SOIL STABILIZATION USING WASTE FIBER MATERIALS

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Project Report Submitted in fulfillment of the requirements for the degree of

Bachelor of Technology

in

Civil Engineering

by

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CERTIFICATE

This is to certify that the project entitled ***SOIL STABILIZATION USING WASTE FIBER MATERIALS*** submitted by Mr. ***Arpan Sen*** (Roll No. **108CE019**) and Mr. ***Rishabh Kashyap*** (Roll. No. **108CE018**) in fulfillment of the requirements for the award of Bachelor of Technology Degree in Civil Engineering at NIT Rourkela is an authentic work carried out by them under my supervision and guidance.

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Place: Rourkela

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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 polypropylene 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 replacement for deep foundation or raft foundation, as a cost effective approach.

CHAPTER – 1

INTRODUCTION

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.

From the beginning of construction work, the necessity of enhancing soil properties has come to the light. Ancient civilizations of the Chinese, Romans and Incas utilized various methods to improve soil strength etc., some of these methods were so effective that their buildings and roads still exist.

In India, the modern era of soil stabilization began in early 1970's, with a general shortage of petroleum and aggregates, it became necessary for the engineers to look at means to improve soil other than replacing the poor soil at the building site. Soil stabilization was used but due to the use of obsolete methods and also due to the absence of proper technique, soil stabilization lost favor. In recent times, with the increase in the demand for infrastructure, raw materials and fuel, soil stabilization has started to take a new shape. With the availability of better research, materials and equipment, it is emerging as a popular and cost-effective method for soil improvement.

Here, in this project, soil stabilization has been done with the help of randomly distributed polypropylene fibers obtained from waste materials. The improvement in the shear strength parameters has been stressed upon and comparative studies have been carried out using different methods of shear resistance measurement.

CHAPTER- 2

LITERATURE REVIEW

2.1 Soil Stabilization

2.1.1 Definition

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.

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:

- 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.

2.1.2 Needs & Advantages

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. [9]

- It improves the strength of the soil, thus, increasing the soil bearing capacity.
- 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.
- It is also used to provide more stability to the soil in slopes or other such places.
- Sometimes soil stabilization is also used to prevent soil erosion or formation of dust, which is very useful especially in dry and arid weather.
- 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.
- It helps in reducing the soil volume change due to change in temperature or moisture content.
- Stabilization improves the workability and the durability of the soil.

2.1.3 Methods [8]

- 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.

- 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.

2.2 Soil properties

2.2.1 Atterberg Limits

1) 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 w_S.

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.

3) 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 w_L.

2.2.2 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 D_{10}, D_{30}, and D_{60} 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 D_{10} also called the effective size or diameter is a very useful data. There is a term called uniformity coefficient C_u which comes from the ratio of D_{60} and D_{10}, it gives a measure of the range of the particle size of the soil sample.

2.2.3 Specific gravity

Specific gravity of a substance denotes the number of times that substance is heavier than water. In simpler words we can define it as the ratio between the mass of any 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

Table- 1

2.2.4 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.

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.

CHAPTER-3

EXPERIMENTAL INVESTIGATIONS

3.1 Scope of work

The experimental work consists of the following steps:

1. Specific gravity of soil
2. Determination of soil index properties (Atterberg Limits)
   i) Liquid limit by Casagrande's apparatus
   ii) Plastic limit
3. Particle size distribution by sieve analysis
4. Determination of the maximum dry density (MDD) and the corresponding optimum moisture content (OMC) of the soil by Proctor compaction test
5. Preparation of reinforced soil samples.
6. Determination of the shear strength by:
   i) Direct shear test (DST)
   ii) Unconfined compression test (UCS).

3.2 Materials

- Soil sample-1
  Location: Behind electrical annex building, academic block, N.I.T Rourkela

- Soil sample- 2
  Location: New lecture gallery complex, N.I.T Rourkela

- Reinforcement: Short PP (polypropylene) fiber

Index and strength parameters of PP-fiber

Behavior parameters	Values
Fiber type	Single fiber
Unit weight	0.91 g/cm^3
Average diameter	0.034 mm
Average length	12 mm
Breaking tensile strength	350 MPa
Modulus of elasticity	3500 MPa
Fusion point	165 °C
Burning point	590 °C
Acid and alkali resistance	Very good
Dispersibility	Excellent

Table- 2

Fig. -1

3.3 Preparation of samples

Following steps are carried out while mixing the fiber to the soil-

- All the soil samples are compacted at their respective maximum dry density (MDD) and optimum moisture content (OMC), corresponding to the standard proctor compaction tests
- Content of fiber in the soils is herein decided by the following equation:

$$\rho_f = \frac{W_f}{W}$$

Where, ρf= ratio of fiber content

W_f = weight of the fiber

W = weight of the air-dried soil

- The different values adopted in the present study for the percentage of fiber reinforcement are 0, 0.05, 0.15, and 0.25.
- In the preparation of samples, if fiber is not used then, the air-dried soil was mixed with an amount of water that depends on the OMC of the soil.
- If fiber reinforcement was used, the adopted content of fibers was first mixed into the air-dried soil in small increments by hand, making sure that all the fibers were mixed thoroughly, so that a fairly homogenous mixture is obtained, and then the required water was added.

3.4 Brief steps involved in the experiments

3.4.1 Specific gravity of the soil

The specific gravity of soil is the ratio between the weight of the soil solids and weight of equal volume of water. It is measured by the help of a volumetric flask in a very simple experimental setup where the volume of the soil is found out and its weight is divided by the weight of equal volume of water.

$$\text{Specific Gravity } G = \frac{W_2 - W_1}{(W_4 - W1) - (W3 - W2)}$$

W1- Weight of bottle in gms

W2- Weight of bottle + Dry soil in gms

W3- Weight of bottle + Soil + Water

W4- Weight of bottle + Water

Specific gravity is always measured in room temperature and reported to the nearest 0.1.

3.4.2 Liquid limit

The Casagrande tool cuts a groove of size 2mm wide at the bottom and 11 mm wide at the top and 8 mm high. The number of blows used for the two soil samples to come in contact is noted down. Graph is plotted taking number of blows on a logarithmic scale on the abscissa and water content on the ordinate. Liquid limit corresponds to 25 blows from the graph.

3.4.3 Plastic limit

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;

$$I_p = w_L - w_P$$

w_L- Liquid limit

w_P- Plastic limit

First and/or second letters

Symbol	Definition
G	gravel
S	sand
M	silt
C	clay
O	organic

Second letter

Letter	Definition
P	poorly graded (uniform particle sizes)
W	well graded (diversified particle sizes)
H	high plasticity
L	low plasticity

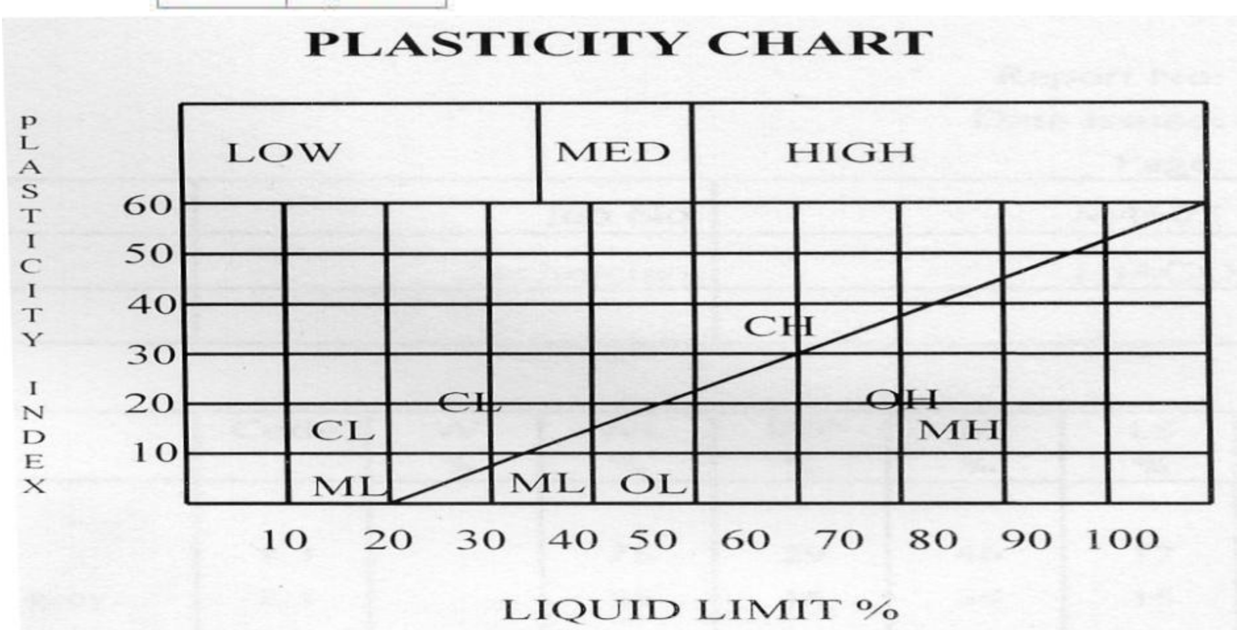

Fig. -2

3.4.4 Particle size distribution

The results from sieve analysis of the soil when plotted on a semi-log graph with particle diameter or the sieve size as the abscissa with logarithmic axis and the percentage passing as the ordinate gives a clear idea about the particle size distribution. From the help of this curve, D_{10} and D_{60} are determined. This D_{10} is the diameter of the soil below which 10% of the soil particles lie. The ratio of, D_{10} and D_{60} gives the uniformity coefficient (C_u) which in turn is a measure of the particle size range.

3.4.5 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. The equations used in this experiment are as follows:

$$\text{Wet density} = \frac{\text{weight of wet soil in mould (gms)}}{\text{volume of mould(cc)}}$$

$$\text{Moisture content \%} = \frac{\text{weight of water (gms)}}{\text{weight of dry soil (gms)}} \text{X } 100$$

$$\text{Dry density } \gamma_d \text{ (gm/cc)} = \frac{\text{wet density}}{1+\frac{\text{moisture content}}{100}}$$

3.4.6 Direct shear test

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:

$$\tau = c + \sigma * \tan(\phi)$$

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.

3.4.7 Unconfined compression test

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 (q_u) 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.

q_u= load/corrected area (A')

q_u- compressive stress

A'= cross-sectional area/ (1- ε)

CHAPTER- 4

RESULTS & DISCUSSIONS

4.1 Specific Gravity

Soil sample- 1

sample number	1	2	3
mass of empty bottle (M1) in gms.	128.41	118.67	122.16
mass of bottle+ dry soil (M2) in gms.	178.41	168.67	172.16
mass of bottle + dry soil + water (M3) in gms.	401.86	396.29	399.03
mass of bottle + water (M4) in gms.	369.67	365.378	367.355
specific gravity	2.81	2.62	2.73
Avg. specific gravity	**2.72**		

Table- 3

Soil sample- 2

sample number	1	2	3
mass of empty bottle (M1) in gms.	112.45	114.93	115.27
mass of bottle+ dry soil (M2) in gms.	162.45	164.93	165.27
mass of bottle + dry soil + water (M3) in gms.	390.088	395.38	398.16
mass of bottle + water (M4) in gms.	359.448	364.07	367.87
specific gravity	2.58	2.68	2.54
Avg. specific gravity	**2.60**		

Table- 4

4.2 Index Properties

4.2.1 Liquid Limit

Soil sample- 1

Sample No.	1	2	3	4	5
Mass of empty can	13.00	12.38	13.58	12.56	13.4
Mass of can + wet soil in gms.	50.70	47.60	48.00	36.60	50.00
Mass of can + dry soil in gms.	42.60	39.70	40.40	31.20	41.70
Mass of soil solids	29.60	27.32	26.82	18.64	28.30
Mass of pore water	8.10	7.90	7.60	5.40	8.30
Water content (%)	27.40	28.90	28.30	29.00	29.30
No. of blows	30	25	24	21	16

Table- 5

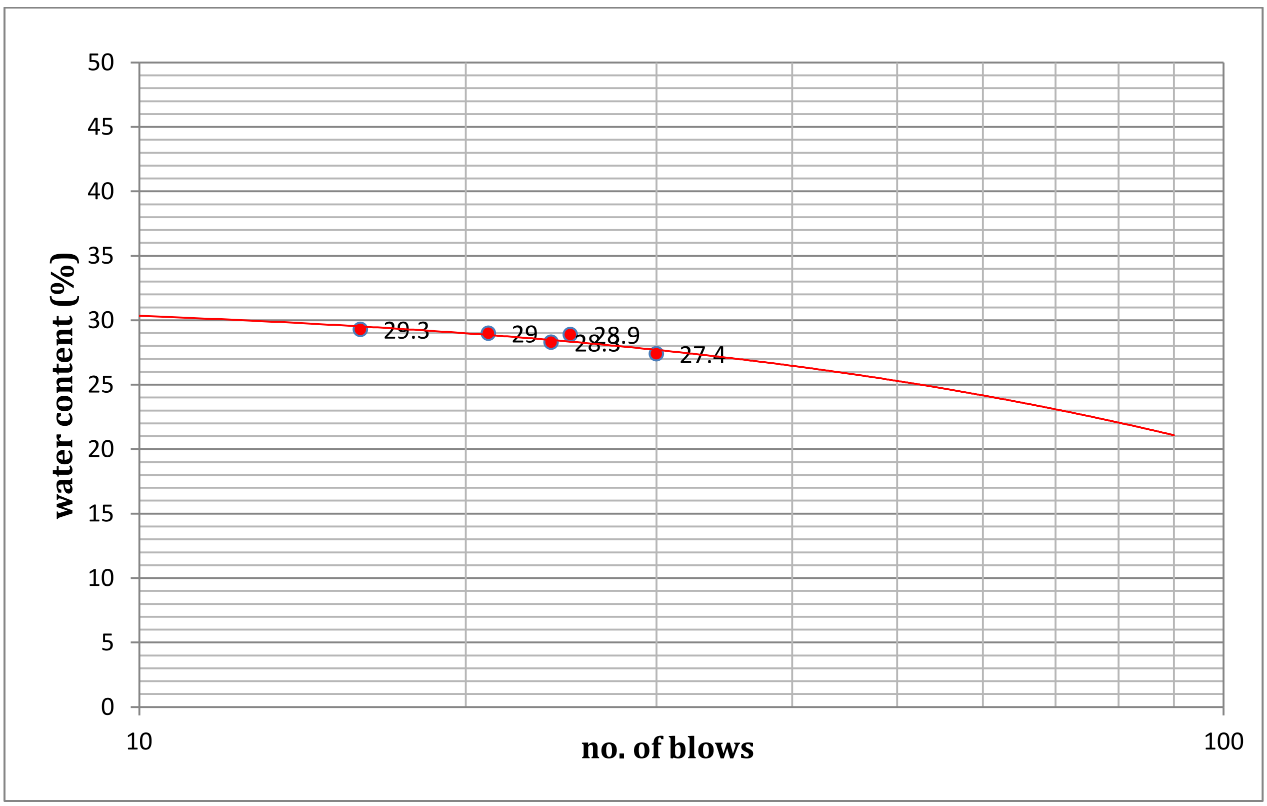

Fig.- 3

Liquid limit as obtained from graph = **28.90**
(corresponding to 25 blows)

Soil sample- 2

Sample No.	1	2	3	4	5
Mass of empty can	13.24	12.56	13.53	13.26	12.96
Mass of can + wet soil in gms.	54.92	53.02	53.06	45.12	51.48
Mass of can + dry soil in gms.	42.00	40.68	41.28	35.74	39.65
Mass of soil solids	28.76	28.12	27.75	22.53	26.69
Mass of pore water	12.92	12.34	11.78	9.33	11.83
Water content (%)	44.95	43.91	42.45	41.40	44.33
No. of blows	18	23	30	35	21

