top cap

by *rubber "O" rings*. Filling the triaxial with water and applying pressure to the water which in turn transmits that pressure to the soil sample in a direction normal to the sample's surface, that is, the sample is compressed axially at its horizontal top and bottom ends and radially on its vertical surface. This normal stress is denoted by $\boldsymbol{\sigma}_c$ and is called the *cell pressure* or the *chamber pressure*, or the *confining pressure*. At this stage, *the first stage* of the test, the axial stress, $\boldsymbol{\sigma}_a$, is equal to the radial stress, $\boldsymbol{\sigma}_r$ , and both are equal to the confining pressure $\boldsymbol{\sigma}_c$.

3. In the second stage of the test, keeping the cell pressure constant and applying additional axial stress, $\Delta\sigma_a$ on the sample through the piston which produces shearing stresses on all planes through the sample except the horizontal and the vertical planes. The horizontal plane, since it has acting on it only normal stress due to confining pressure, $\boldsymbol{\sigma}_c$, as well as due to the additional axial stress applied through the piston, $\Delta\sigma_a$, becomes the major principal plane and the axial stress $\boldsymbol{\sigma}_a = \boldsymbol{\sigma}_c + \Delta\boldsymbol{\sigma}_\mathbf{a}$ becomes the minor principal stress, $\boldsymbol{\sigma}_3$. The intermediate principal stress, $\boldsymbol{\sigma}_2$, in the triaxial compression test is equal to the minor principal stress since on account of the axial symmetry the two orthogonal horizontal principal stresses are equal to each other and equal to cell pressure.
4. Continuously increasing the additional axial stress applied to the sample through the piston until the sample fails. At failure, then, the existing axial stress is denoted by
$\boldsymbol{\sigma}_{1f} = \boldsymbol{\sigma}_c + \Delta\boldsymbol{\sigma}_{\boldsymbol{af}}$ and the existing radial stress is denoted by $\boldsymbol{\sigma}_{3f} = \boldsymbol{\sigma}_c$.

During the tri-axial test a number of observations are described below can be made on physical changes occurring in the soil sample:

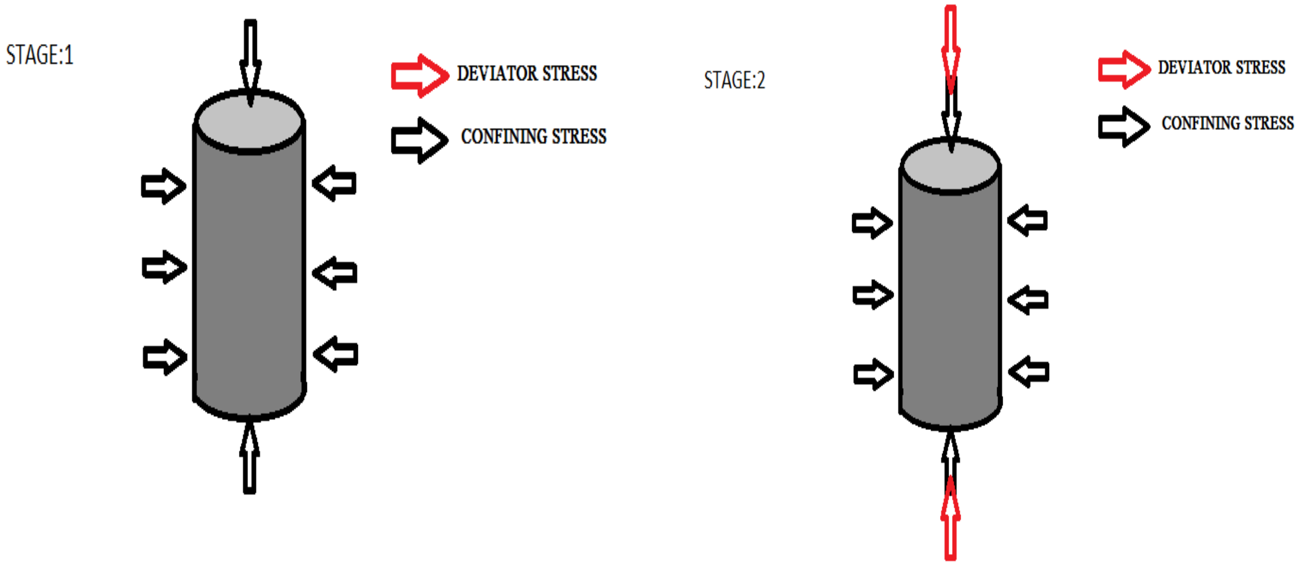
1. As cell pressure is applied, the pore water pressure in the sample increases at the instant of application of cell pressure by an amount equal to the applied cell pressure. This increase in pore water pressure can be measured by a *pore water pressure measuring apparatus* connected to the *pore water line* after arranging that the valve in the *drainage line* is closed and the valve in the pore water line is open. The pore water line which is itself filled with water enables the water in the voids of the soil sample to be connected up through the water in the pores of the saturated porous stone to the water in the pore water pressure measuring apparatus.

