Characteristic Changes in Sandy Soil Reinforced with Natural Fibers

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***Abstract**: Reinforced earth is a composite material, which is a combination of soil and reinforcement, suitably placed to withstand the developed tensile stresses and also it improves the resistance of the soil in the direction of the greatest stress. India has been taking importance to transportation sector as they thought faster the transportation faster will be the growth in development of various sectors. So, Indian government has initiated different schemes like Golden quadrilateral, Jawaharlal Nehru National Urban Renewal Mission (JNNURM), Pradhan Mantri Gram Sadak Yojana (PMGSY) etc. Flexible pavement is more common in India and has got different layers i.e., Sub grade, Sub base, Base course and Wearing course. Sub base is the main load bearing area which minimizes the load transformation to a possible extent on the sub grade, in some cases sand in sub base in proper density and to maintain the compacted state of such sand for the service life of the road is quite difficult. Placed sand must retain the required placement density and offer same CBR value as at initial placement condition to maintain the stability of the road itself. For obtaining the required value of CBR for sub base can be achieved by addition of many alternatives such as cement, Industrial by products such as fly ash, Ground granulated blast furnace slag, low calcium fly ash, Meta kaolin, cement kiln dust, fibers [plastic waste, glass waste etc.] & cement along with fibers are used.*

Keywords — *Natural fiber materials jute, coir. Triaxial compression test, Unconfined compression test, Direct shear test, California bearing ratio test, Equivalent Confining Stress Concept, Pseudo – Cohesion Concept**,** IS 2720, Part XVI,*

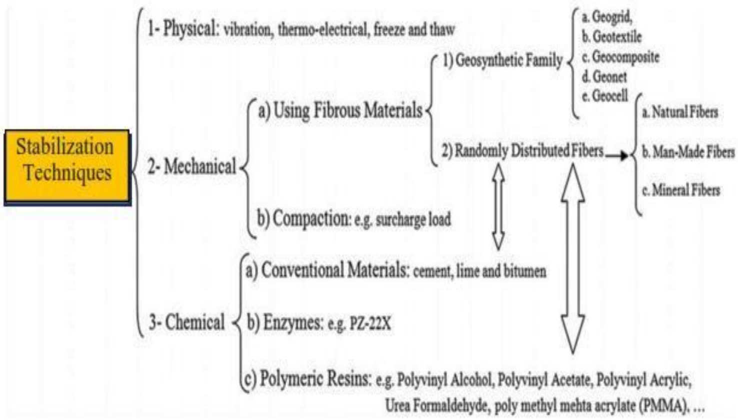
I. INTRODUCTION

Fiber Reinforced Soil (Ply Soil)

Randomly distributed fibers reinforced soil – termed as RDFS is among the latest ground improvement techniques in which fibers of desired type and quantity are added in soil, mixed randomly and laid in the position after compaction.

The term "Reinforced Soil" refers to a soil that is aided by reinforcement which resists the stresses through friction and adhesion which in turn increases the strength and stability of soil. By addition of fibers to sand, reduces the thickness of layer giving rise to reduction in cost, indeed providing better compactness and interlocking system between natural fibers and soil.

II. STABILIZATION TECHNIQUES



III. ADVANTAGES OF FIBER-REINFORCED SOIL

Randomly distributed fiber reinforced soil (RDFS) offers many advantages as listed below:

1. Increased shear strength with maintenance of strength isotropy.
2. Beneficial for all type of soils (i.e. sand, silt and clay).
3. Reduce post peak strength loss.
4. Increased ductility.
5. Increased seismic performance.
6. No catastrophic failure.
7. Great potential to use natural or waste material such as coir fibers, shredded tire and recycled waste plastic strips and fibers.
8. Provide erosion control and facilitate vegetation development.
9. Reduce shrinkage and swell pressure of expansion soil.
10. No appreciable change in permeability.
11. Unlike lime, cement and other chemical stabilization methods, the construction using fiber reinforcement is not significantly affected by weather conditions.

IV. DIRECTION OF PLACEMENT

Fibers can be oriented or randomly mixed in soil. In oriented category, the inclusions are placed within the soil at specific positions and direction where as in random category, inclusions, are mixed with soil and placed within the probable shear zone

V. FACTORS AFFECTING THE STRENGTH CHARACTERISTICS OF ENGINEERING PROPERTIES OF RDFS

The factors on which the strength characteristics and other engineering properties of RDFS depend:

1. Type of soil it includes soil gradation expressed in terms of mean grain size (D50) and uniformity coefficient (Cu).
2. Type of Fiber: Monofilament or fibrillated
3. Denier of Fiber: It is the weight (in gm) of 9000 m long fiber.
4. Fiber length
5. Aspect ratio: It is defined as the ratio of the length of fiber to its diameter (vi) Fiber soil surface friction.

VI. CONDUCTED TESTS

The fallowing tests are conducted to study the effect of fiber reinforcement on strength characteristics and other engineering properties of the RDFS (Randomly Distributed Fiber Reinforced Soil)

1. Triaxial compression test
2. Unconfined compression test
3. Direct shear test
4. California bearing ratio test

VII. FORCE TRANSFER FROM SOIL TO REINFORCEMENT

Fig.1 shows cohesion less soil mass reinforced by a flat strip. The force at the two ends of the strip is not same when there is transference of force by friction to the soil mass (Vidal, 1969). If the average cortical stress in the soil is ‘σv’ in the region, the difference between the forces at the ends of a reinforcing element AB of length „dl" is given by