Table- 6

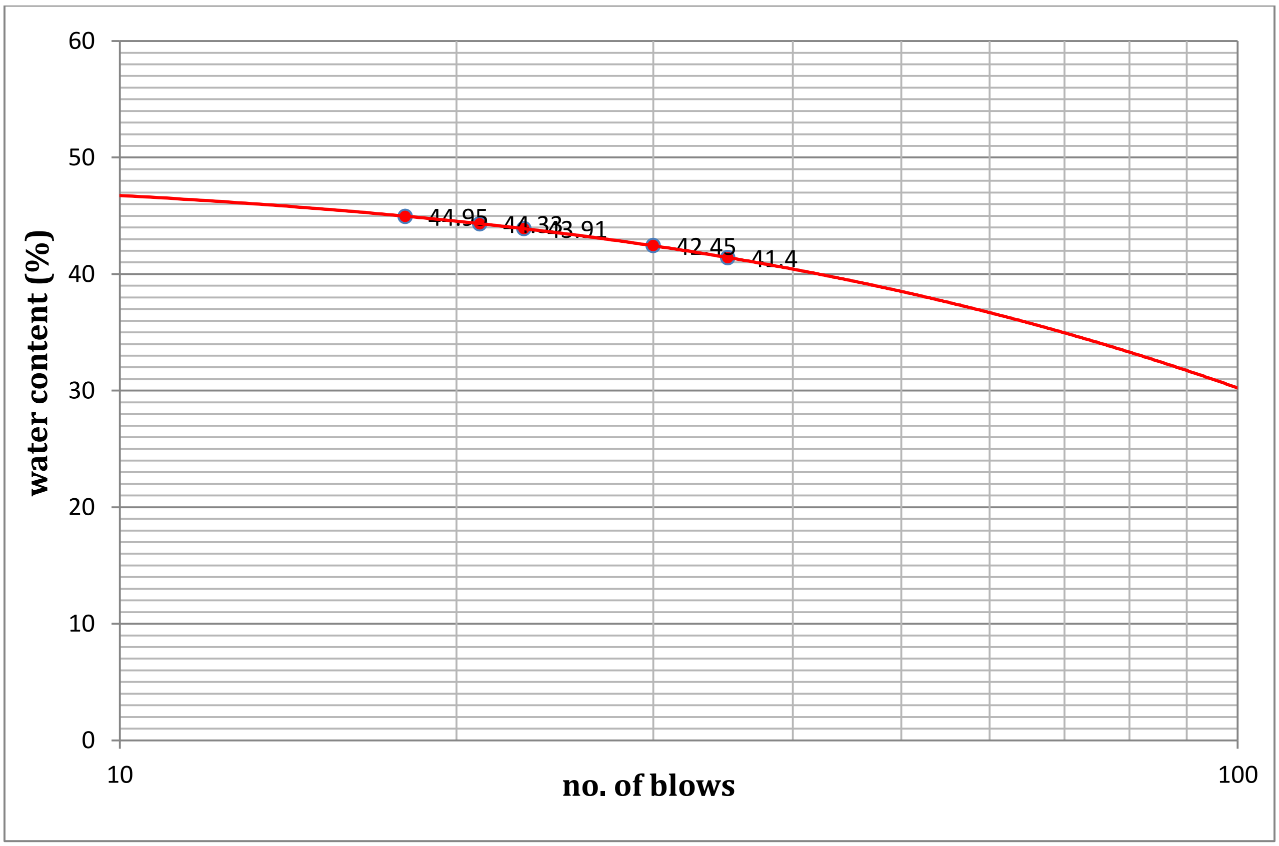

Fig.- 4

Liquid limit as obtained from graph = **43.491**
(corresponding to 25 blows)

4.2.2 Plastic Limit

Soil sample- 1

Sample No.	1	2	3
Mass of empty can	5.54	5.86	5.47
Mass of (can+wet soil) in gms.	9.4	10.6	9.9
Mass of (can + dry soil) in gms.	8.7	9.7	9.1
Mass of soil solids	3.1	3.8	3.6
Mass of pore water	0.7	0.9	0.8
Water content (%)	22.38	23.43	21.94
Average Plastic Index	**22.58**		

Table- 7

Soil sample-2

Sample No.	1	2	3
Mass of empty can	5.62	5.67	5.76
Mass of (can+wet soil) in gms.	10.60	9.80	9.50
Mass of (can + dry soil) in gms.	9.80	9.10	8.90
Mass of soil solids	4.18	3.43	3.14
Mass of pore water	0.80	0.70	0.60
Water content (%)	19.14	20.41	19.12
Average Plastic Index	**19.56**		

Table- 8

4.2.3 Plasticity Index

Soil sample- 1

$I_p = W_L - W_P = 28.90 - 22.58 = 6.32$

Soil sample- 2

$I_p = W_L - W_P = 43.91 - 19.56 = 24.35$

According to USUC classification of soils,

Soil sample- 1
ML: silt, low plasticity

Soil sample- 2
CL: clay, low plasticiy

4.3 Particle Size Distribution

Soil sample- 1

Sieve size	Retained (g)	Retained (%)	Cumulative retained (%)	Cumulative finer (%)
20	0	0	0	100
10	83.98	9.94	9.94	90.06
6.25	126.41	14.96	24.90	74.40
4.75	64.15	7.59	32.49	60.39
2	447.58	52.97	85.46	22.00
1	18.94	2.24	87.70	12.3
0.425	29.91	2.83	90.53	9.471
0.15	9.76	1.16	91.69	8.32
0.075	5.96	0.7	92.39	7.61
<0.075	64	7.57	99.96	0.04

Table- 9

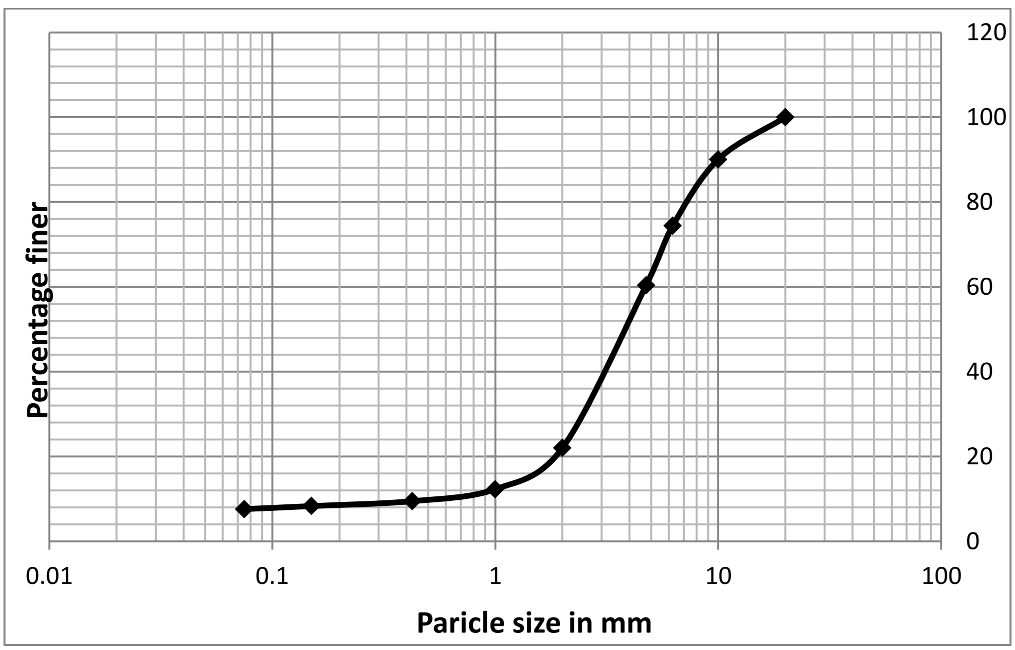

Fig. -5

Uniformity Coefficient= 7.9/5.8 = 1.362

Soil sample- 2

Sieve size	Retained (g)	Retained (%)	Cumulative retained (%)	Cumulative finer (%)
20	0	0	0	100
10	84.04	9.88	9.88	90.12
6.25	125.39	14.75	24.63	76.37
4.75	63.97	7.52	32.15	67.85
2	445.92	52.46	84.61	18.01
1	19.21	2.26	86.87	13.13
0.42	29.86	3.51	90.38	6.02
0.15	9.53	1.12	91.5	8.5
0.075	6.17	0.72	92.22	7.78
<0.075	66	7.76	99.78	0.02

Table- 10

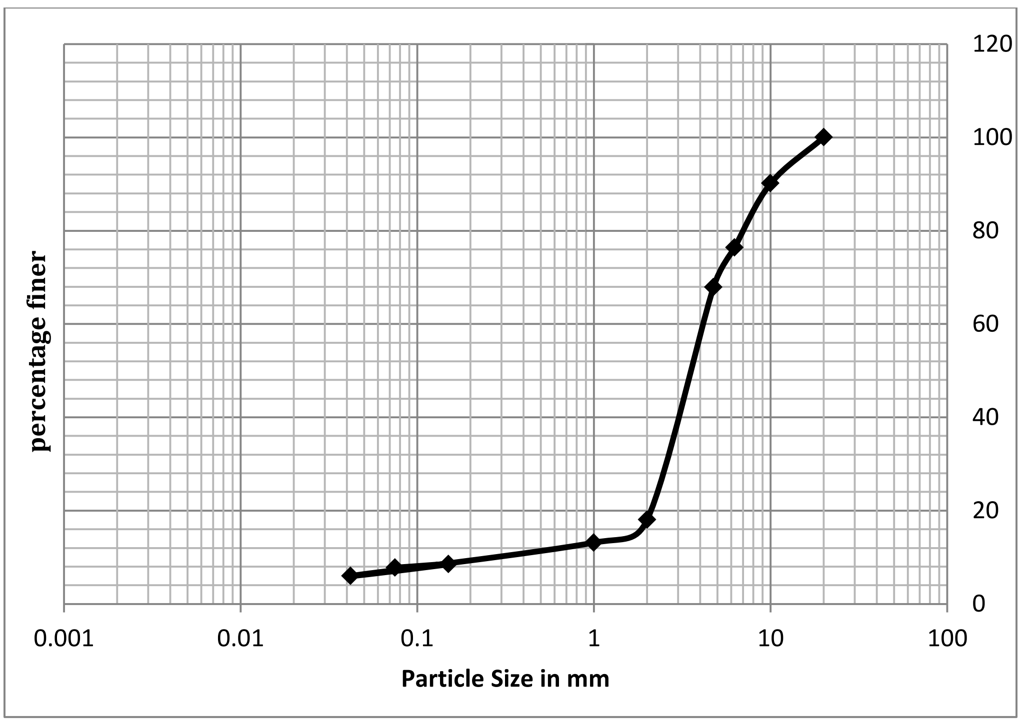

Fig. - 6

Uniformity Coefficient = 7.9/5.8 = 1.362

4.4 Standard Proctor Compaction Test

Soil Sample- 1

Test No.	1	2	3	4	5
Weight of empty mould(W_m) gms	2059	2059	2059	2059	2059
Internal diameter of mould (d) cm	10	10	10	10	10
Height of mould (h) cm	13	13	13	13	13
Volume of mould (V)=(π/4) d^2h cc	1000	1000	1000	1000	1000
Weight of Base plate (W_b) gms	2065	2065	2065	2065	2065
Weight of empty mould + base plate (W') gms	4124	4124	4124	4124	4124
Weight of mould + compacted soil + Base plate (W_1) gms	6089	6179	6271	6086	6080
Weight of Compacted Soil (W_1-W') gms	1965	2055	2147	2108	2102
Container no.	20.15	21.15	19.47	21.49	21.12
Weight of Container (X_1) gms	20.19	21.14	19.48	21.55	21.14
Weight of Container + Wet Soil (X_2) gms	84.81	124.16	89.93	154	113
Weight of Container + dry soil (X_3) gms	79.59	114.24	82.05	138.13	100.5
Weight of dry soil (X_3-X_1) gms	59.4	93.1	62.57	116.58	79.36
Weight of water (X_2-X_3) gms	5.22	9.92	7.88	15.87	12.5
Water content W%= X_2-X_3/X_3-$_1$	8.79	10.65	12.59	13.61	15.75
Dry density Υ_d= V_t/1 + (W/100) gm/cc	1.81	1.86	1.91	1.85	1.82

Table- 11

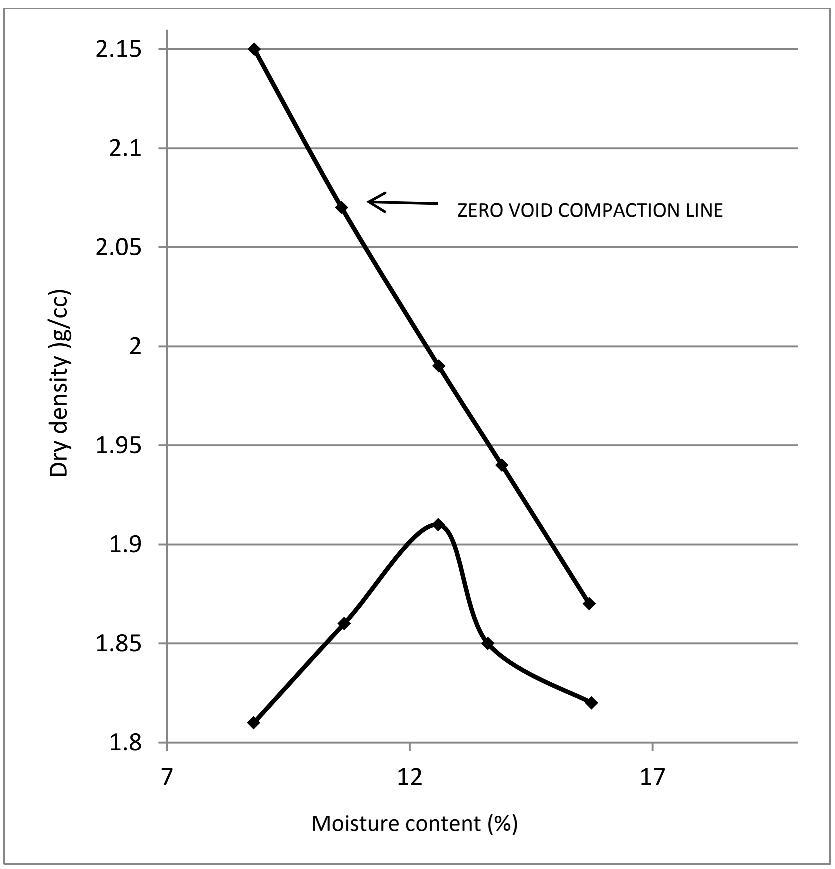

Fig. - 7

From the figure on the left side, it is evident that,

Optimum Moisture Content (OMC) = 12.6%

Maximum Dry Density (MDD) = 1.91 g/cc

Soil sample- 2

Test No.	1	2	3	4	5
Weight of empty mould(Wm) gms	2062	2062	2062	2062	2062
Internal diameter of mould (d) cm	10	10	10	10	10
Height of mould (h) cm	13	13	13	13	13
Volume of mould (V)=(π/4) d2h cc	1000	1000	1000	1000	1000
Weight of Base plate (Wb) gms	2071	2071	2071	2071	2071
Weight of empty mould + base plate (W') gms	4133	4133	4133	4133	4133
Weight of mould + compacted soil + Base plate (W1) gms	6174	6261	6427	6347	6348
Weight of Compacted Soil (W1-W') gms	2041	2128	2294	2214	2215
Container no.	19.47	21.15	21.12	20.15	21.49
Weight of Container (X1) gms	19.49	21.6	21.14	20.19	21.55
Weight of Container + Wet Soil (X2) gms	90.21	122.57	113.12	125.00	119.28
Weight of Container + dry soil (X3) gms	82.51	110.04	99.74	108.94	102.32
Weight of dry soil (X3-X1) gms	63.02	88.87	78.6	88.75	80.77
Weight of water (X2-X3) gms	7.7	12.53	13.38	16.06	16.96
Water content W%= X2-X3/X3-X1	12.18	14.4	17.02	18.1	21
Dry density ϒd= ϒt/(1 + (W/100)) gm/cc	1.79	1.86	1.96	1.875	1.83