2. If we wish to eliminate the pore water pressure induced in the soil sample on account of the applied cell pressure, this can be achieved by closing the valve in the pore water line and opening the valve in the drainage line. Water will then drain from the sample through the drainage line to the valve in the drainage line. Water will then drain from the sample through the drainage line to *burette* connected to this line in which amount of water drained can be measured. The process of consolidation takes time. The change in sample volume equals the observed amount of water drained.
3. In stage two of the test, as the additional axial stress is applied, the sample compresses in the axial direction. Or to state it another way, associated with application of additional axial stress is an axial strain. As the sample compresses the piston moves down. The downward movement of the piston can be measured and from that observation the axial strain, can be determined.
4. On application of the additional axial stress some pore water pressure develops in the sample; this pressure can also be measured with the pore water measuring apparatus so long as the valve in the drainage line is closed. On the other hand, if we wish that any pore water pressure developed be allowed to dissipate, which of course will take time, then the valve in the pore water line is closed and valve in drainage line is opened and water is allowed to drain from the drainage line to burette. This flow of water can be measured and it equals the change in the sample volume.
5. During the triaxial test, the cell pressure is of course, kept constant and the additional axial stress applied is measured.

In summary, then, the triaxial compression test consists of two stages:

1. The first stage, in which after the sample is set in the triaxial cell, cell pressure is applied to the sample to subject it to normal stresses only.
2. The second stage, in which additional axial stress is applied and shear stresses are induced in the sample and are continuously increased by increasing the additional axial stress until sample fails.

During the two stages, measurements are made of the stresses applied, of the induced pore water pressures or the changes in the sample volume, and of the axial strain.



## MATERIALS USED

| MATERIAL REQUIRED | QUANTITY | SOURCE |
|---|---|---|
| JUTE BAGS | 4-5 (BIG SIZE) | FROM MOKSH (WAKNAGHAT) |
| SOIL | 8-10 kg | FROM CHAMBAGHAT |

## DIMENSIONAL PARAMETERS OF VARIOUS MATERIALS

| SR. NO. | DIMENSIONS | JUTE |
|---|---|---|
| 1 | MASS PER UNIT AREA | 5.5 |
| 2 | THICKNESS | NIL |
| 3 | DIAMETER | 1mm ,2mm |
| 4 | LENGTH | 35 mm |
| 5 | WIDTH | NIL |

***Soil was brought from a sand quarry near Chambaghat, Solan (H.P)***

# RESULTS & DISCUSSION

## 1. GRAIN SIZE DISTRIBUTION

a) We have taken 1 kg of soil sample and performed GSD test for three times to achieve accuracy in results.

b) We tested soil on sieves of size 2mm,710 microns,600 microns,425 microns,300 microns,250 microns,150 microns,75 microns

c) We used sieve shaker for sieving

d) The results obtained were as follows:

| **WT. OF SIEVE + SOIL RETAINED [gm]** | **TEST 1** | **TEST 2** | **TEST 3** |
|---|---|---|---|
| SIEVE SIZE [mm] | | | |
| 40 | 830 | 830 | 830 |
| 20 | 830 | 830 | 830 |
| 10 | 470 | 470 | 470 |
| 4.75 | 390 | 390 | 390 |

| WT OF SIEVE + SOIL RETAINED [gm] | TEST 1 | TEST 2 | TEST 3 |
| --- | --- | --- | --- |
| SIEVE SIZE [mm] | | | |
| 2mm | 401 | 400 | 400 |
| 710 microns | 550 | 540 | 545 |
| 600 microns | 410 | 400 | 410 |
| 425 microns | 480 | 480 | 481 |

| WT OF SIEVE + SOIL RETAINED [gm] | TEST 1 | TEST 2 | TEST 3 |
| --- | --- | --- | --- |
| SIEVE SIZE [mm] | | | |
| 300 micron | 440 | 451 | 446 |
| 250 micron | 460 | 461 | 461 |
| 150 micron | 401 | 399 | 400 |
| 75 micron | 361 | 359 | 362 |