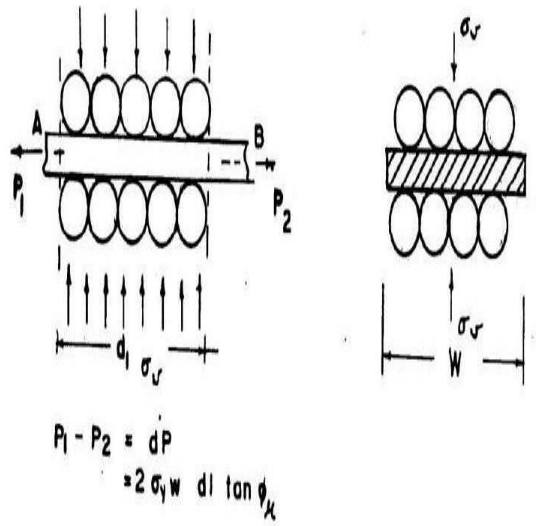
$$dP = \sigma v.2w.dl.\tan\Phi u$$

Where, “w” is the width of the reinforcement and is Φu the angle of friction between the reinforcement and the soil.

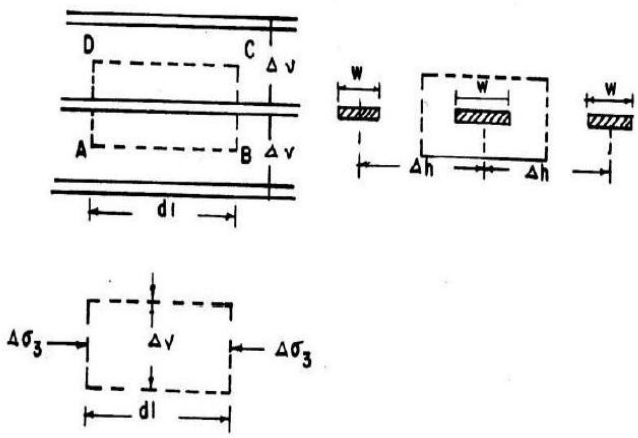
Therefore, if we consider a soil mass with spacing at spacing of “Δh” and “Δv” as shown in the Fig.2 the effect of this reinforcement on the soil mass will be to restraint by imposing an additional stress of $\Delta\sigma 3 = \Delta h\,(dp/\Delta v)$ in the horizontal direction on face AD over that prevailing on face BC.
This restraint on the soil mass increases the resistance of the soil to failure under applied stresses and the result interpreted in two related ways.

1. *Equivalent Confining Stress Concept*

Fig. 3 shows the comparison of failure stresses on two soils, one unreinforced and the other reinforced. The increase in the deviator stress is seen to be Δσ3 times Kp, where Kp is the coefficient of passive earth pressure equal to tan2 (45 + Φ/2) and Δσ3 is the equivalent confining stress on sand imposed by the reinforcement (Yang, 1972).



1) **Fig. 1 Stress Transfer by Soil Reinforcement**



2) **Fig. 2 Confining Stress on Soil by Reinforcement**

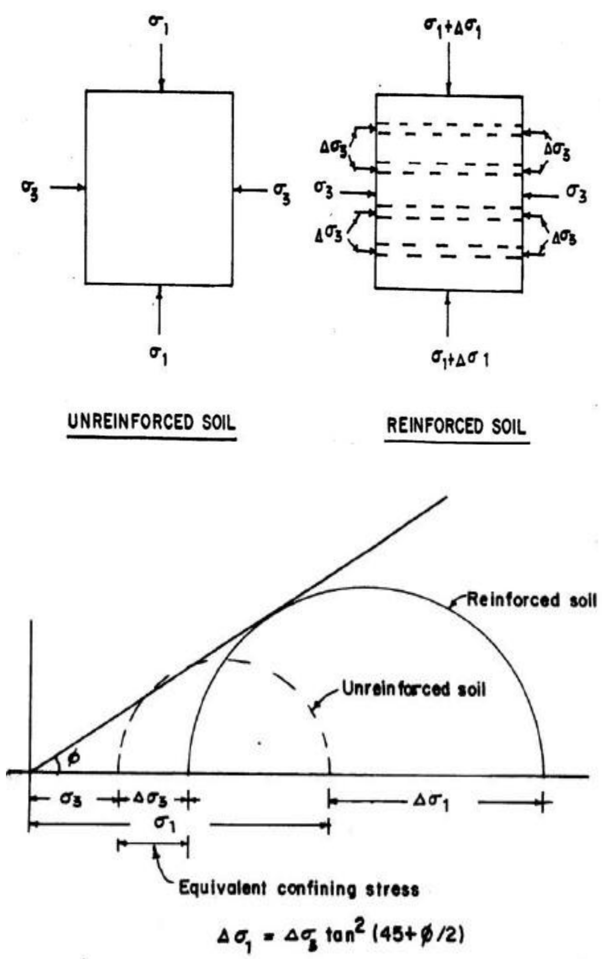


Fig. 3 Equivalent Stress Confining Concept

2. Pseudo – Cohesion Concept

This concept (Schlosser and Long, 1974) proposes that the reinforcement induces an anisotropic or pseudo-cohesion to the soil which depends on the spacing and strength of the reinforcement. Fig. shows the approach. The increase in deviator stress at failure is

$\Delta\sigma 1 = 2c \tan(45 + \Phi/2)$

Where, "c" is the pseudo-cohesion induced in the soil and Φ is the angle of friction. Both the equivalent confining stress concept and the pseudo-cohesion concept are linked to the stress induced in the reinforcement. If 'αf' is the force in the reinforcement per unit width of the soil mass and Δv is the vertical spacing.