Table- 12

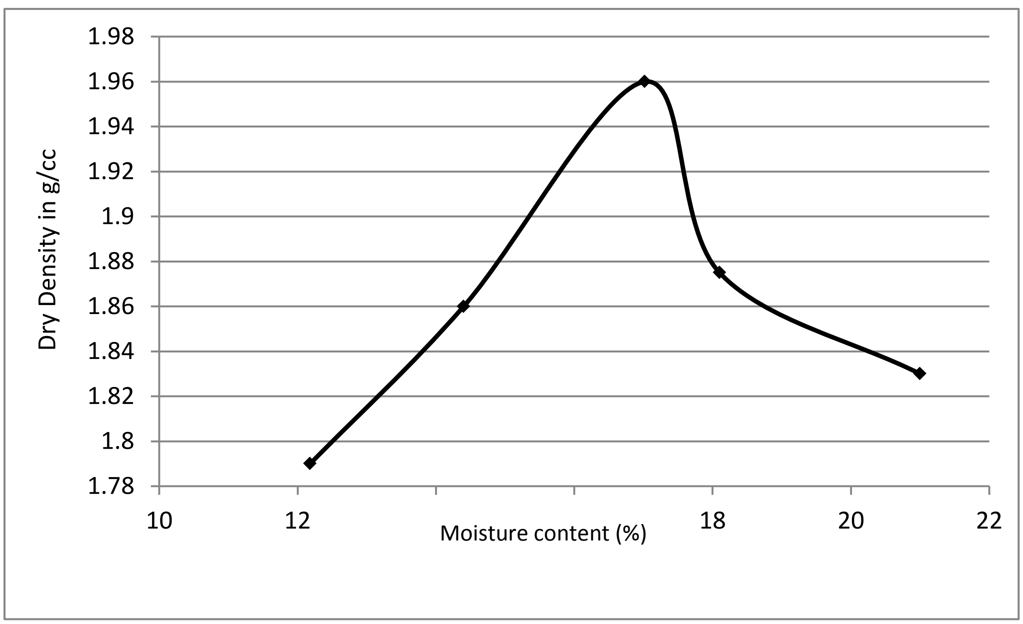

Fig. - 8

From the figure on the left side, it is evident that,

Optimum Moisture Content (OMC) = 17.02%

Maximum Dry Density (MDD) = 1.96 g/cc

4.5 Direct Shear Test

Soil sample- 1

Volume of shear Box	90 cm3
Maximum dry density of soil	1.91 gm/cc
Optimum moisture content of soil	12.6 %
Weight of the soil to be filled in the shear box	1.91x90 = 171.9 gm
Weight of water to be added	(12.6/100)x171.9= 21.66 gm

Table- 13

i) Unreinforced soil

Sample No.	Normal Stress(kg/cm^2)	Proving ring reading	Shear Load (N)	Shear Load (kg)	Shear Stress (kg/cm^2)
1	0.5	54	206.58	21.06	0.59
2	1	84	321.35	32.76	0.91
3	1.5	106	405.51	41.34	1.14
4	2	168	451.42	46.02	1.27

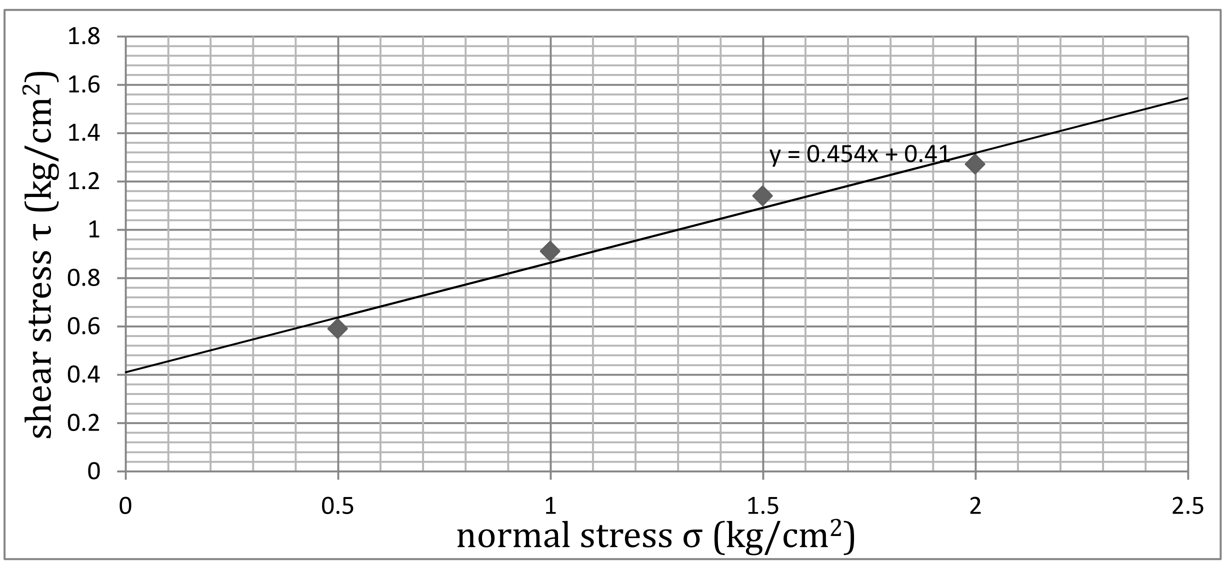

Fig. - 9

Computing from graph,

Cohesion (C) = 0.325 kg/cm^2 ; Angle of internal friction (φ) = 47.72°

ii) Reinforcement = 0.05%

Sample no.	Normal load (σ)	Proving constant	Shear load (N)	Shear load (kg)	Shear stress (kg/cm^2)
1	0.5	76	290.27	29.62	0.83
2	1.0	120	458.19	46.75	1.31
3	1.5	160	612.08	62.45	1.75
4	2.0	206	786.96	80.30	2.25

Table- 14

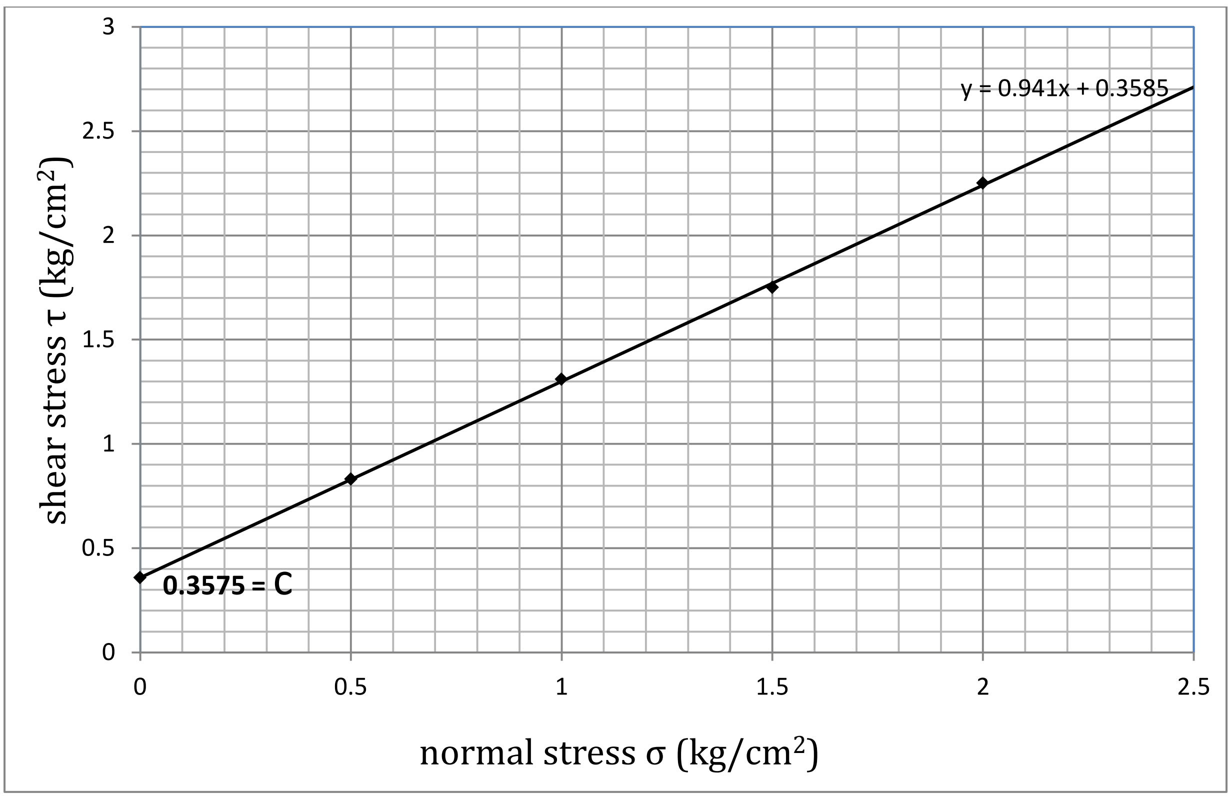

Fig. - 10

Computing from graph,

Cohesion (C) = 0.3575 kg/cm^2

Angle of internal friction (φ) = 48.101°

iii) Reinforcement = 0.15%

Sample no.	Normal load (σ)	Proving constant	Shear load (N)	Shear load (kg)	Shear stress (kg/cm^2)
1	0.5	78	297.23	30.33	0.85
2	1.0	121	461.68	47.11	1.32
3	1.5	164	626.07	63.88	1.79
4	2.0	207	793.99	81.02	2.27

Table- 15

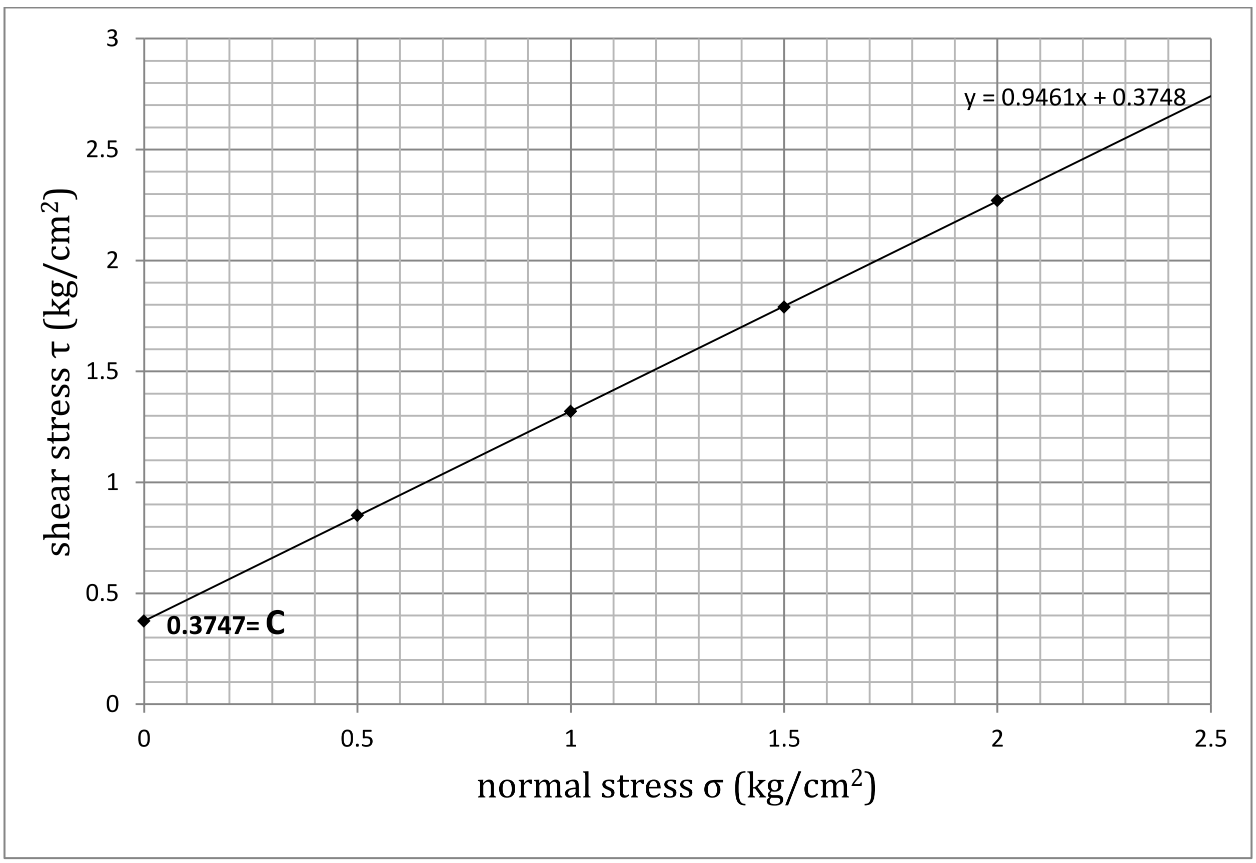

Fig. - 11

Computing from graph,

Cohesion (C) = 0.3747 kg/cm^2

Angle of internal friction (φ) = 48.254°

Reinforcement = 0.25%

Sample no.	Normal load (σ)	Proving constant	Shear load (N)	Shear load (kg)	Shear stress (kg/cm^2)
1	0.5	79	300.79	30.69	0.86
2	1.0	122	468.64	47.82	1.34
3	1.5	166	636.61	64.96	1.82
4	2.0	209	800.95	81.73	2.29

Table- 16

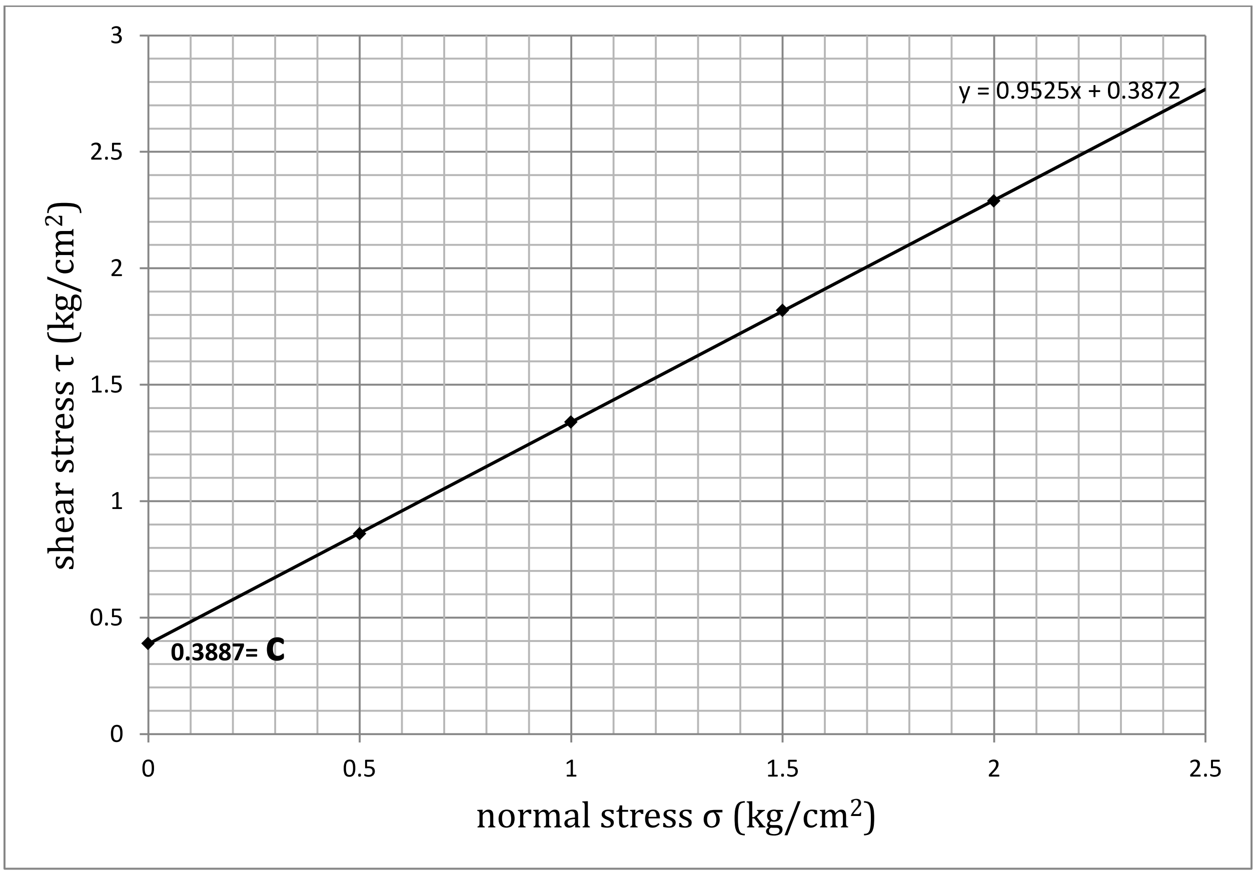

Fig. -12

Computing from graph,

Cohesion (C) = 0.3887 kg/cm^2

Angle of internal friction (φ) = 48.483°

Soil sample-2

Volume of shear box	90 cm3
Maximum Dry Density	1.96 g/cc
Optimum Moisture Content of soil	17.02%
Weight of the soil to be filled in the shear box	90*1.96= 176.4 gms.
Weight of water to be added	30.0238 gms.