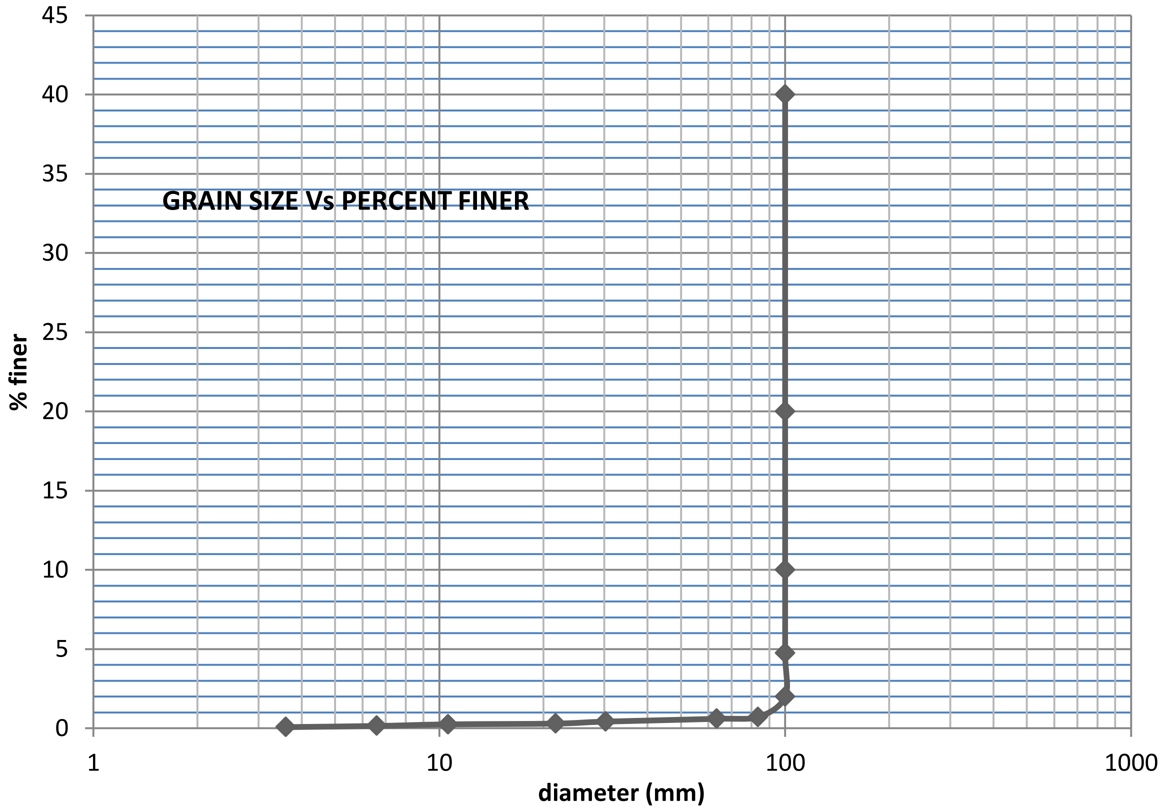
FOR COARSE GRAINED SOIL 4.75 MM AND ABOVE

| SIEVE SIZE [mm] | MASS OF SIEVE [gm] | MASS OF SIEVE + SOIL RETAINED[gm] | MASS OF SOIL RETAINED [gm] | % OF MASS OF SOIL RETAINED [gm] | CUMULATIVE % OF SOIL RETAINED [gm] | % FINER [passing] |
|---|---|---|---|---|---|---|
| [1] | [2] | [3] | 4=[3-2] | 5 | 6 | 7= 100-[6] |
| 40 | 830 | 830 | 0 | 0 | 0 | 100 |
| 20 | 830 | 830 | 0 | 0 | 0 | 100 |
| 10 | 470 | 470 | 0 | 0 | 0 | 100 |
| 4.75 | 390 | 390 | 0 | 0 | 0 | 100 |

FOR FINE GRAINED SOIL 2MM-75 MICRONS

| SIEVE SIZE | MASS OF SIEVE [gm] | MASS OF SIEVE + SOIL RETAINED[gm] | MASS OF SOIL RETAINED [gm] | % of MASS OF SOIL RETAINED [gm] | CUMULATIVE % OF SOIL RETAINED [gm] | % FINER [passing] |
|---|---|---|---|---|---|---|
| [1] | [2] | [3] | 4=[3-2] | [5] | [6] | 7= 100-[6] |
| 2mm | 400 | 400 | 0 | 0 | 0 | 100 |
| 710 micron | 380 | 545 | 165 | 16.5 | 16.5 | 83.5 |
| 600 micron | 370 | 406 | 36 | 20.1 | 36.6 | 63.4 |
| 425 micron | 350 | 480 | 130 | 33.1 | 69.7 | 30.3 |

**The concluded graph between percent finer and particle size was as follows**



1.We use pycnometer method to find specific gravity of soil .

We performed this test three times to achieve accuracy in results.

2.Specific gravity test value helps us to classify soil to some

extent as-

Its value ranges as follows –

1. Coarse grained soil= [2.6-2.7]
2. Fine grained soil=[2.7-2.8]
3. Organic soil=[2.3-2.5]

## The readings of this test are shown in this table:

| WEIGHT OF PYCNOMETER BOTTLE AT VARIOUS STAGES [gm] | TEST 1 | TEST 2 | TEST 3 |
|---|---|---|---|
| W1= WEIGHT OF EMPTY PYCNOMETER | 465.5 | 465.5 | 465.5 |
| W2= WEIGHT OF PYCNOMETER PARTIALLY FILLED WITH SOIL | 665.6 | 665.4 | 665.6 |
| W3= WEIGTH OF PYCNOMETER WITH SOIL AND WATER FILLED UPTO BRIM | 1387.9 | 1387.8 | 1387.8 |
| W4= WEIGHT OF PYCNOMETER FILLED WITH WATER | 1261 | 1260 | 1260 |

## RESULT OF SPECIFIC GRAVITY TEST

- After taking readings we took their average and put them into the formula given below for calculating specific gravity.
- Gs= [W2-W1] / [W2-W1]-[W3-W4]
- We found the value of Gs to be 2.7 , which tells us that it is a fine grained soil .