$\alpha f/\Delta v$ is the equivalent confining pressure $\Delta\sigma 3$

And

$\Delta\sigma 1 = (\alpha f/\Delta v) \tan 2(45 + \Phi/2)$

Or

$\Delta\sigma 1 = 2c \tan(45 + \Phi/2)$ which yields

$c = (\alpha f/2\Delta v) \tan(45 + \Phi/2)$

The value of "αf" is equal to the tensile strength of the reinforcement, if the reinforcement fails by breakage or the maximum force transferred by the friction between the soil and reinforcement pulls off.

In the above concept outlined, it is necessary that the reinforcement layer must be close enough so that there is effective transfer of stress by friction or adhesion as the case may be and hence the granular soils of high relative density are particularly suitable for use in reinforced earth. The concept outlined above can also hold good for cohesive soils to a very limited extent only since the adhesion of the clay to the reinforcement is small and its effect on reinforcement is small and its effect on restraint doesn't have a multiplying effect as in granular materials. Fig. shows the increase in strength at failure of an undrained clay sample with reinforcement.

VIII. JUTE

Jute is the name of the plant or fiber, jute fibers are composed primarily of the plant materials cellulose and lignin. The fibers are off-white to brown, and 1-4 meters (3-13 feet) long

Table 1: Physical Properties of Jute

Density, g/cc	1.47
Dia, µm	10-50
Tenacity, gm/denier	3 to 5
Elongation at break (%)	1.0 to 1.8
Moisture Regain (%) at 65% R.H	12.5
Young's modulus, GPa	22

Table 2: chemical composition of jute

Constituents	%
Alpha Cellulose	60.0-63.0
Hemi Cellulose	21.0-24.0
Lignin	12.0-13.0
Fats & Waxes	0.4—1.0
Pectin	0.2-1.5
Ash	0.7-1.2

COIR

Coir is the fibrous husk of the coconut shell. There are two types of coconut fibers, brown fiber extracted from matured coconuts and white fibers extracted from immature coconuts. Brown fibers are thick, strong and have high abrasion resistance. White fibers are smoother and finer, but also weaker.

The fibers consist mainly of lignin, tannin, cellulose, pectin and other water soluble substances. However, due to its high lignin content, coir degradation takes place much more slowly than in other natural fibers.

Table 3: Physical properties of coir

Density, g/cc	1.4
Dia, µm	10-20
Tenacity, gm/denier	10
Elongation % at break	30
Moisture regain % at 65% RH	10.5%
Young's modulus, GPa	4-5

Table 4: Chemical composition of coir

Constituents	%
Cellulose	35.6
Hemi Cellulose	15.4
Pectin	5.1
Lignin	32.7
Extractives	3.0
Fats	-

2) Different Parameters considered in the experiment

Type of Sand	SP
Type of Fibers	Jute, Coir
Fiber %	0.5,1.0,1.5,2.0
Fiber length ,mm	5,10,20

IX. EXPERIMENTAL PROCEDURE

Following are the tests which have been carried out in laboratory

A. INDEX Properties

1. Specific Gravity Test by Pycnometer
2. Grain Size Distribution
3. Relative Density test

B. GEOTECHNICAL PROPERTIES

1. Compaction Test
2. California Bearing Ratio Test

Methodology

Specific Gravity Test

Specific gravity (G_s) of solid particles is the ratio of the mass of a given volume of solids to the mass of an equal volume of gas-free distilledwater at 4^0 C temperatures.

$$G_s = \frac{\gamma_s}{\gamma_t}$$

Where γ_w = unit weight of water

The specific gravity of sand was determined in laboratory using a density bottle (as per **IS: 2720 – Part III, 1980**). The bottle of 250 ml capacity was cleaned and dried at a temperature of1050 C to 1100 C and cooled. The weight of the bottle was taken. About 200 gm of oven dry Sample of sand was taken in the bottle and weighed. Distilled water was then added to cover the sample and the sand was allowed to soak water for 30 minutes. Air entrapped in the sand was expelled by gentle heating. More water was added to the bottle up to a mark and weighed. Then the bottle was emptied, washed and refilled with distilled water up to that previous mark and weighed. The specific gravity of sand was determined by the equation,

$$G_s = \frac{(M_2 - M_1)}{((M_2 - M_1) - (M_3 - M_4))}$$

Where M_1 = mass of the empty bottle
M_2 = mass of the empty bottle and dry sand
M_3 = mass of the empty bottle, sand and water
M_4 = mass of the bottle filled with water

Grain Size Distribution

Particle size analysis or sieve analysis is a method of separation of sands into different fraction based on the particle size. It expresses quantitatively the proportions, by mass of various sizes of particles present in the sand. It is shown graphically on a particle size distribution curve. Oven dry sand samples of 1000 gm were taken for sieve analysis. Sieves of size 4.75 mm, 2.36mm, 1.12mm, 600µ, 425µ, 300µ, 150µ and75µ were used for sieving (as per **IS: 2720 – Part IV, 1985**). All samples were passed through 4.75 mm sieve and very little fines (< 5 %) were retained in pan through 75µ sieve. Hence all samples were considered to be clean sands having very little fines and no gravel fractions. By taking the weights of sand fraction retained on various sieves, particle size distribution curve was plotted. The percentage finer (*N*) than a given size has been plotted as ordinate (on natural scale) and the corresponding particle size as abscissa (on log scale). The particle size distribution curve, also known as gradation curve represents the distribution of particle of different sizes in the sand mass. The particle size distribution curve also reveals whether the sand is well graded (particle of different sizes in good proportion) or poorly graded (particle almost of same sizes). From this curve, mean grain size (D_{50}), coefficient of uniformity (C_u), and coefficient of curvature (C_c) were determined.

Mean grain size (D_{50}) is the particle size corresponding to 50 % finer, which means 50 % of the sand is finer than this size.