Table- 17

i) Unreinforced

Sample no.	Normal load (σ)	Proving constant	Shear load (N)	Shear load (kg)	Shear stress (kg/cm²)
1	0.5	53	202.86	20.70	0.58
2	1.0	75	286.74	29.26	0.82
3	1.5	96	367.20	37.47	1.05
4	2.0	117	447.66	45.68	1.28

Table- 18

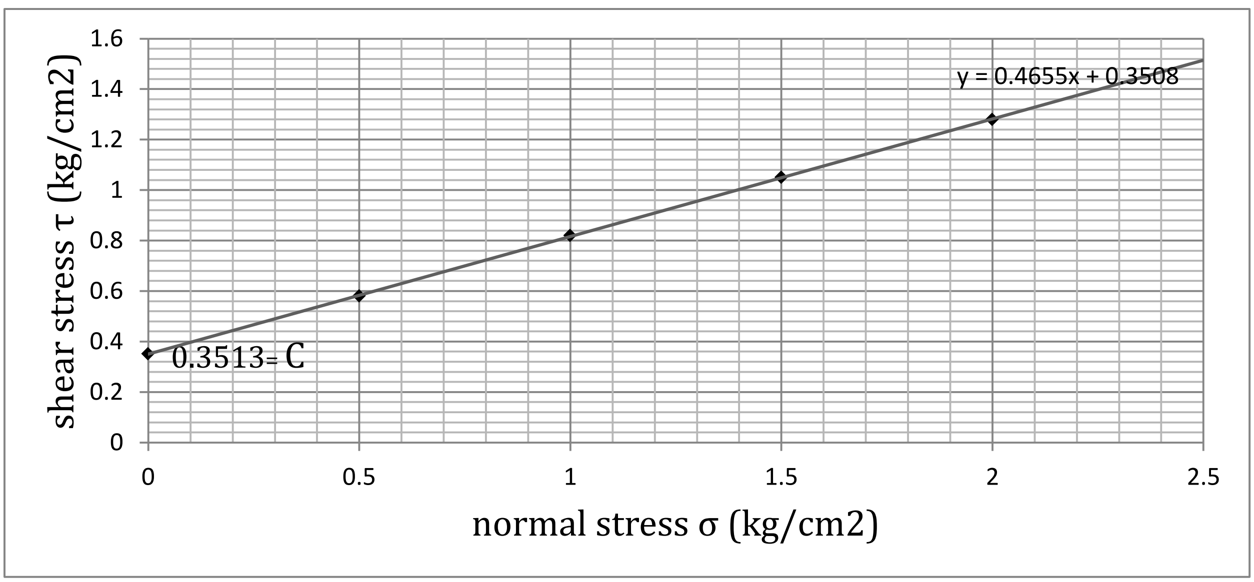

Fig. - 13

Computing from graph,

Cohesion (C) = 0.3513 kg/cm² ; Angle of internal friction (ϕ) = 27.82°

ii) Reinforcement = 0.05%

Sample no.	Normal load (σ)	Proving ring reading	Shear load (N)	Shear load (kg)	Shear stress (kg/cm²)
1	0.5	66	252.11	25.70	0.72
2	1.0	88	336.09	34.26	0.96
3	1.5	111	427.13	43.54	1.22
4	2.0	130	497.17	50.68	1.42

Table- 19

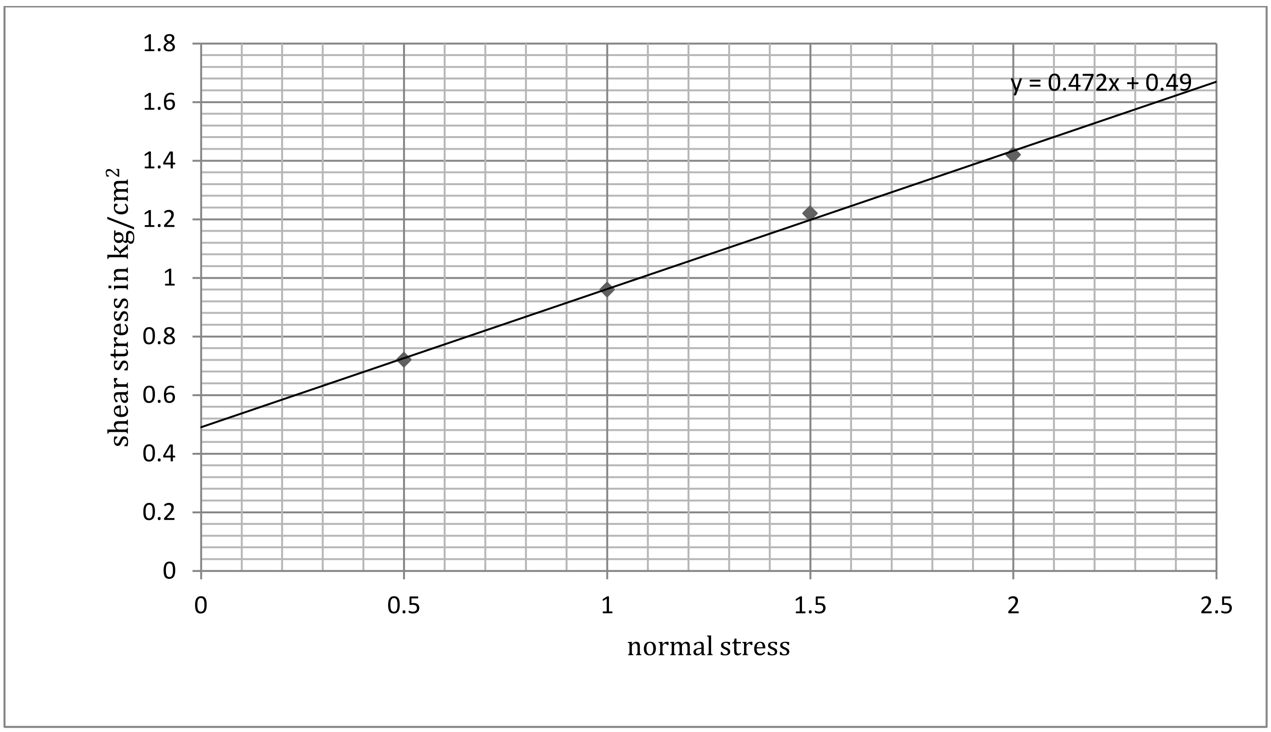

Fig. -14

Computing from graph,

Cohesion (C) = 0.4732 kg/cm²

Angle of internal friction (ϕ) = 29.02°

iii) Reinforcement = 0.15%

Sample no.	Normal load (σ)	Proving ring reading	Shear load (N)	Shear load (kg)	Shear stress (kg/cm^2)
1	0.5	72	275.46	28.11	0.788
2	1	99	378.75	38.65	1.083
3	1.5	126	482.05	49.19	1.378
4	2	151	577.7	58.93	1.651

Table- 20

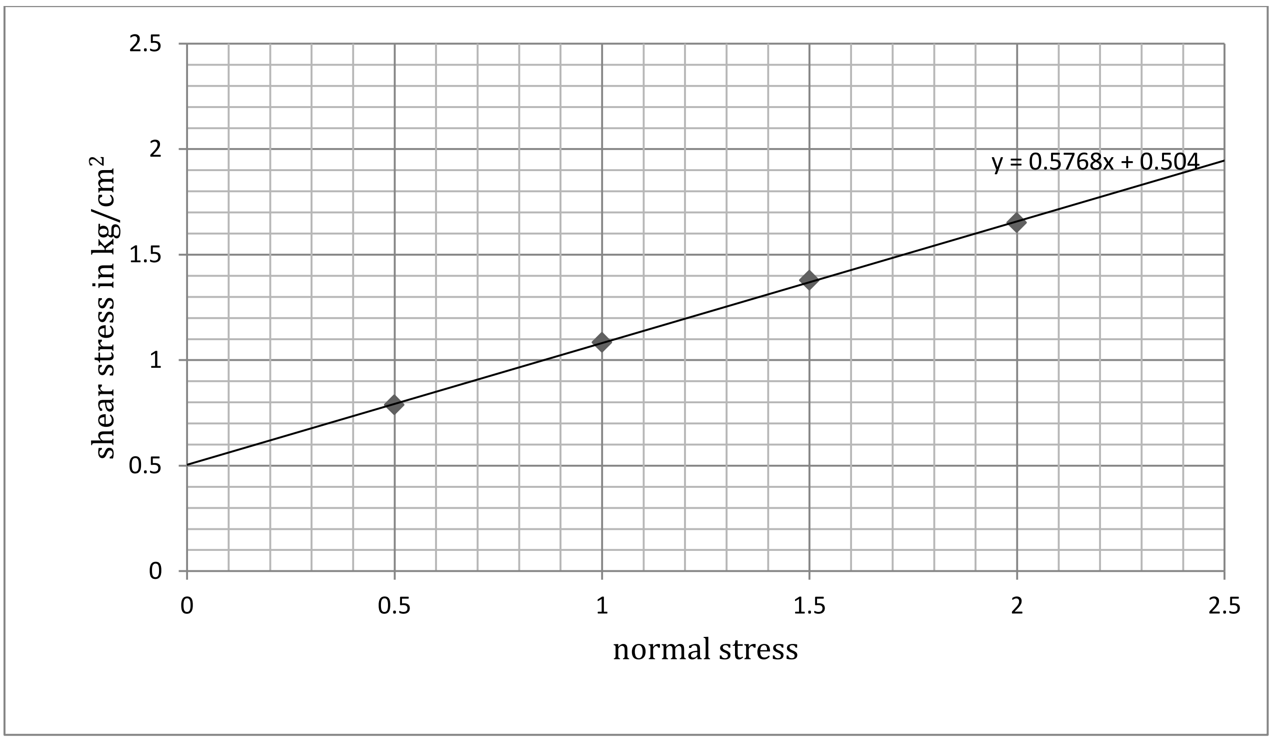

Fig. -15

Computing from graph,

Cohesion (C) = 0.504 kg/cm^2

Angle of internal friction (ϕ) = 29.95°

iv) Reinforcement = 0.25%

Sample no.	Normal load (σ)	Proving ring reading	Shear load (N)	Shear load (kg)	Shear stress (kg/cm^2)
1	0.5	78	298.41	30.45	0.85
2	1	107	409.36	41.77	1.17
3	1.5	137	524.69	53.54	1.5
4	2	164	626.02	63.88	1.79

Table- 21

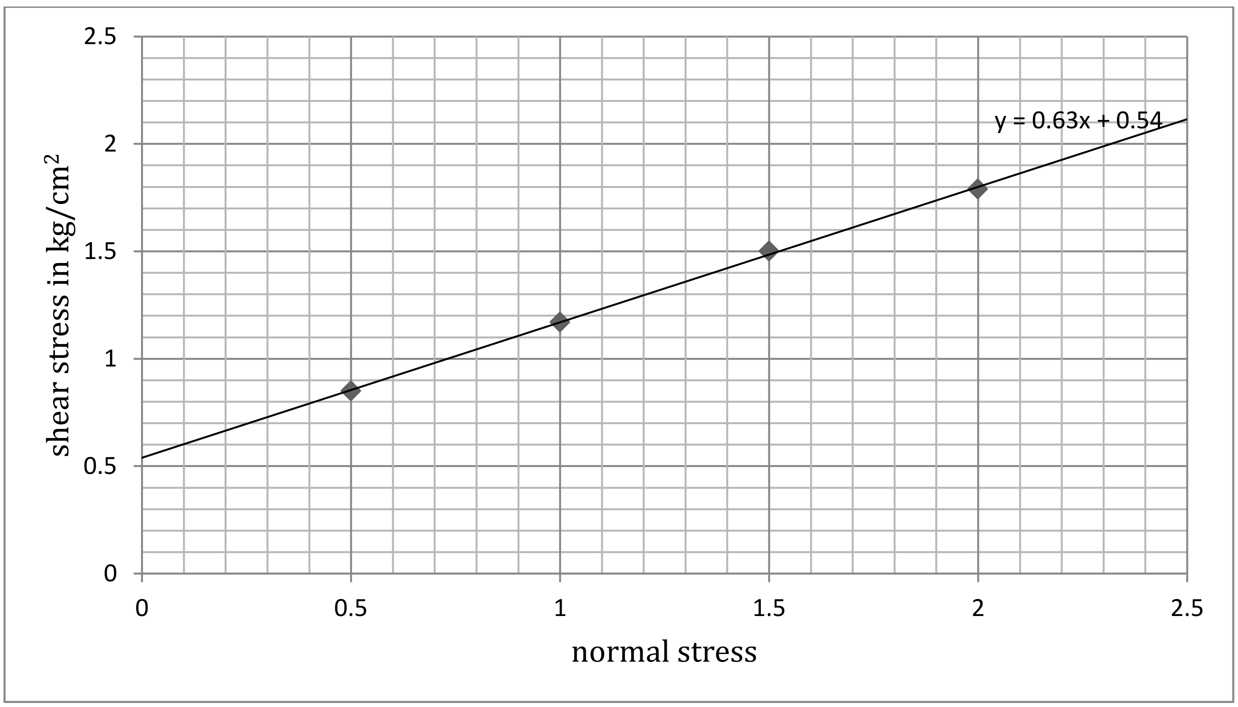

Fig. - 16

Computing from graph,

Cohesion (C) = 0.5375 kg/cm^2

Angle of internal friction (ϕ) = 32°

4.6 Unconfined Compression Strength Test

Soil sample- 1

i) Unreinforced

Dial gauge reading	Strain(ϵ)	Proving ring reading	corrected area	load (N)	Axial Stress (Mpa)
50	0.0033	35	19.72	40.81	0.0207
100	0.0067	62	19.82	69.19	0.0349
150	0.0100	79	19.92	92.11	0.0462
200	0.0133	91	20.03	106.12	0.0530
250	0.0167	98	20.13	114.27	0.0567
300	0.0200	93	20.24	108.44	0.0536
350	0.0233	85	20.34	99.11	0.0487