## ATTERBERG LIMIT TEST

**1.** PLASTIC LIMIT:

1. The moisture content at which soil can be moulded to different shapes without rupture , is called as plastic limit .
2. Experimentally the water content at which soil can be moulded to a thread of 3mm dia is plastic limit of soil.
3. In our case the soil cracked down even when it was just rolled to a ball.
4. We tested the 4 samples of chambaghat soil.
5. By using formulation as= $(W_2-W_1)/(W_3-W_4)$.

P.L= (weight of moist sample-weight of dry sample)/(weight of dry sample in box – weight of box )

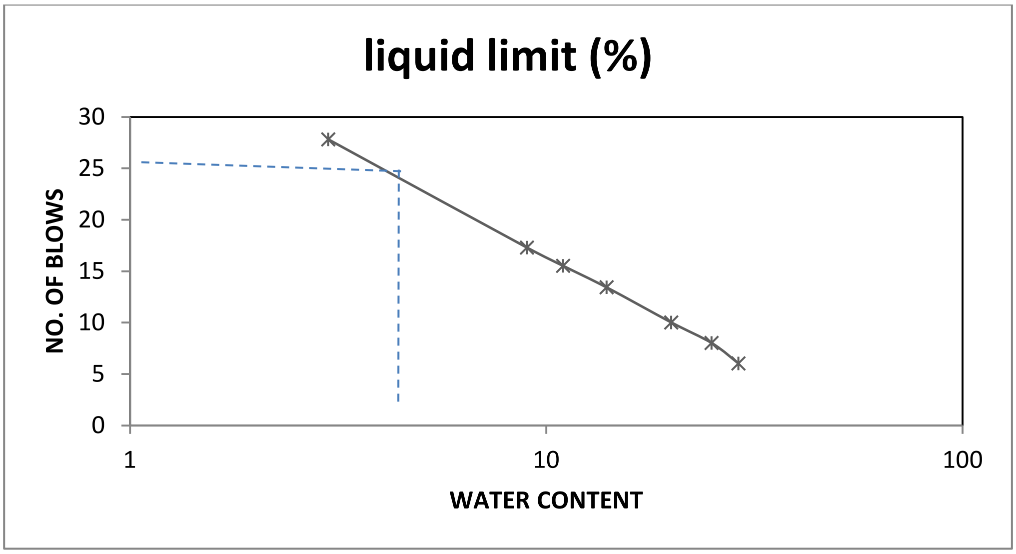
THE READINGS ARE AS SHOWN IN TABULAR FORM:

| s.n.o | Sample box no. | Sample box weight (gm) | Sample box + dry soil sample weight (gm) | box + oven dried sample weight (gm) | weight of water in each sample (gm) | Plastic limit (%) |
|---|---|---|---|---|---|---|
| 1 | 45 | 27 | 53.9 | 49.1 | 4.8 | 21.71 |
| 2 | 24 | 27 | 47.2 | 43.2 | 4 | 24.69 |
| 3 | 22 | 27 | 52.4 | 43.3 | 9.1 | 21 |
| 4 | 36 | 27 | 41.2 | 38.8 | 2.4 | 20.33 |

**After taking average of the four plastic limits , plastic limit of sample was found to be 22%**

3. LIQUID LIMIT:
   1. Water content at which soil is practically in liquid state but bears infinitesimal resistance against flow . Experimentally water content at which groove made through grooving tool of casagrande apparatus touches it by giving blows and counting them too.
   2. We took 7 sample readings for this experiment .
   3. A plot between water content and number of blows is made .
   4. Water content for $25^{th}$ blow is the liquid limit of the soil.

| SR. NO. | Water content % | No. of blows |
|---|---|---|
| 1 | 27.8 | 3 |
| 2 | 17.28 | 9 |
| 3 | 15.5 | 11 |
| 4 | 13.4 | 14 |
| 5 | 10 | 20 |
| 6 | 8 | 25 |
| 7 | 6 | 29 |