The uniformity of sand is expressed qualitatively by the term uniformity coefficient (*C*u),

$$= \frac{D_{60}}{D_{10}}$$

Where D_{60} = particle size such that 60 % of the sand is finer than this size

D_{10} = effective size= particle size such that 10 % of the sand is finer than this size

The larger the numerical value of *C*u, the more is the range of particles. Sands with a value of *C*u less than 6 are poorly graded sand and value of *C*u 6 or more, are well graded.

The general shape of particle size distribution curve is described by another coefficient known as the coefficient of curvature (*C*c) or the coefficient of gradation (*C*g),

$$C_c = \frac{D_{30}^2}{D_{60} * D_{10}}$$

Where D_{30} = particle size corresponding to 30 % finer

D_{60} = particle size corresponding to 60 % finer

D_{10} = particle size corresponding to 10 % finer

For well graded sand, the value of Cc lies between 1 and 3 and for poorly graded sand the *C*c value is less than 1.

Relative Density Test

Most significant property of cohesion less soil (granular soil) is relative density whereas for cohesive soil is consistency. Relative density is the index property of a cohesion less soil. The engineering properties of a mass of cohesion less soil depend to a large extent on its relative density (D_r). Relative density is a term generally used to describe the degree of compaction of coarse-grained soils. As per **IS: 2720-14 (1983)** relative density test was performed. The relative density is defined as

$$D_r = \frac{e_{max} - e_{nat}}{e_{max} - e_{min}}$$

Where e_{nat} = voids ratio in the natural state e_{max} = maximum void ratio of the soil in the loosest condition e_{min} = minimum void ratio of the soil in the densest condition

If $e = e_{min}$, $D_r = 100$ and the soil is in its densest state $e = e_{max}$, $D_r = 0$ and the soil is in its loosest state D_r varies from 0 to 100 always ($0 \leq D_r \leq 100$)

Compaction Test

R.R Proctor while building dams in the USA in the early thirties, develop the principles of compaction in a series. As a tribute to proctor the standard laboratory compaction test which he devised is called the standard proctor test. The compaction characteristics and the degree of compaction can be obtained from laboratory tests. In these tests a specified amount of compactive effort is applied to a constant volume of soil mass.

In standard Proctor Test also called the light compaction test confining to **IS: 2720, Part VII-1974**, a standard volume (944cc) is filled up with the 3kg of Desert sand mixed with 0.5%, 1.0% 1.5%, 2.0% of 0.5cm, 1.0cm, 2.0cm natural fibers such as Jute and Coir in three layers. Each Layer is compacted by 25blows of standard hammer of weight 2.5kg falling from a height of 30.5cm. Knowing the wet weight of the compacted soil and its water content the dry unit weight of soil can be calculated:

$$\gamma_t = \frac{weight\ of\ the\ compacted\ soil}{Volume\ of\ the\ mould}$$

$$\gamma_d = \frac{\gamma_t}{(1+w/100)}$$

Where, γ_t= Bulk Density of the soil γ_d= Dry density of the soil
W= moisture content

The test is repeated at different water contents. The dry unit weight of each compacted sample is plotted against the water content and the curve called compaction curve. The peak point on the compaction curve corresponds to the maximum dry unit weight. The water content corresponding to the maximum dry unit weight is known as the optimum moisture content (OMC). The obtained MDD and OMC of a soil specimen are used to find out strength properties of the soil.

California Bearing Ratio

The CBR test was originally developed by O.J. Porter for the California Highway Department during the 1920s. It is a load - deformation test performed in the laboratory or the field, whose results are then used with an empirical design chart to determine the thickness of flexible pavement, base, and other layers for a given vehicle loading. Though the test originated in California, the California Department of Transportation and most other highway agencies have since abandoned the CBR method of pavement design. In the 1940s, the US Army Corps of Engineers (USACE) adopted the CBR method of design for flexible airfield pavements. The USACE and USAF design practice for surfaced and unsurfaced airfields is still based upon CBR today (US Army, 2001; US Army and USAF, 1994). The CBR determination may be performed either in the laboratory, typically with a recompacted sample, or in the field. Because of typical logistics and time constraints with the laboratory test, the field CBR is more typically used by the military for design of contingency roads and airfields. The thickness of different elements comprising a pavement is determined by CBR values. The CBR test is a small scale penetration test in which a cylindrical plunger of 3 in2 (5 cm in dia) cross-section is penetrated into a soil mass (i.e., subgrade material) at the rate of 0.05 in. per minute (1.25 mm/minute). Observations are taken between the penetration resistance (called the test load) versus the penetration of plunger. The penetration resistance of the plunger into a standard sample of crushed stone for the corresponding penetration is called standard load. The California bearing ratio, abbreviated as CBR is defined as the ratio of the test load to the standard load, expressed as percentage for a given penetration of the plunger.

$$CBR = \frac{Test\ Load}{Standard\ load} * 100$$

Different Standard loads for different plungers were given in a tabular form

Table 6: Standard Loads Adopted for Different Penetrations for the Standard Material with a CBR value of 100%.