Table- 22

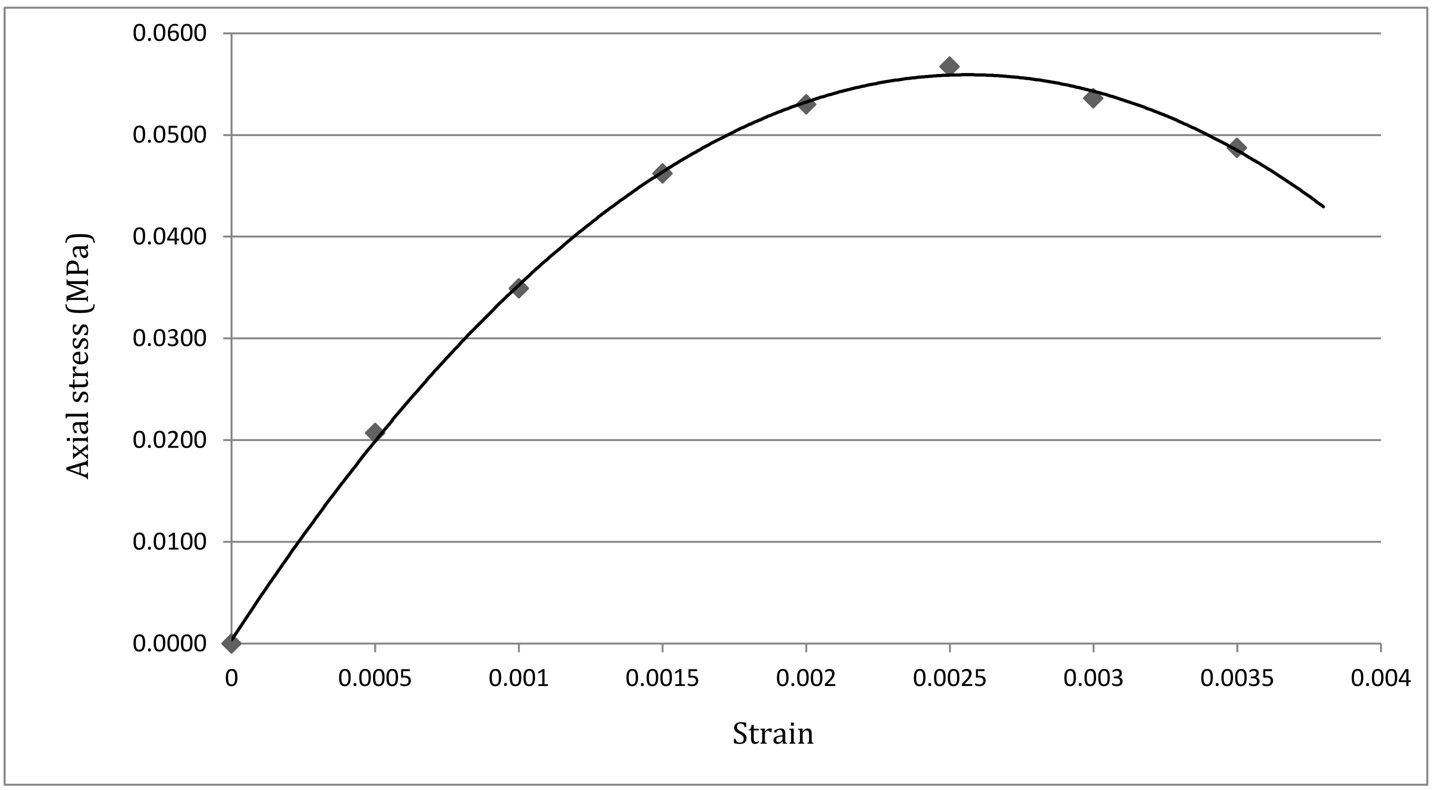

Fig. - 17

As obtained from graph,

UCS = 0.0562 MPa

ii) Reinforcement = 0.05%

Dial gauge reading	Strain(ϵ)	Proving ring reading	corrected area	load (N)	Axial Stress (Mpa)
50	0.0033	48	19.72	55.97	0.0284
100	0.0067	65	19.82	75.79	0.0382
150	0.0100	93	19.92	108.44	0.0544
200	0.0133	102	20.03	118.93	0.0594
250	0.0167	109	20.13	127.09	0.0631
300	0.0200	105	20.24	122.43	0.0605
350	0.0233	96	20.34	111.94	0.0551

Table- 23

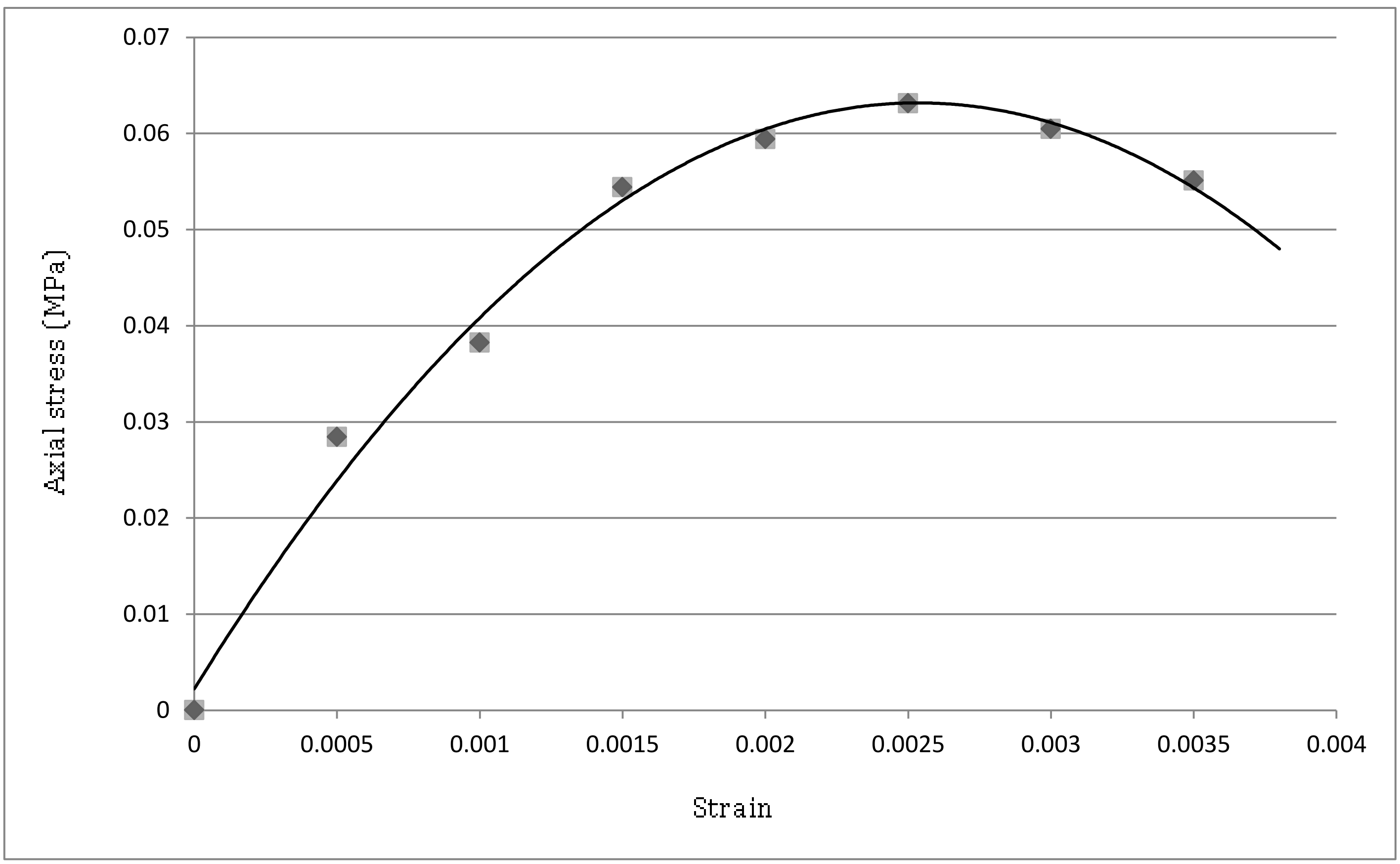

Fig. -18

As obtained from graph,

UCS = 0.0631 MPa

iii) Reinforcement = 0.15%

Dial gauge reading	Strain(ϵ)	Proving ring reading	corrected area	load (N)	Axial Stress (Mpa)
50	0.0033	47	19.72	54.8	0.0277
100	0.0067	71	19.82	82.79	0.0417
150	0.0100	94	19.92	109.6	0.0550
200	0.0133	105	20.03	122.43	0.0612
250	0.0167	110	20.13	128.26	0.0639
300	0.0200	103	20.24	120.1	0.0593
350	0.0233	92	20.34	107.27	0.0527

Table- 24

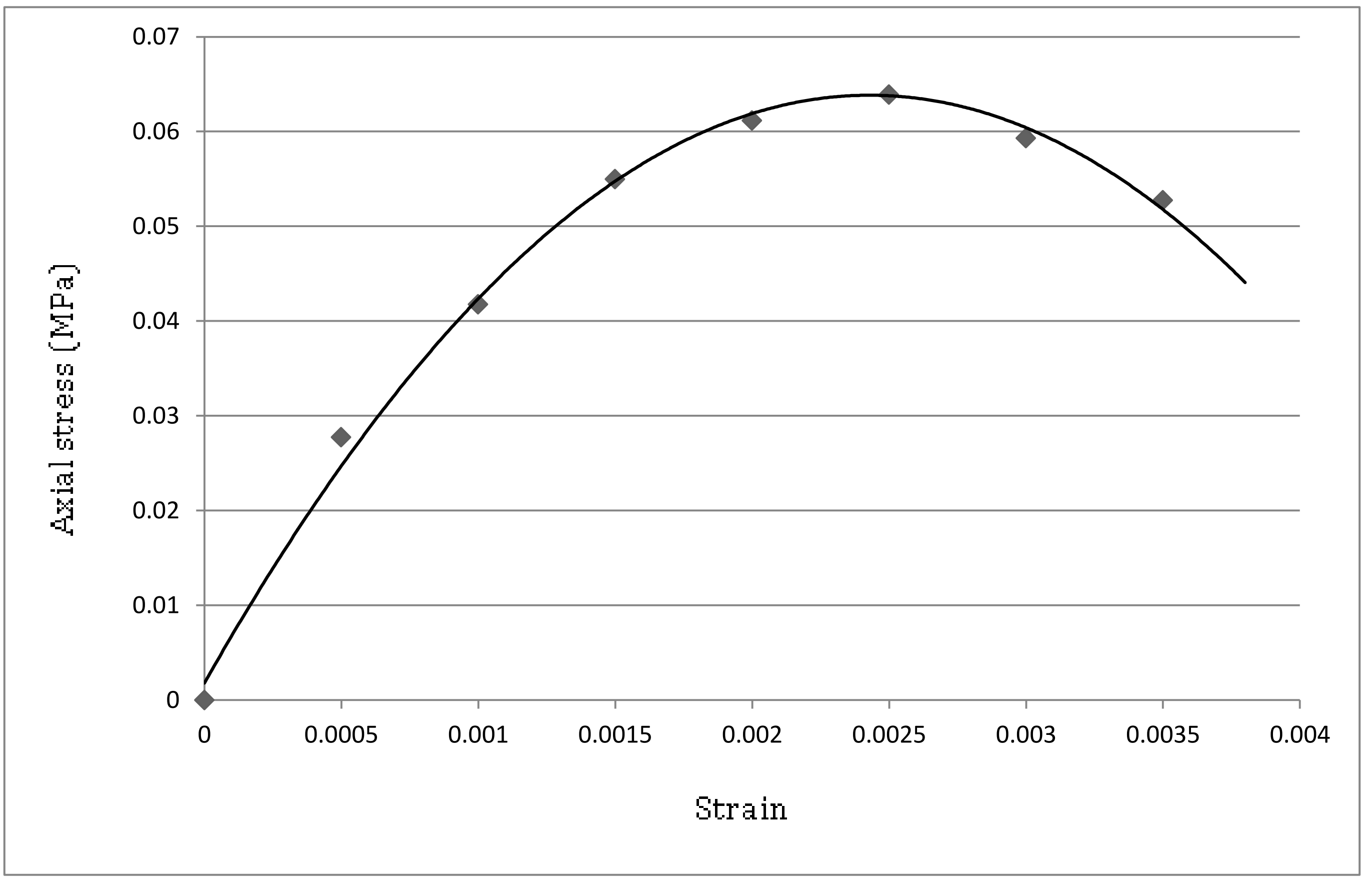

Fig. -19

As obtained from graph,

UCS = 0.0637 MPa

iv) Reinforcement = 0.25%

Dial gauge reading	Strain(ϵ)	Proving ring reading	corrected area	load (N)	Axial Stress (Mpa)
50	0.0033	51	19.72	59.47	0.0302
100	0.0067	69	19.82	80.45	0.0406
150	0.0100	94	19.92	109.6	0.0550
200	0.0133	105	20.03	122.43	0.0612
250	0.0167	111	20.13	129.43	0.0643
300	0.0200	106	20.24	123.6	0.0611
350	0.0233	93	20.34	108.44	0.0533

Table- 25

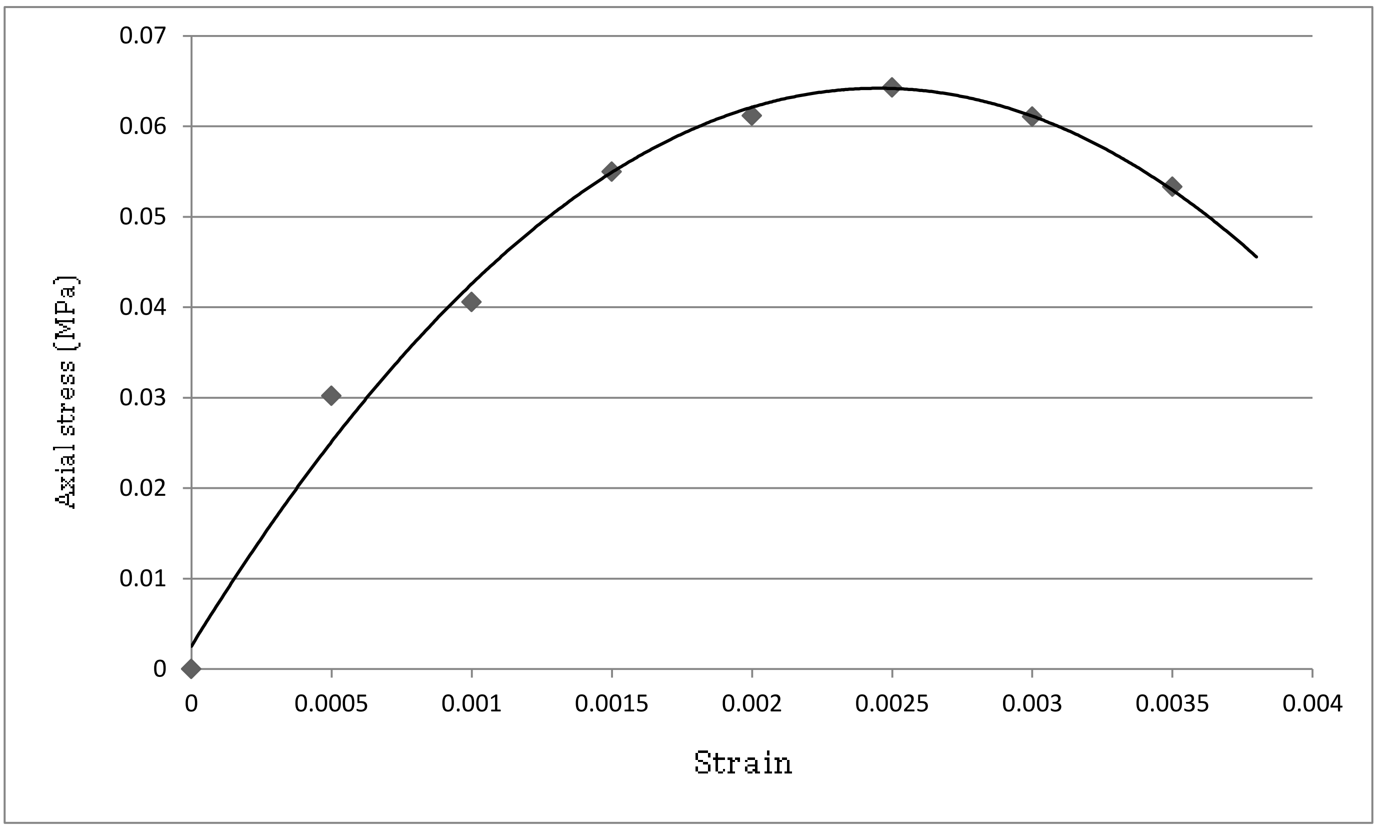

Fig. - 20

As obtained from graph,

UCS = 0.0643 MPa

Soil sample- 2

i) Unreinforced

Dial gauge reading	Strain(ϵ)	Proving ring reading	corrected area	load (N)	Axial Stress (Mpa)
50	0.0033	42	19.72	48.97	0.0248
100	0.0067	78	19.82	90.95	0.0459
150	0.0100	102	19.92	118.93	0.0597
200	0.0133	114	20.03	132.92	0.0663
250	0.0167	119	20.13	138.75	0.0689
300	0.0200	115	20.24	134.09	0.0662
350	0.0233	107	20.34	124.76	0.0613