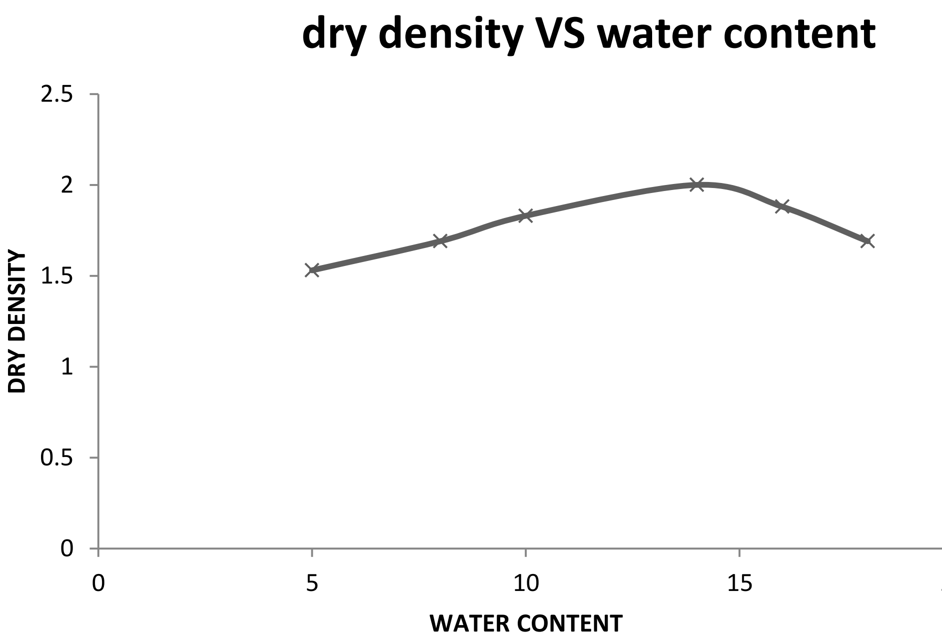

**From graph we found the liquid limit of the soil sample to be 14.8%.**

## PROCTOR TEST:

1. This test is done to know the maximum compaction or maximum dry density , and at what water content it could be achieved.
2. This water content is called as optimum moisture content (OMC).
3. The procedure include to detect the dry density at various water levels ( 5% , 8% , 10% , 14% , 16% , 18% as we have done in our experiment.)
4. A plot between dry density and water content is made .
5. The peak of graph gives the OMC.
6. Key formulations used in this test is=

$$\gamma_{d} = \gamma / 1+w$$

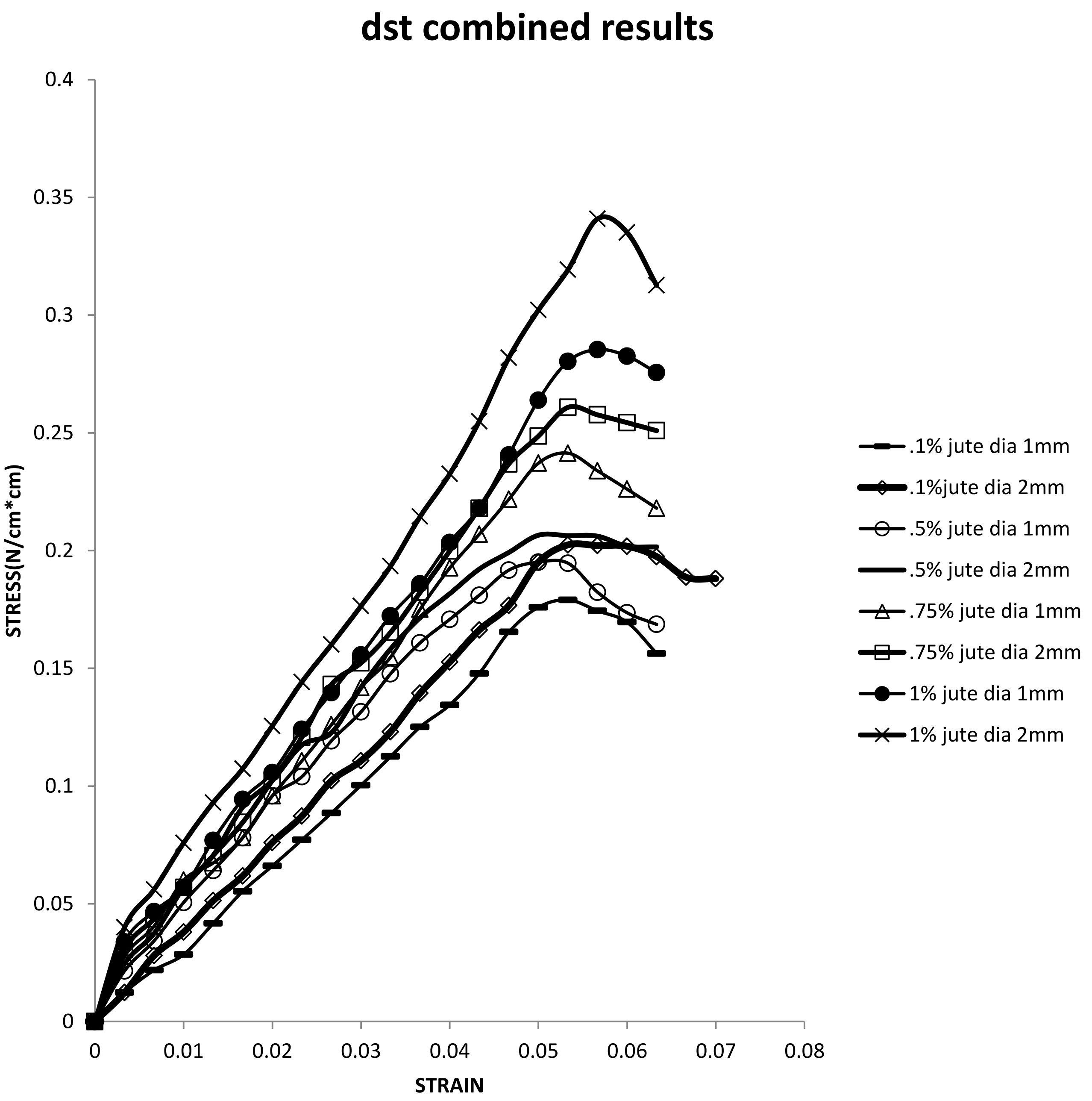
| **Water content w %** | **Dry density $\gamma_d$ (g/cc)** |
|---|---|
| 5 | 1.53 |
| 8 | 1.69 |
| 10 | 1.83 |
| 14 | 2 |
| 16 | 1.88 |
| 18 | 1.69 |