Penetration of the plunger (inch)	Standard Load (lb)	Penetration of plunger (inch)	Standard load (kg)
0.1	3000	2.5	1370
0.2	4500	5.0	2055
0.3	5700	7.5	2630
0.4	6900	3180	3180
0.5	7800	12.5	3600

As per **IS: 2720, Part XVI (1965),** CBR test is carried out on a compacted Desert Sand Reinforced with Natural fibers of lengths 0.5cm, 1.0cm, 2.0cm in percentages of about 0.5%, 1.0%, 1.5% and 2.0% in a CBR mould 150 mm in diameter and 175 mm in height, provided with detachable collar of 50 mm and a detachable perforated base plate. A displacer disc, 50 mm deep to be kept in the mould during the specimen preparation, enables a specimen of 125 mm deep to be obtained. The moulding dry density and water content should be the same as would be maintained during field compaction. To simulate worst moisture condition of the field, the specimens are kept submerged in water for about 4 days before testing. Generally, CBR values of both soaked as well as unsoaked samples are determined. Both during soaking and penetration test, the specimen is covered with equal surcharge weights to simulate the effect of overlying pavement or the particular layer under construction. Each surcharge slotted weight, 147 mm in diameter with a central hole 53 mm in diameter and weighing 2.5 kg is considered approximately equivalent to 6.5 cm of construction. A minimum of two surcharge weights (i.e. 5kg surcharge load) is placed on the specimen. Load is applied on the penetration piston so that the penetration is approximately 1.25mm/min. The load readings are recorded at penetrations, 0, 0.5, 1, 1.5, 2, 2.5, 3, 3.5, 4, 4.5, 5, 5.5, 6, 6.5, 7, 8, 9, 10, 11, 12, and 12.5mm. The maximum load and penetration is recorded if it occurs for a penetration of less than 12.5 mm.

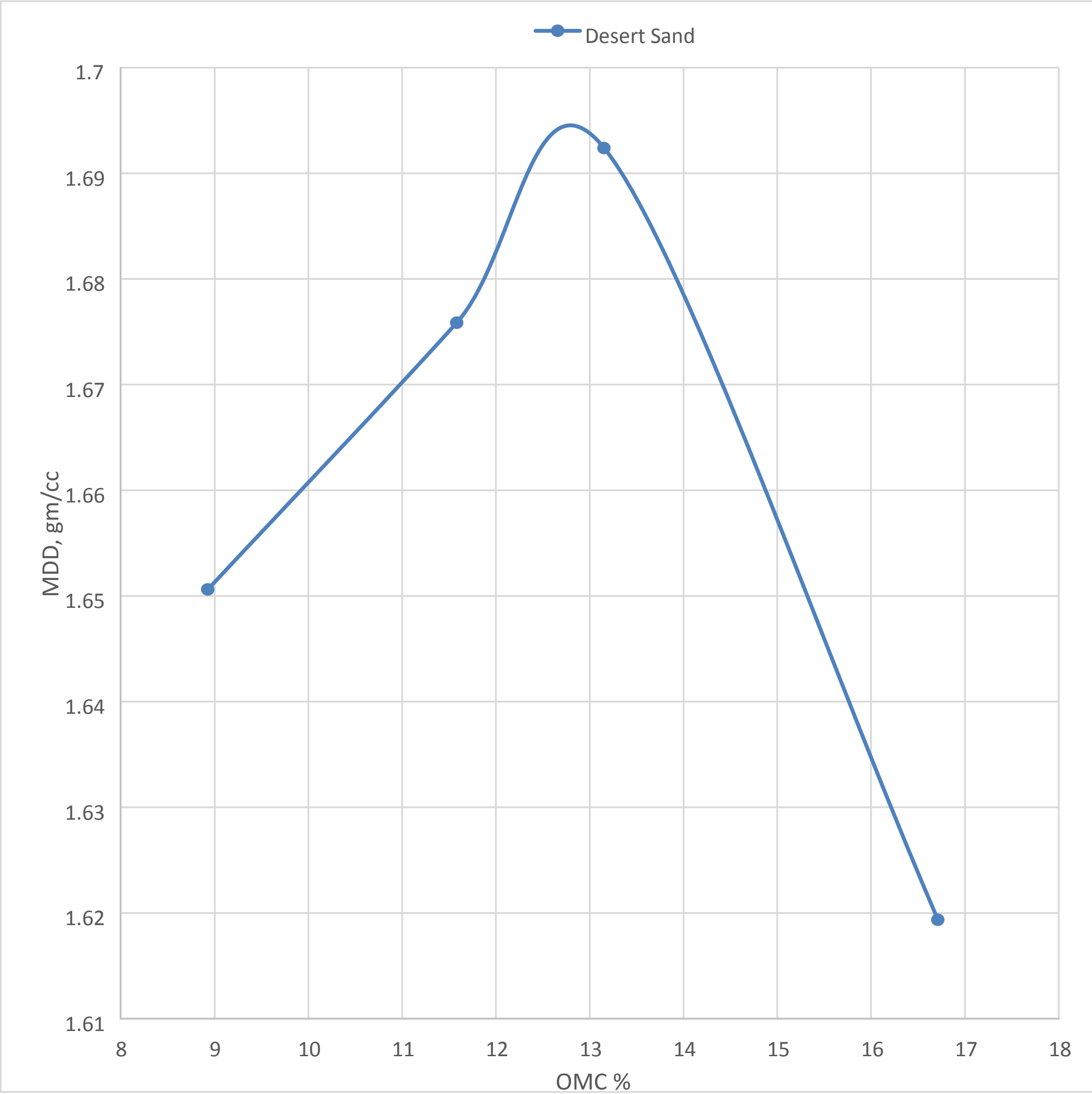
CBR Apparatus

The curve is mainly convex upwards although the initial portion of the curve may be concave upwards due to surface irregularities. A correction is then applied by drawing a tangent to the curve at the point of greatest slope. The corrected origin will be the point where the tangent meets the abscissa. The CBR values are usually calculated for penetrations of 2.5 mm and 5mm. Generally the CBR values at 2.5mm penetration will be greater than 5mm penetration and in such a case the former is taken as the CBR value for design purposes. If the CBR value corresponding to a penetration of 5mm exceeds that for 2.5mm, the test is repeated. If identical results follow, the bearing ratio corresponding to 5mm penetration is taken for design.