Table- 26

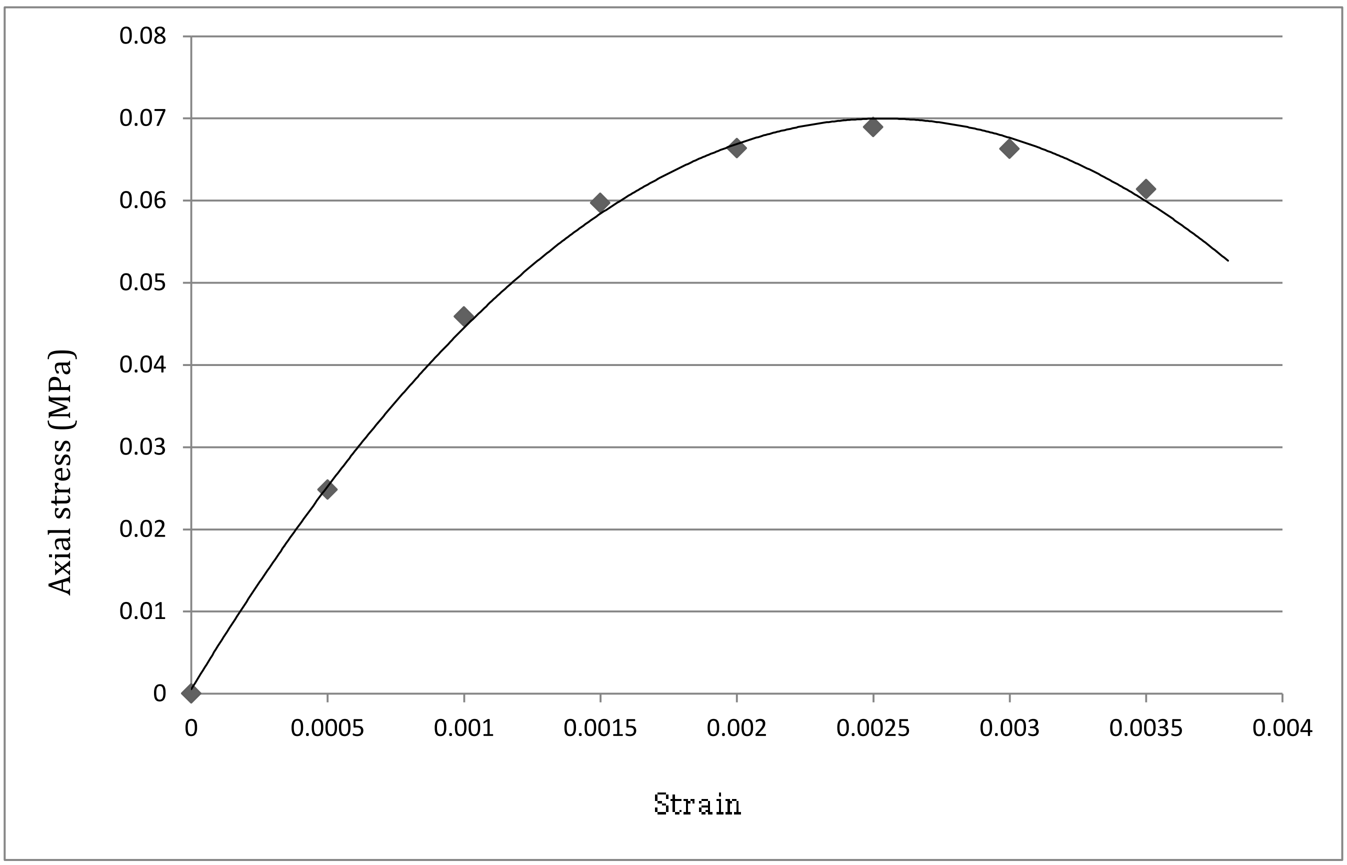

Fig. - 21

As obtained from graph,

UCS = 0.0692 MPa

ii) Reinforcement = 0.05%

Dial gauge reading	Strain(ϵ)	Proving ring reading	corrected area	load (N)	Axial Stress (Mpa)
50	0.0033	63	19.72	73.46	0.0372
100	0.0067	105	19.82	122.43	0.0617
150	0.0100	130	19.92	151.58	0.0760
200	0.0133	154	20.03	179.56	0.0897
250	0.0167	162	20.13	188.89	0.0938
300	0.0200	155	20.24	180.73	0.0893
350	0.0233	142	20.34	165.57	0.0814

Table- 27

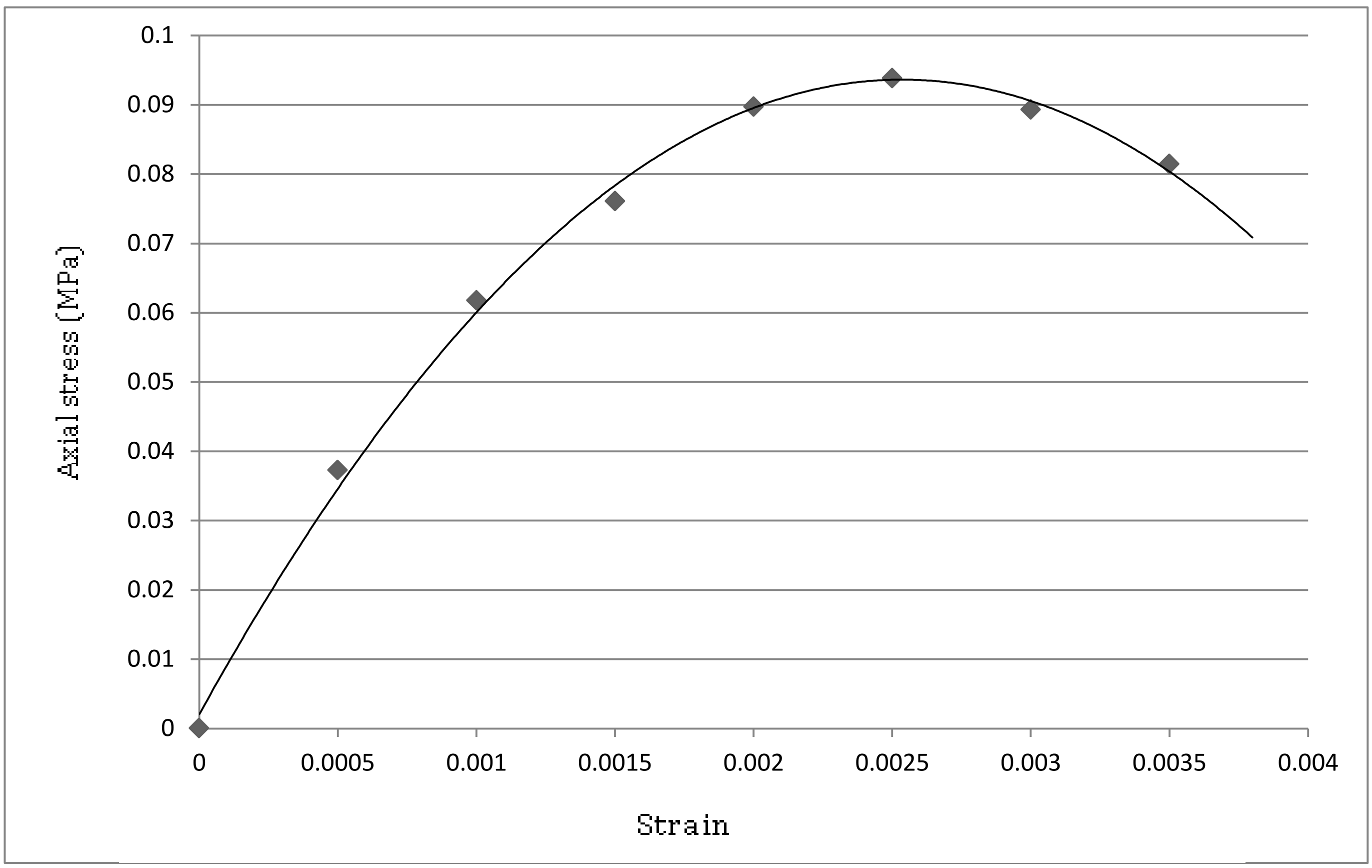

Fig. - 22

As obtained from graph,

UCS = 0.0938 MPa

iii) Reinforcement = 0.15%

Dial gauge reading	Strain(ϵ)	Proving ring reading	corrected area	load (N)	Axial Stress (Mpa)
50	0.0033	69	19.72	80.45	0.0408
100	0.0067	108	19.82	125.93	0.0635
150	0.0100	145	19.92	169.07	0.0849
200	0.0133	158	20.03	184.23	0.0919
250	0.0167	166	20.13	193.56	0.0961
300	0.0200	161	20.24	187.73	0.0927
350	0.0233	152	20.34	177.23	0.0871

Table- 28

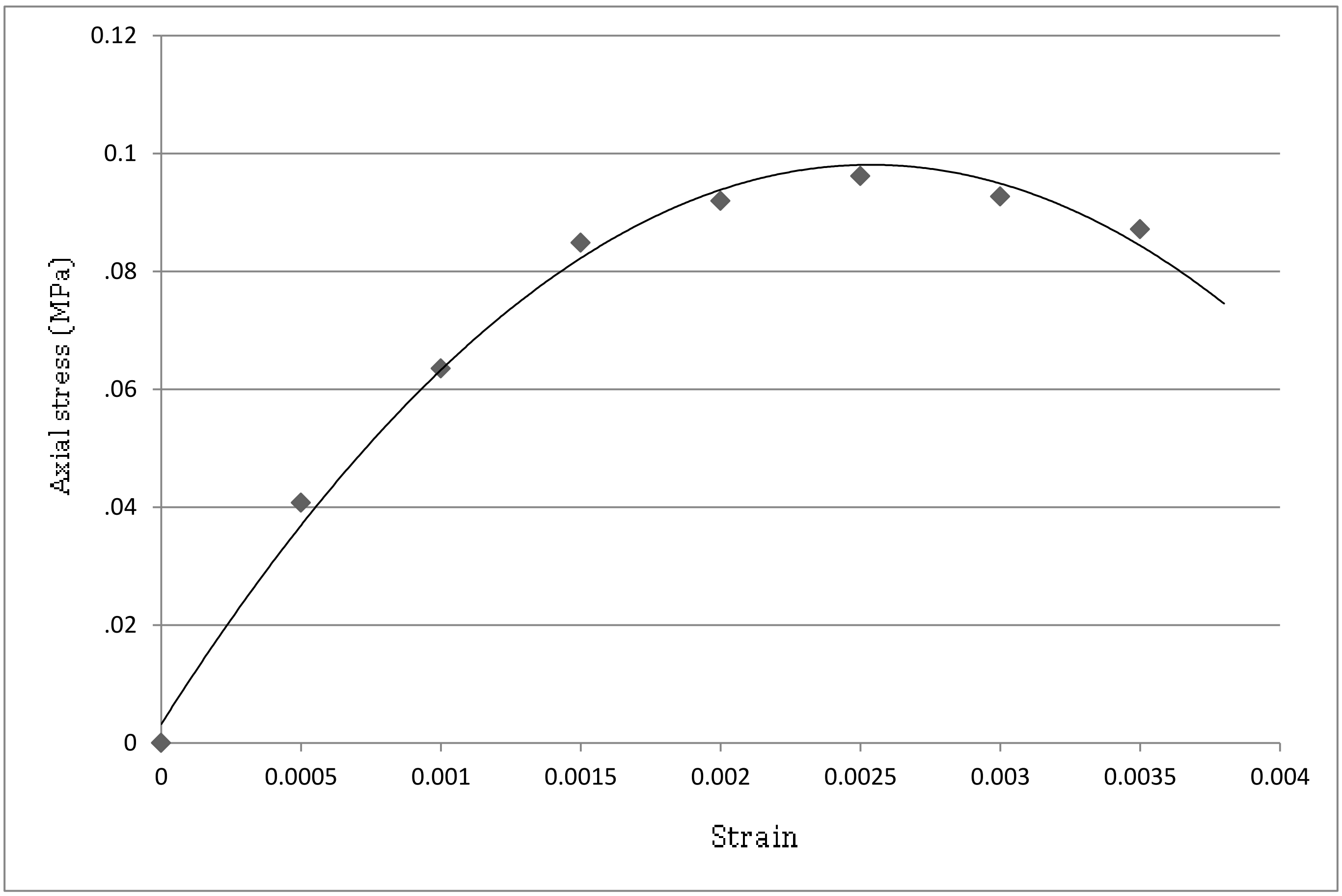

Fig. - 23

As obtained from graph,

UCS = 0.0965 MPa

iv) Reinforcement = 0.25%

Dial gauge reading	Strain(ϵ)	Proving ring reading	corrected area	load (N)	Axial Stress (Mpa)
50	0.0033	76	19.72	88.62	0.0449
100	0.0067	112	19.82	130.59	0.0659
150	0.0100	151	19.92	176.07	0.0884
200	0.0133	167	20.03	194.72	0.0972
250	0.0167	179	20.13	208.71	0.1037
300	0.0200	170	20.24	198.22	0.0979
350	0.0233	157	20.34	183.06	0.0900