THE OMC was found out to be 14.5% and dry density to be 2g/cc or 19.6 $kN/m^3$ .

DIRECT SHEAR TEST RESULTS:

- Main objective is to determine shear strength parameters of the given soil sample at known density and moisture content by direct shear test.
- Internal friction which is the resistance due to friction between individual particles at their contact points and interlocking of particles.
- Cohesion which is resistance due to inter-particles forces which tend to hold the particles together in the soil mass.
- The purpose of direct shear test is to get the ultimate shear resistance , peak shear resistance cohesion, angle of internal friction, $\Phi$ and shear stress-strain characteristics of the soils.
- Shear parameters are used in the design of earthen dams and embankments. These are used in calculating the bearing capacity of soil-foundation systems.
- Parameters help in estimating the earth pressure behind the retaining walls. The values of these parameters are also used in checking the stability of natural slopes, cuts and fills.


dst combined results
STRESS(N/cm*cm)
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
STRAIN
.1% jute dia 1mm
.1%jute dia 2mm
.5% jute dia 1mm
.5% jute dia 2mm
.75% jute dia 1mm
.75% jute dia 2mm
1% jute dia 1mm
1% jute dia 2mm

TABULATED DST RESULTS FOR JUTE DIA=1mm

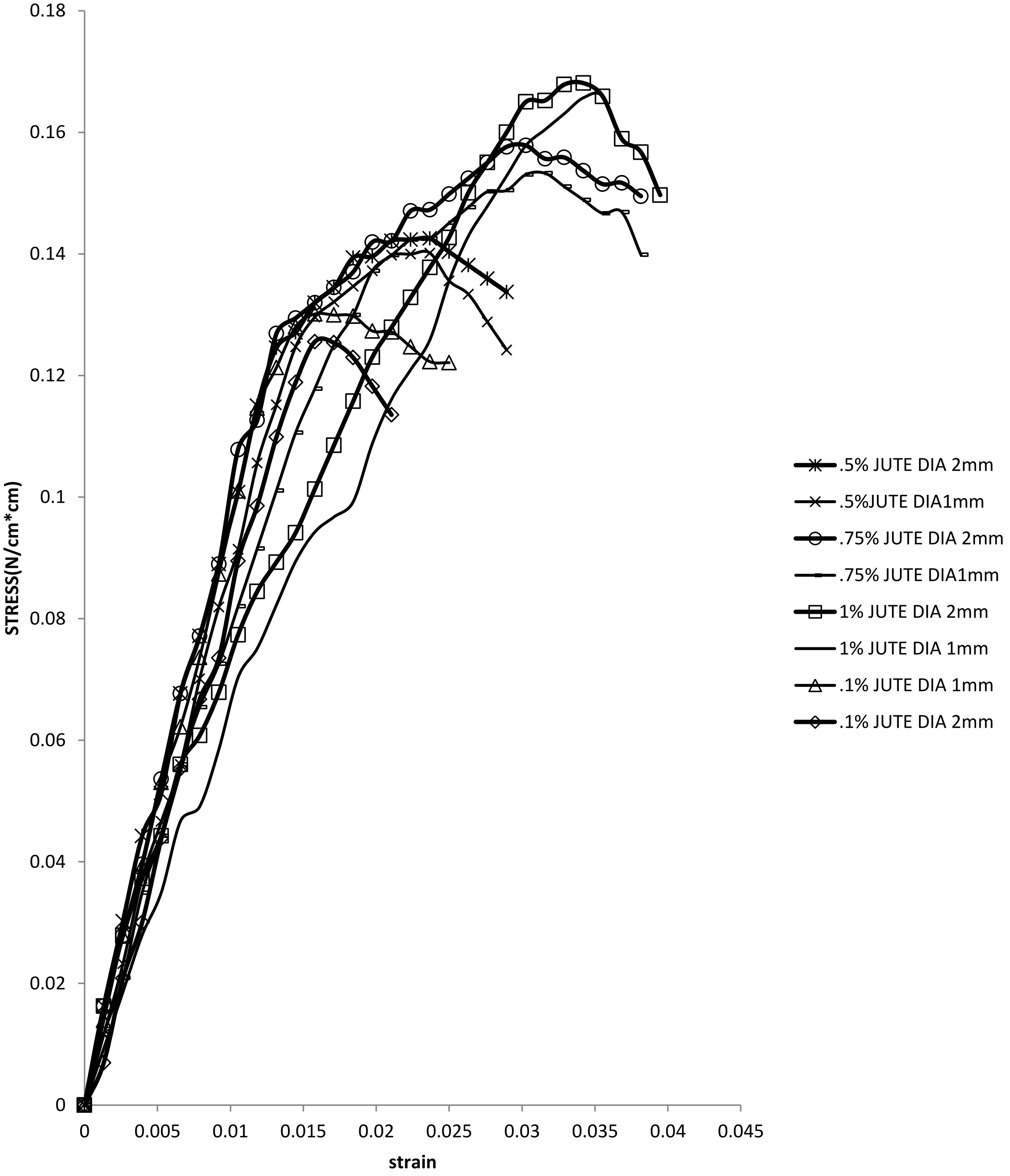
| S.N.O | PROPORTION OF JUTE(%/WT OF SAMPLE) | SHEARING STRESS(KN/m*m) | NORMAL STRESS(KN/m*m) |
|---|---|---|---|
| 1. | .1% | 1.790 | 4.025 |
| 2. | .5% | 1.950 | 3.954 |
| 3. | .75% | 2.413 | 4.025 |
| 4. | 1% | 2.853 | 4.098 |