Properties of Desert Sand

INDEX Properties	
Specific Gravity	2.61
Gravel %	0
Sand %	94.19
Percent finer than 75μ sieve	4.26
D10,D30,D60	0.08, 0.09, 0.15
Coefficient of uniformity, Cu	1.875
Coefficient of conformity, Cc	0.675
IS Classification	SP
γdmax , gm/cc	1.698
γdmin , gm/cc	1.463
Engineering Properties	
MDD ,gm/cc	1.695
OMC , %	12.8
Unsoaked CBR, %	16.5
Soaked CBR, %	14.3

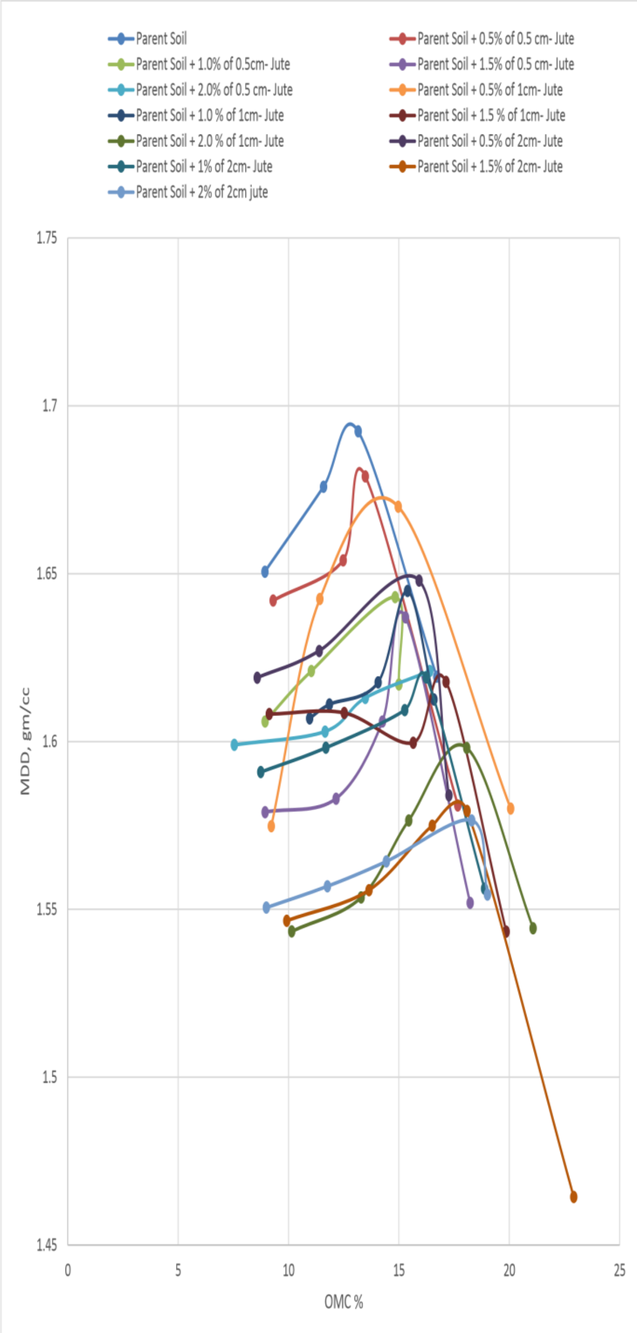
Moisture-Density Relationship Parent Soil