Table- 29

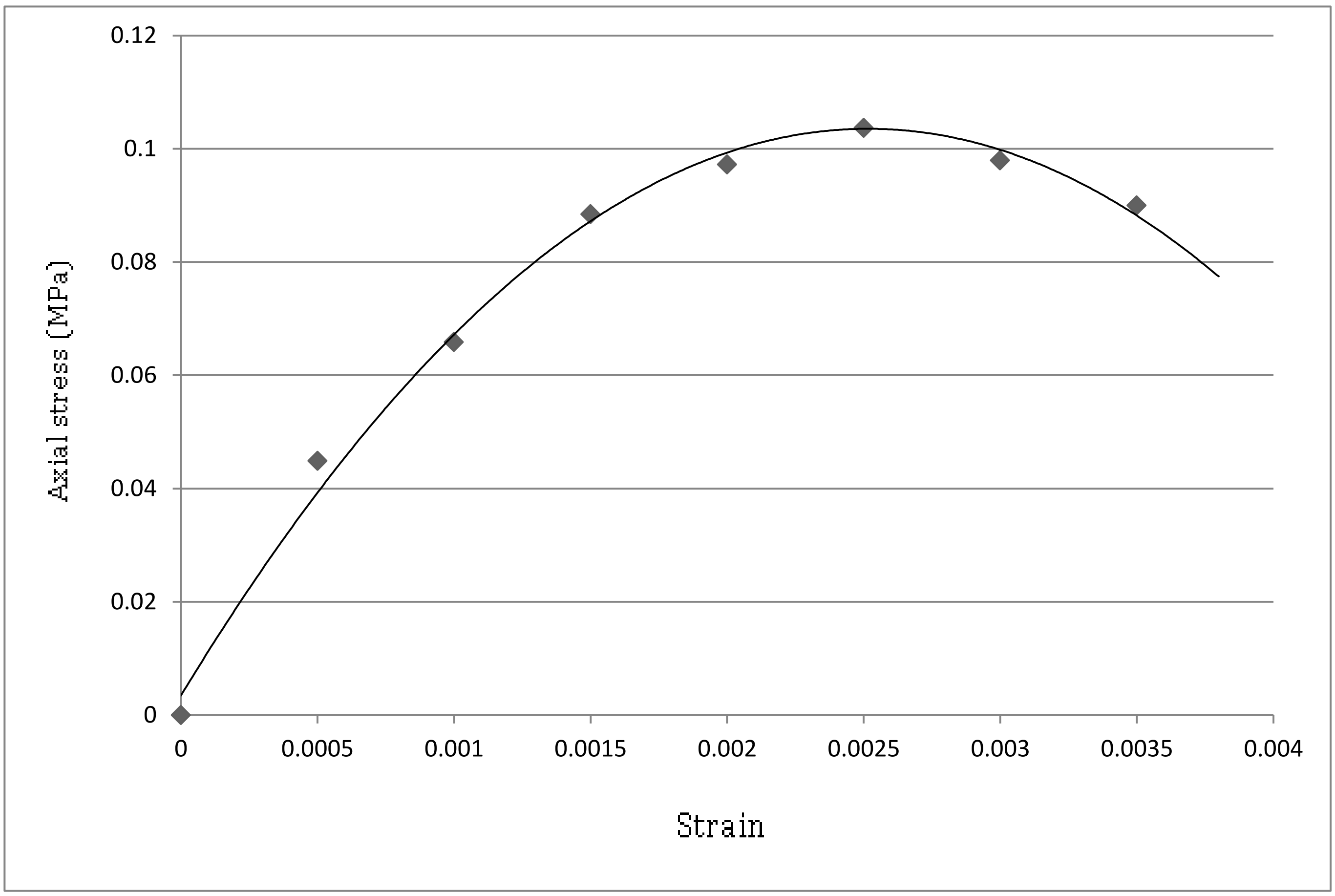

Fig. - 24

As obtained from graph,

UCS = 0.1037 MPa

4.7 Discussions

The relationship between shear strength parameters and fiber content-

(a) cohesion and fiber content

Soil sample- 1

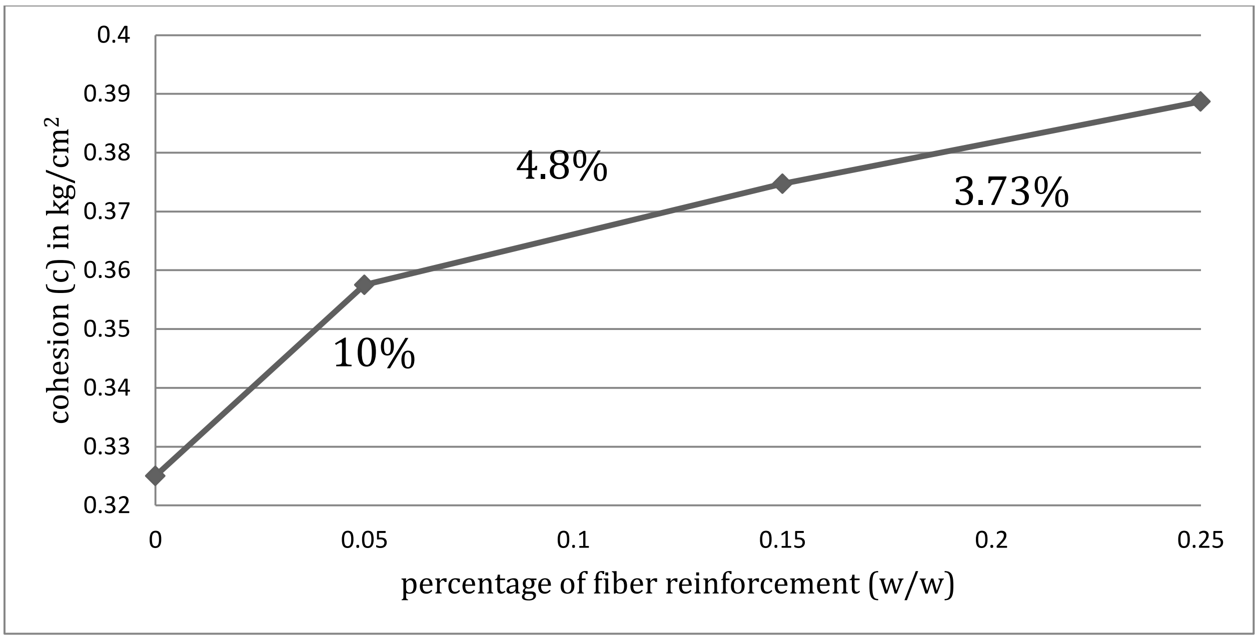

Fig. - 25

Soil sample- 2

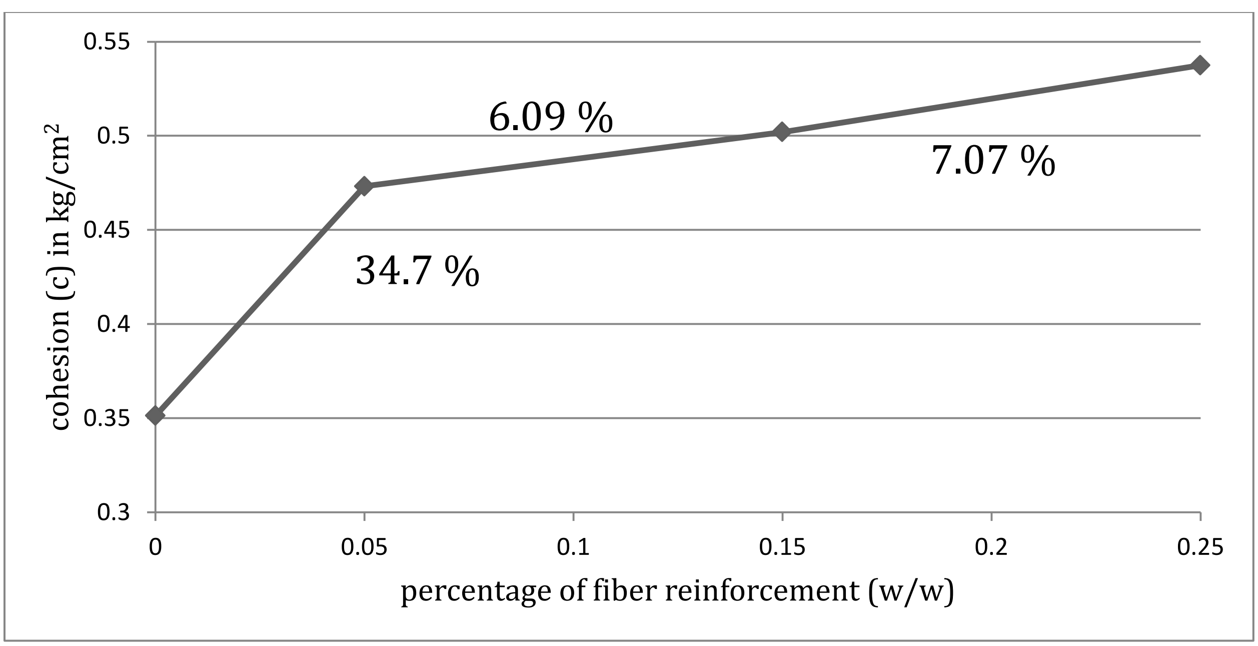

Fig. - 26

(b) angle of internal friction and fiber content

Soil sample- 1

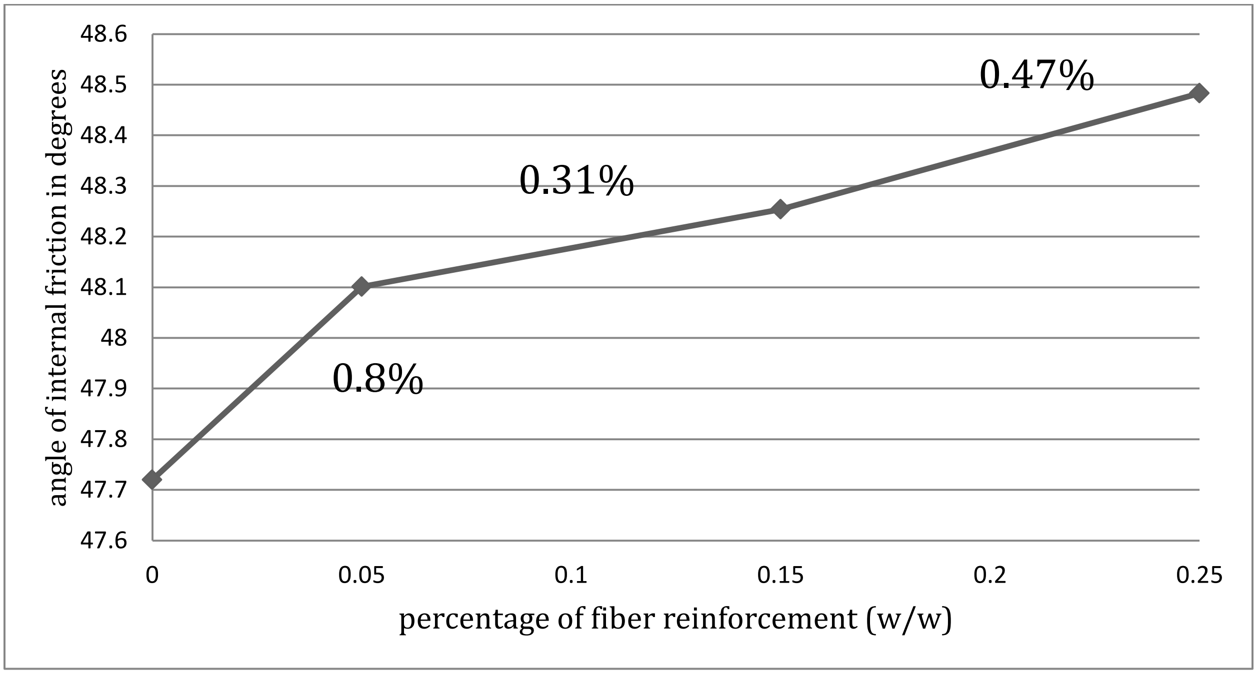

Fig. - 27

Soil sample- 2

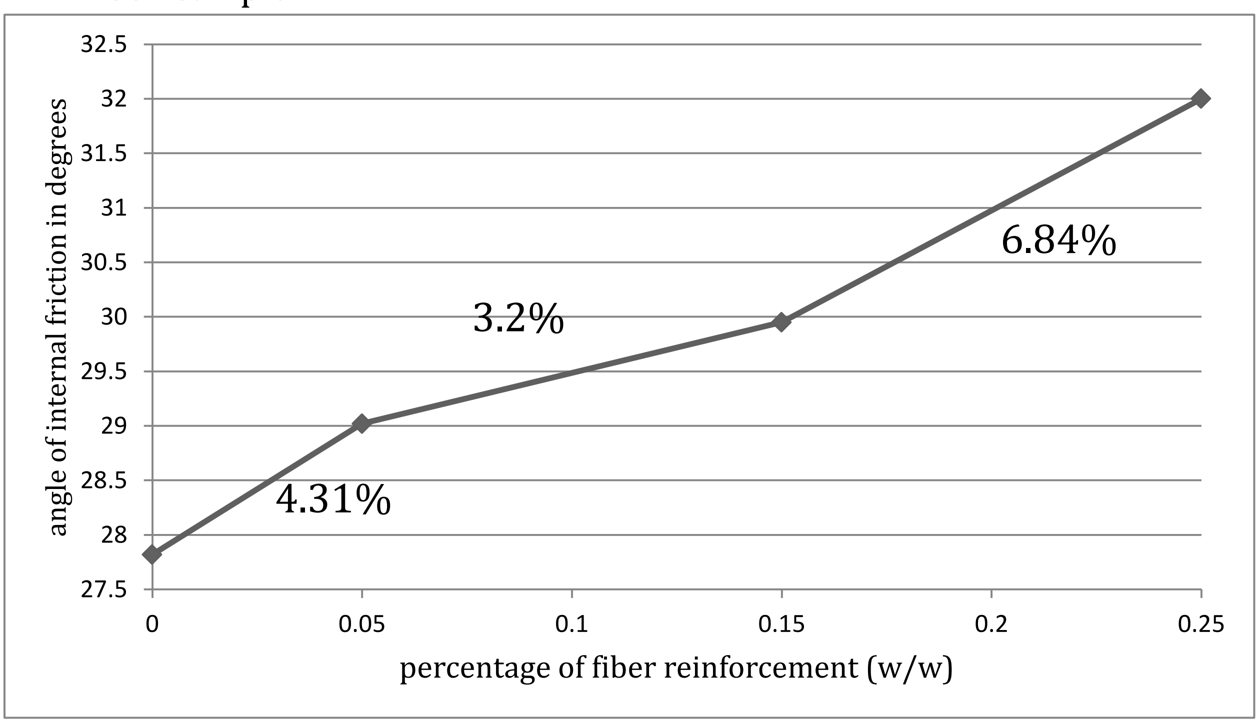

Fig. - 28

The relationship between the UCS and fiber content.