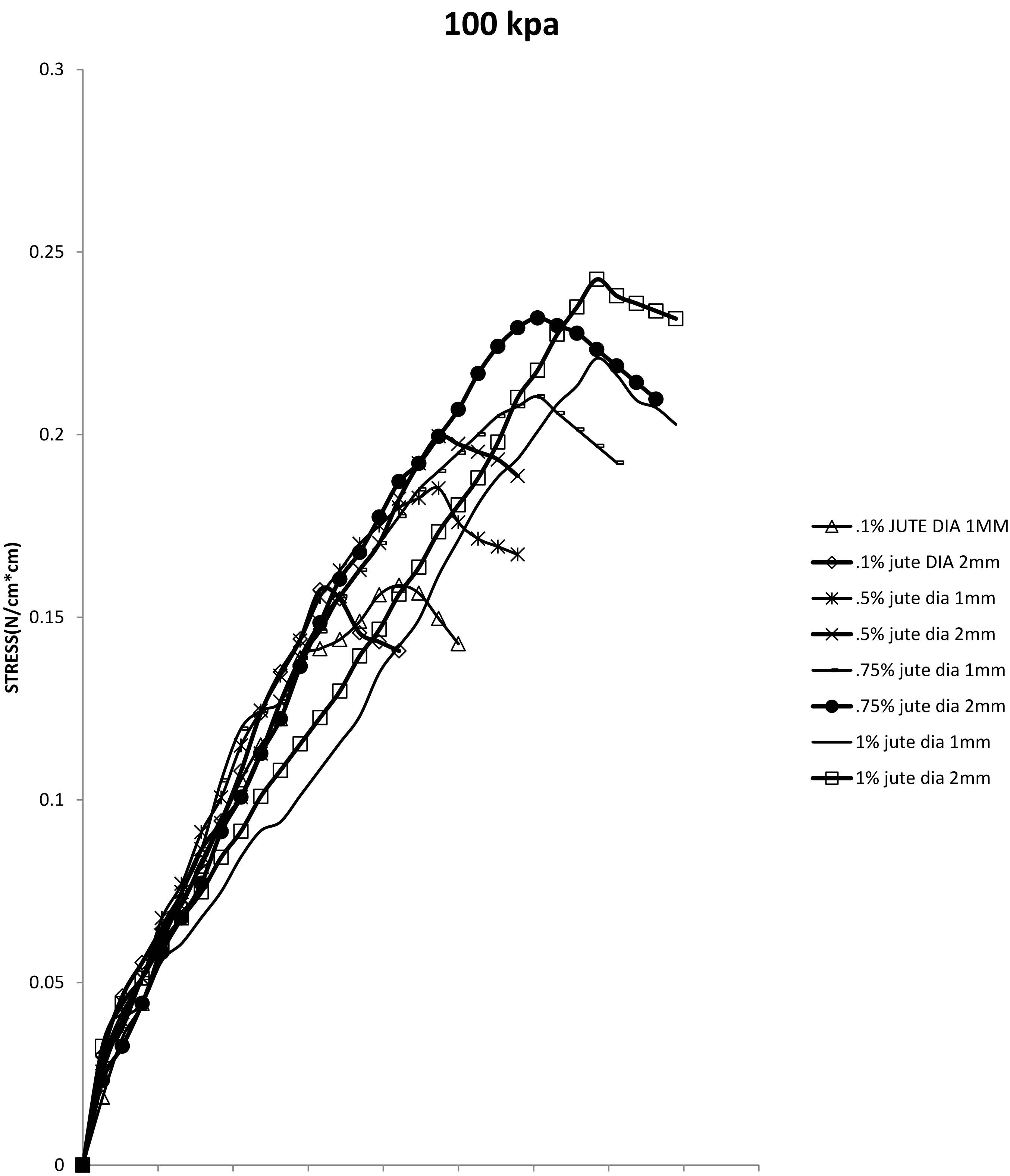
TABULATED DST RESULTS FOR JUTE DIA=2mm

| S.N.O | PROPORTION OF JUTE(% WT/ SAMPLE) | SHEARING STRESS(KN/m*m) | NORMAL STRESS(KN/m*m) |
|---|---|---|---|
| 1 | .1% | 2.024 | 4.025 |
| 2 | .5% | 2.065 | 3.954 |
| 3 | .75% | 2.608 | 4.025 |
| 4 | 1% | 3.465 | 4.098 |