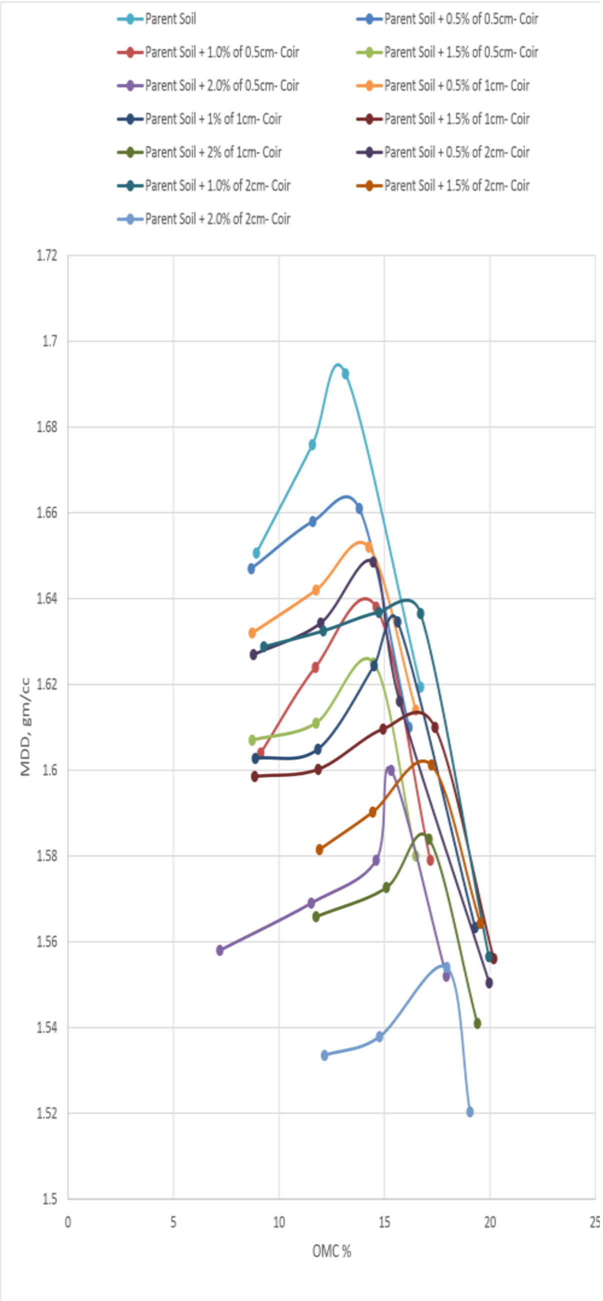
MDD vs. OMC for Desert Sand

Standard Proctor Results of Reinforced Desert Sand

Fiber Length	Fiber %	Desert Sand	
		MDD	**OMC**
Jute 0.5cm	0.5	1.68	13.9
	1.0	1.645	14.8
	1.5	1.639	15.2
	2.0	1.623	16
Jute 1.0cm	0.5	1.672	14.5
	1.0	1.63	15.6
	1.5	1.61	16.5
	2.0	1.6	17.7
Jute 2.0cm	0.5	1.65	15.3
	1.0	1.62	16.2
	1.5	1.582	17.4
	2.0	1.576	18.5
Coir 0.5cm	0.5	1.667	13.3
	1.0	1.64	13.9
	1.5	1.627	14.5
	2.0	1.6	15.2
Coir 1.0cm	0.5	1.654	13.8
	1.0	1.636	15.5
	1.5	1.612	16.4
	2.0	1.585	17.2
Coir 2.0cm	0.5	1.649	14.5
	1.0	1.62	15.9
	1.5	1.602	16.7
	2.0	1.55	17.5



MDD vs. OMC Combined Curves for Desert Sand Reinforced with Jute Fiber



MDD vs. OMC Combined Curves of Desert Sand Reinforced with Coir Fiber

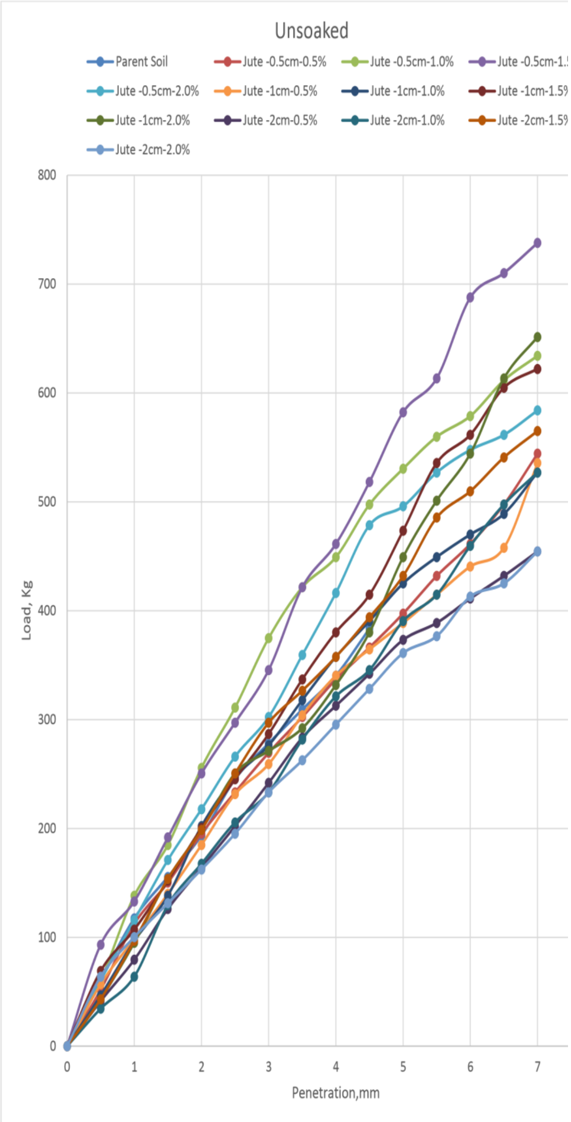
California Bearing Ratio:

CBR-value is used as an index of soil strength and bearing capacity. This value is broadly used and applied in design of the base and the sub-base material for pavement. Sand is often used for the construction of these pavement layers and also for embankments