Soil sample- 1

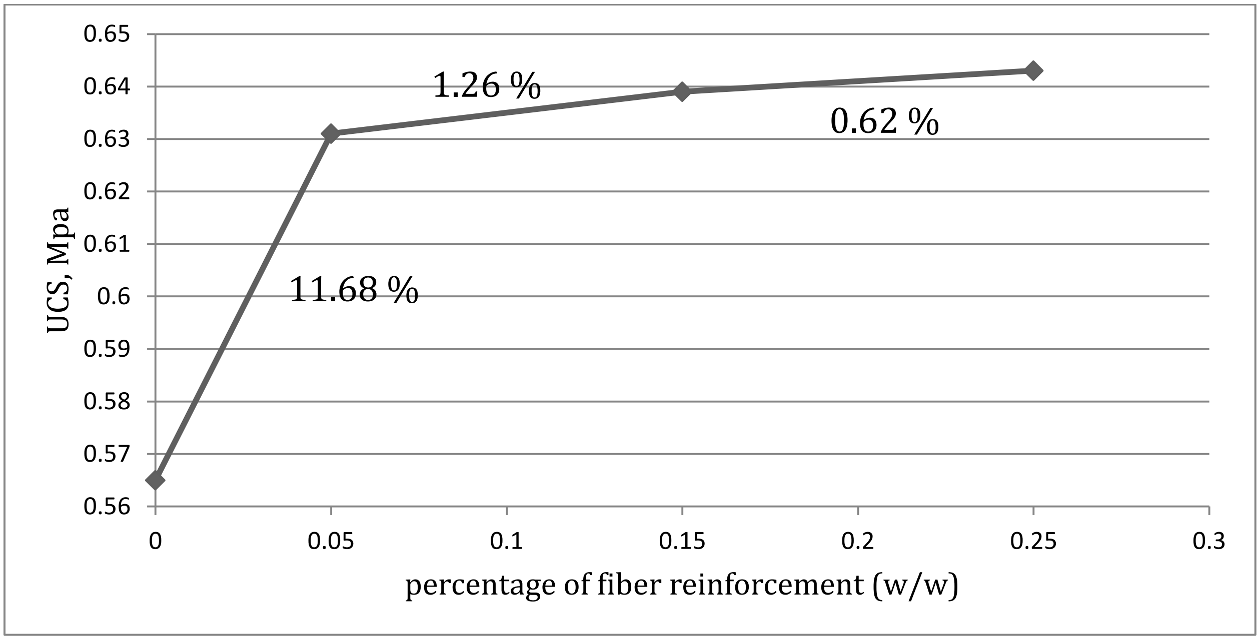

Fig. - 29

Soil sample- 2

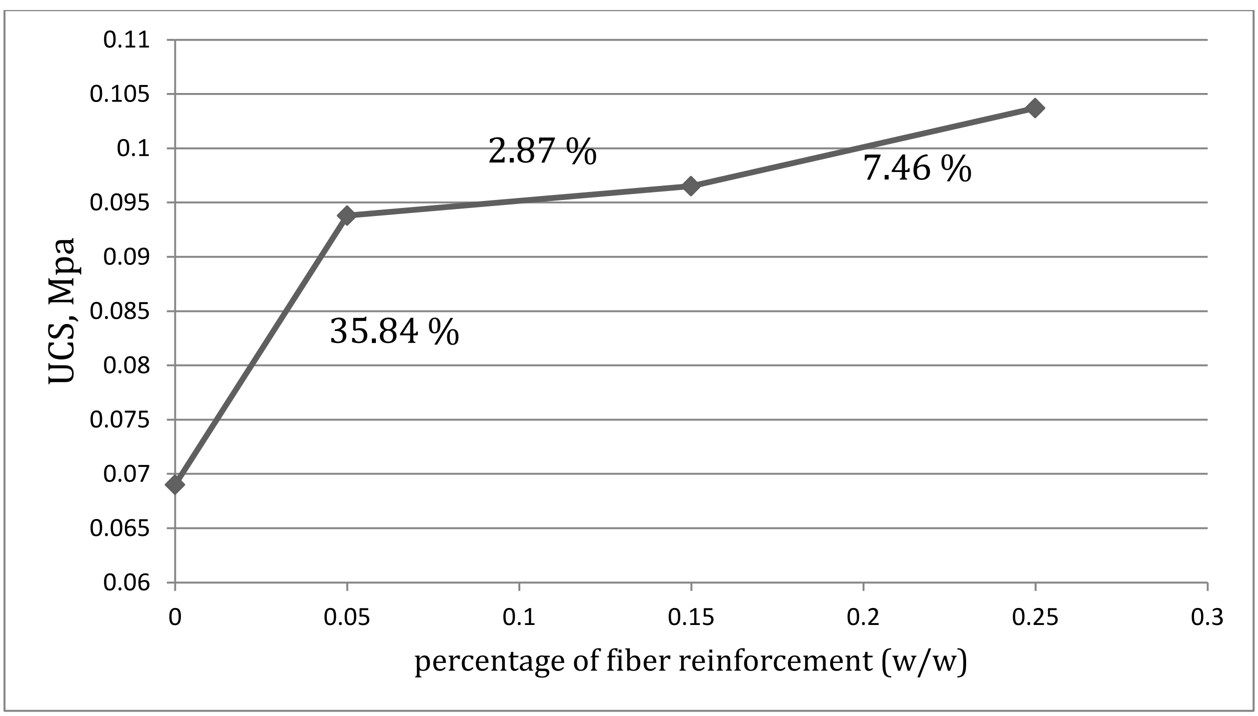

Fig. - 30

4.7.1 Inferences from Direct Shear Test

Soil sample- 1

- Cohesion value increases from 0.325 kg/cm2 to 0.3887 kg/cm2, a net **19.6%**
- The increment graph shows a gradual decline in slope.
- The angle of internal friction increases from 47.72 to 48.483 degrees, a net **1.59%**
- The increment in shear strength of soil due to reinforcement is **marginal**.

Soil sample- 2

- Cohesion value increases from 0.3513 kg/cm2 to 0.5375 kg/cm2, a net **53.0%**
- The increment graph for cohesion shows a gradual decline in slope.
- The angle of internal friction increases from 27.82 to 32 degrees, a net **15.02%**
- The increment graph for φ shows a variation in slope- alternate rise and fall.
- The increment in shear strength of soil due to reinforcement is **substantial**.

Comparison of shear parameters between soil sample- 1 and soil sample- 2

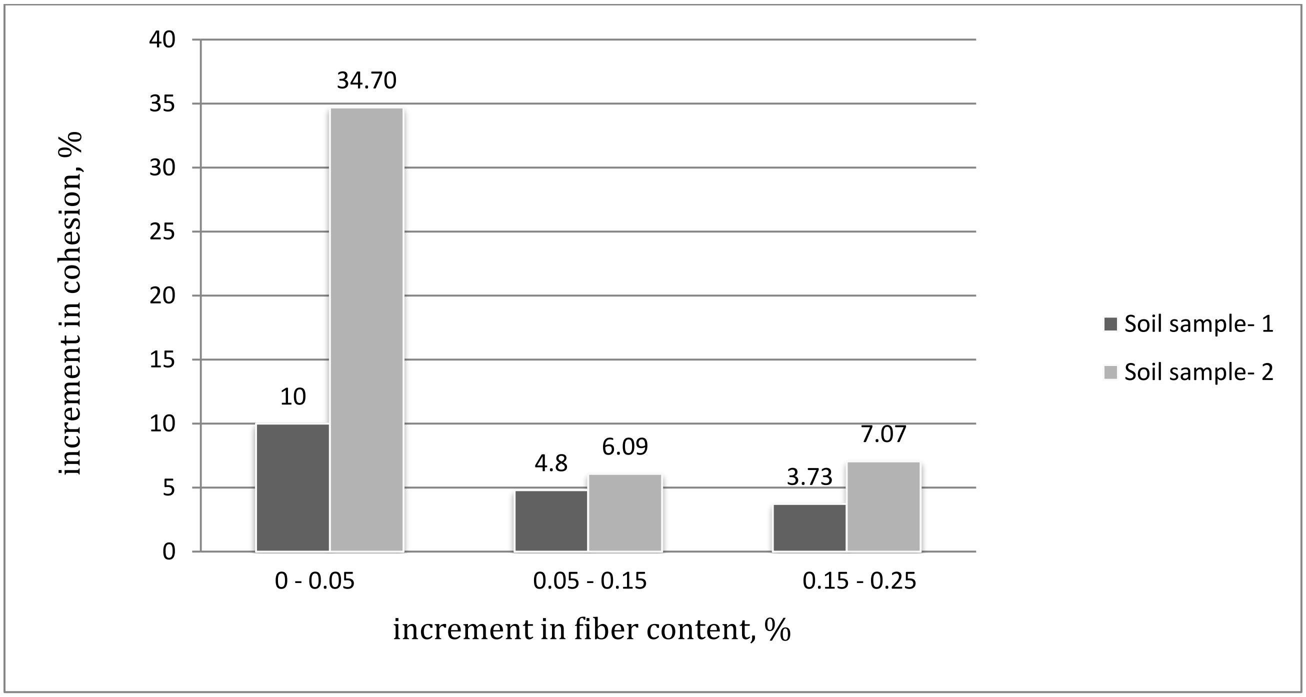

Fig. - 31

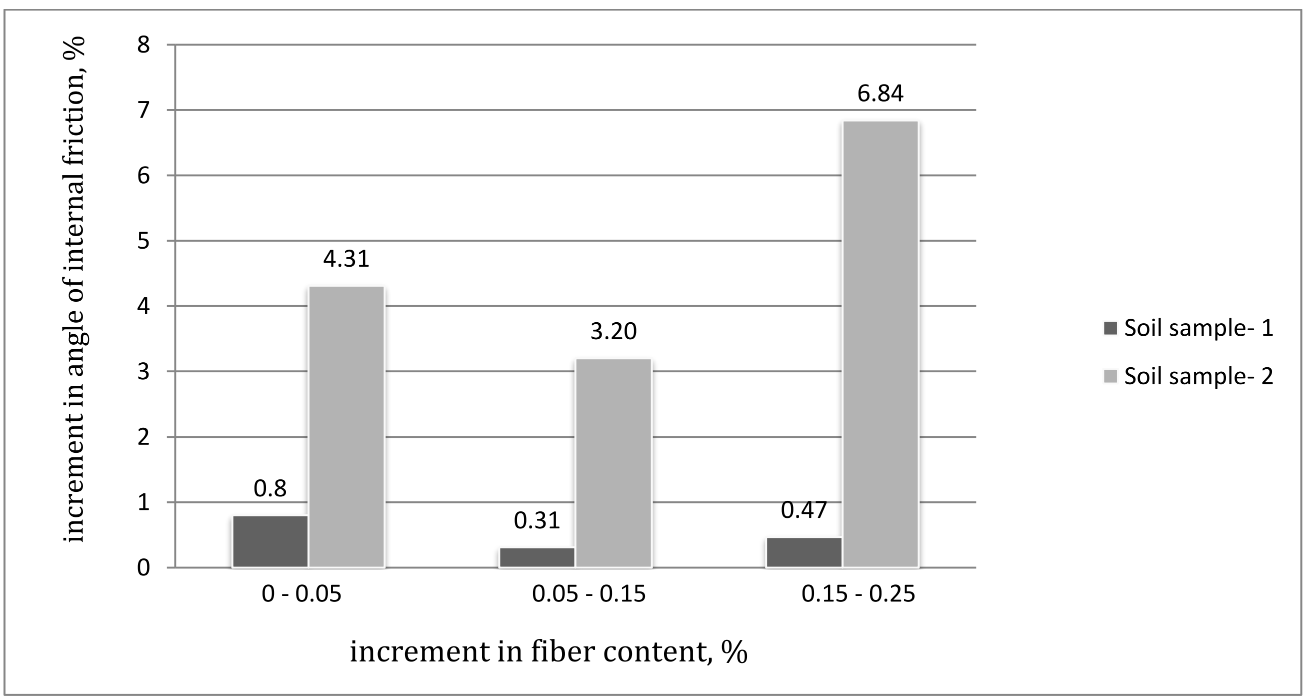

Fig. - 32

4.7.2 Inferences from Unconfined Compression Test

Soil sample- 1

- UCS value increases from 0.0643 MPa to 0.0562 MPa, a net **14.4%**
- The slope of increment graph is continuously decreasing with an initially steep slope

Soil sample- 2

- UCS value increases from 0.0692 MPa to 0.1037 MPa, a net **49.8%**
- The slope of the increment graph varies with alternate rise and fall

Comparison between soil sample-1 and soil sample- 2 for UCS

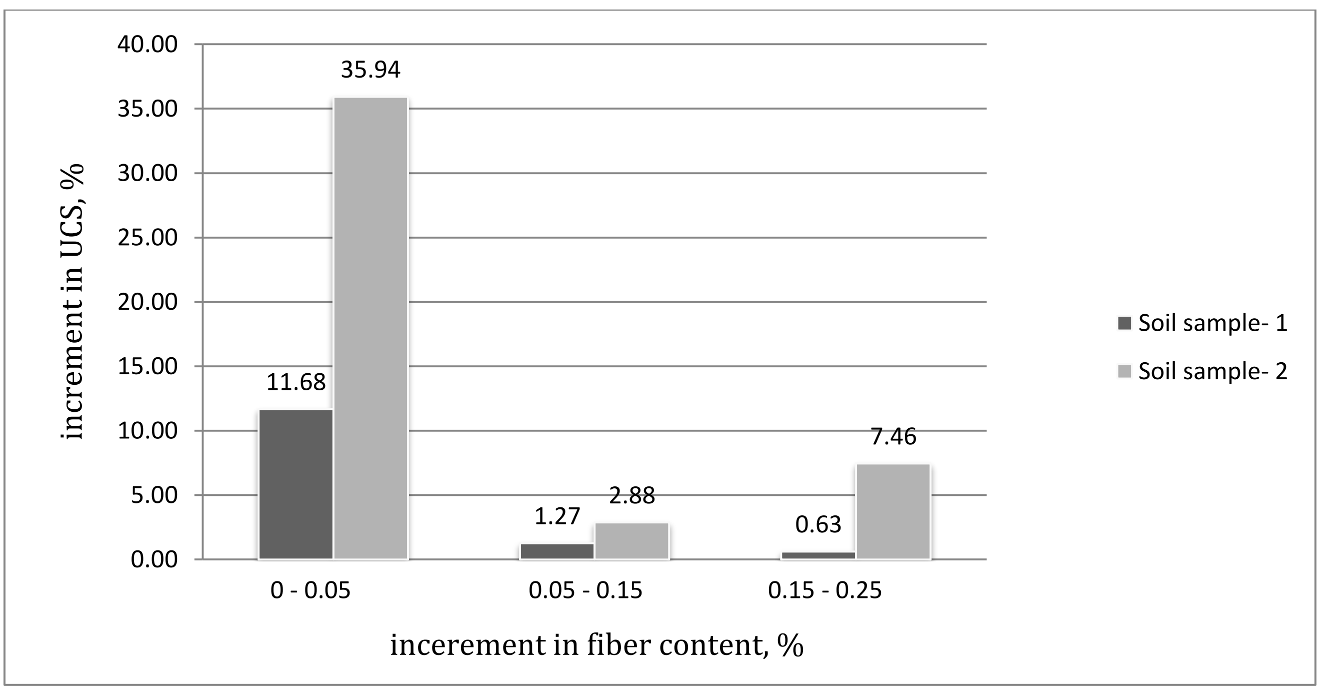

Fig. - 33

CONCLUSIONS

CONCLUSIONS

On the basis of present experimental study, the following conclusions are drawn:

1. Based on direct shear test on soil sample- 1, with fiber reinforcement of 0.05%, 0.15% and 0.25%, the increase in cohesion was found to be 10%, 4.8% and 3.73% respectively (illustrated in figure- 25). The increase in the internal angle of friction (φ) was found to be 0.8%, 0.31% and 0. 47% respectively (illustrated in figure- 27). Since the net increase in the values of c and φ were observed to be 19.6%, from 0.325 kg/cm2 to 0.3887 kg/cm2 and 1.59%, from 47.72 to 48.483 degrees respectively, for such a soil, randomly distributed polypropylene fiber reinforcement is not recommended.
2. The results from the UCS test for soil sample- 1 are also similar, for reinforcements of 0.05%, 0.15% and 0.25%, the increase in unconfined compressive strength from the initial value are 11.68%, 1.26% and 0.62% respectively (illustrated in figure-29). This increment is not substantial and applying it for soils similar to soil sample-1 is not effective.
3. The shear strength parameters of soil sample- 2 were determined by direct shear test. Figure- 26 illustrates that the increase in the value of cohesion for fiber reinforcement of 0.05%, 0.15% and 0.25% are 34.7%, 6.09% and 7.07% respectively. Figure 27 illustrates that the increase in the internal angle of friction (φ) was found to be 0.8%, 0.31% and 0. 47% respectively. Thus, a net increase in the values of c and φ were observed to be 53%, from 0.3513 kg/cm^2 to 0.5375 kg/cm^2 and 15.02%, from 27.82 to 32 degrees. Therefore, the use of polypropylene fiber as reinforcement for soils like soil sample- 2 is recommended.

4. On comparing the results from UCS test of soil sample- 2, it is found that the values of unconfined compressive strength shows a net increment of 49.8% from 0.0692 MPa to 0.1037 MPa (illustrated in figure- 30). This also supports the previous conclusion that use of polypropylene fibers for reinforcing soils like soil sample- 2 is recommended.

5. Overall it can be concluded that fiber reinforced soil can be considered to be good ground improvement technique specially in engineering projects on weak soils where it can act as a substitute to deep/raft foundations, reducing the cost as well as energy.

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