TRIAXIAL TEST RESULTS: (CU-TEST)

1. The objective is to know changes in shear strength parameters which occur when different proportions of jute threads with varying diameters are used.
2. To know changes in shear strength paramerters with different confining pressures (50 kpa. 100kpa).


0.18
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
STRESS(N/cm*cm)
0
0.005
0.01
0.015
0.02
0.025
0.03
0.035
0.04
0.045
strain
.5% JUTE DIA 2mm
.5%JUTE DIA1mm
.75% JUTE DIA 2mm
.75% JUTE DIA1mm
1% JUTE DIA 2mm
1% JUTE DIA 1mm
.1% JUTE DIA 1mm
.1% JUTE DIA 2mm



100 kpa
STRESS(N/cm*cm)
strain
0
0.05
0.1
0.15
0.2
0.25
0.3
0.005
0.01
0.015
0.02
0.025
0.03
0.035
0.04
0.045
.1% JUTE DIA 1MM
.1% jute DIA 2mm
.5% jute dia 1mm
.5% jute dia 2mm
.75% jute dia 1mm
.75% jute dia 2mm
1% jute dia 1mm
1% jute dia 2mm

TABULATED RESULTS OF CU TEST FOR 50Kpa OF CONFINING PRESSURE:

| S.N.O | PROPORTION OF JUTE(%/WT OF SAMPLE) | STRAIN(%) | SHEARING STRESS(KN/m*m) |
|---|---|---|---|
| 1 | .1% | 1.57 | .765 |
| 2 | .5% | 2.36 | 1.016 |
| 3 | .75% | 3.02 | 1.11 |
| 4 | 1% | 3.4 | 1.680 |

TABULATED RESULTS OF CU TEST FOR 100Kpa OF CONFINING PRESSURE:

| S.N.O | PROPORTION OF JUTE(%/WT OF SAMPLE) | STRAIN(%) | SHEARING STRESS(KN/m *m) |
|---|---|---|---|
| 1 | .1% | 1.57 | 1.255 |
| 2 | .5% | 2.36 | 1.401 |
| 3 | .75% | 3.02 | 1.53 |
| 4 | 1% | 3.42 | 1.656 |

# 7.CONCLUSION:

1. Visible effect is seen in strength parameters of soil by reinforcing with jute fiber.
2. There is increase in strength parameters by increasing the proportion of jute fiber in the soil.
3. Increase in strength parameters is also seen with increasing diameter of jute fiber.
4. About 197% increase was seen in shearing stress of normal soil and soil reinforced with 1% jute of dia=1mm, and for dia =2mm, there was 255% increase.
5. By CU test with 50Kpa of confining pressure shear strength increased by 75% for dia= 1mm, when soil was reinforced with 1% jute fiber, for same proportion 100% of increase was seen for jute dia =2mm.
6. By CU test with 100Kpa of confining pressure shear strength increased by 129% for 1% jute dia=1mm, and 133% for jute dia=2mm.

# 8.SCOPE OF THE STUDY

1. Soil stabilization techniques have a great future in coming years this technique can improve the quality of marginal soils and good foundations for new structures can be made .
2. Now we can use readily available jute and plastic bags as soil stabilizing materials which serves two purpose at one time that is soil stabilization and effective use of jute and plastic bags.
3. Here we are focusing on strength parameters of soil on addition of plastic bags and jute bags , durability of plastic bags and jute bags is the sphere where researches are to carried on .

# 9. PHOTO GALLERY:

FIG.1 JUTE FIBRE

FIG.2 JUTE FIBRE OF DIA 2mm AND 1mm.

FIG.3 PERFORMING SPECIFIC GRAVITY TEST

FIG.4 SAMPLE FOR CU TRIAXIAL TEST

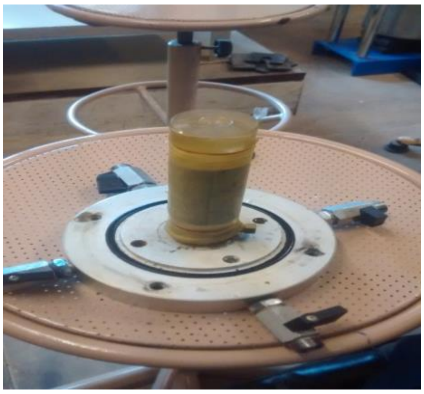


FIG .5 SAMPLE PLACED ON PEDESTAL

FIG.6 FAILED SAMPLE IN CU TEST

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