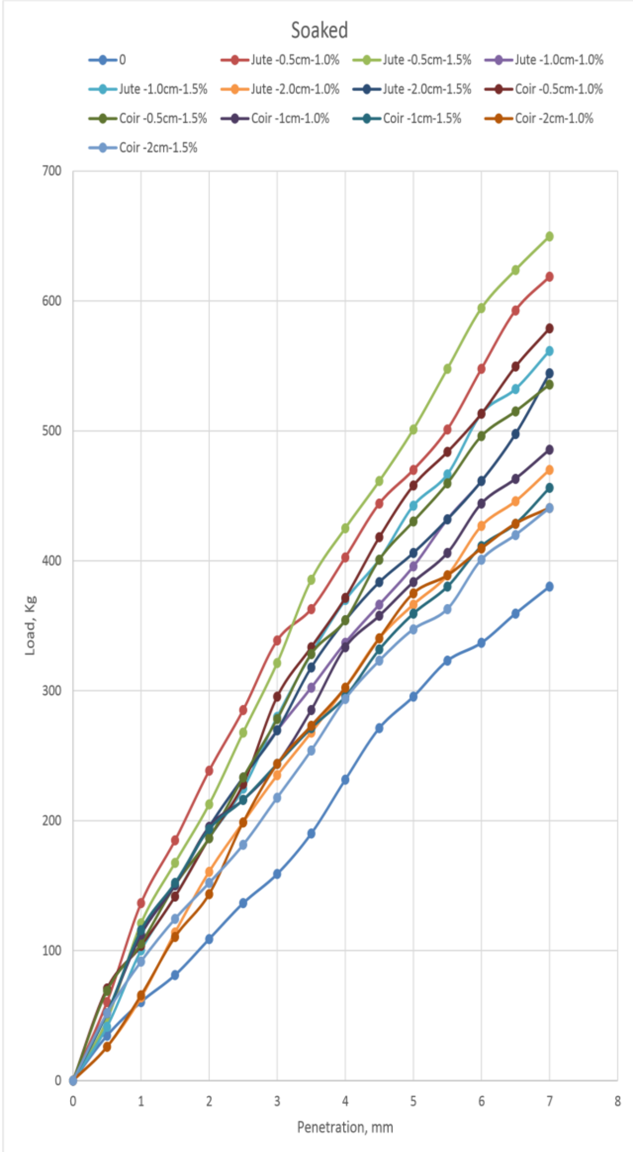
CBR Results of Reinforced Desert Sand

Fiber Length	Fiber %	CBR	
		Unsoaked	Soaked
Jute 0.5cm	0.5	19.87	17.3
	1.0	25.8	22.8
	1.5	28.5	24.4
	2.0	24.1	21.2
Jute 1.0cm	0.5	18.9	16.7
	1.0	21.8	19.23
	1.5	23	21.5
	2.0	20.7	18.78
Jute 2.0cm	0.5	18.5	15.9
	1.0	19	17.81
	1.5	21	19.7
	2.0	17.6	16.9
Coir 0.5cm	0.5	19	17.3
	1.0	23.8	22.3
	1.5	22.5	20.9
	2.0	21.23	18.2
Coir 1.0cm	0.5	18.4	16.2
	1.0	21.23	18.7
	1.5	19.8	17.5
	2.0	18.9	16.9
Coir 2.0cm	0.5	17.8	15.1
	1.0	20.8	18.21
	1.5	18.75	16.95
	2.0	16.7	16.4

Reinforced Soil



Load vs. Penetration Curve for Reinforced Desert Sand-Unsoaked Condition



Load vs. Penetration Curve of Reinforced Desert Sand-Soaked Condition

X. CONCLUSION

The study investigates about the influence of the fibers up on strength characteristics of Desert sand. The following conclusions have been drawn based on the laboratory investigations carried out.

(A) Compaction Characteristics

1. **Maximum dry density -** Regarding compaction characteristics of randomly mixed coarse grained soils used in the test, it is seen that MDD value decreases abruptly from 1.695gm/c.c for virgin soil to 1.68gm/c.c for jute fibers of 0.5cm when mixed in 1.5%.Thereafter, the decrease in MDD value is not significant. The value of MDD vary much when length of the fiber is altered. Similar characteristics are observed for coir fibers.
2. **Optimum Moisture Content -** Regarding change in OMC value in randomly mixed soil with natural fiber, it is observed that OMC values increase for both natural fibers when they are added in increasing percentage. However, the increase in OMC value is more in case of jute fibers compared to coir fibers. For jute fibers at any percentage of fiber mixed, OMC value is higher with length of the fiber. This is same for coir fibers as well.

(B) Strength Characteristics

1. **Unsoaked California Bearing Ratio -** The CBR value of randomly mixed soil used in experimental investigation seems to reach maximum value of 28.5, when jute fiber of length 0.5cm is mixed in 1.5%. Similar maximum improvement in CBR value is also observed for coir fiber used is of same length and mixed in same percentage. However, in case of coir fiber, the maximum CBR achieved is slightly lesser at 23.8 compared to 16.5 at virgin soil.
2. **Soaked California Bearing Ratio:**

   The CBR value of randomly mixed soil used in experimental investigation seems to reach maximum value of 28.5, when jute fiber of length 0.5cm is mixed in 1.5%. Similar maximum improvement in CBR value is also observed for coir fiber used is of same length and mixed in same percentage. However, in case of coir fiber, the maximum CBR achieved is slightly lesser at 23.8 compared to 16.5 at virgin soil.

XI. REFERENCES

[1] IS: 1498 (1970), "Indian Standard Methods of Test for Soils: Classification and Identification of Soil for General Engineering Purposes", Bureau of Indian Standards.

[2] IS: 2720 (Part 7) (1980), "Methods of Test for Soils, Determination of water content dry density relationship using light compaction test", Bureau of Indian Standards.

[3] IS: 2720 (Part3) (Sec.-II) (1980), "Indian Standard Methods of Test for Soils: Determination of Specific Gravity-Section II: Fine, Medium and Coarse Grained Soils", Bureau of Indian Standards.

[4] IS: 2720 (Part 5) (1983), "Indian Standard Methods of Test for Soils: Determination of Density Index (relative density) of Cohesionless soil", Bureau of Indian Standards.

[5] IS: 2720 (Part 4) (1985), "Indian Standard Methods of Test for Soils: Grain Size Analysis", Bureau of Indian Standards.

[6] Gray, D.H., and Al-Refeai, T. (1986), "Behavior of Fabric versus Fiber-Reinforced Sand", J. Geotech. Eng., 112(8), pp. 804-820.

[7] Gray, D.H., and Ohashi, H. "Mechanics of fibre reinforcing in sand", Journal of Geotechnical Engineering, ASCE, Vol.109 (3), pp.335-353.

[8] IS: 2720(Part 16) (1987), Methods of Test for Soils, "Laboratory Determination of CBR", Bureau of Indian Standards.

[9] Maher, M.H., and Gray, D.H. (1990) "Static Response of Sands Reinforced with Randomly Distributed Fibres", Journal of Geotechnical Engineering, ASCE, Vol.116 (7), pp.1661-1677.

[10] Lindh, E., and Eriksson, L. (1990), "Sand Reinforced with Plastic Fibres, a Field Experiment", Performance of Reinforced Soil Structures, McGown, A., Yeo, K., and Andrawes, K.Z., Editors, Thomas Telford, Proceedings of the International Reinforced Soil Conference held in Glasgow, Scotland, September, pp.471-473.

[11] AI-Refeai, T.O. (1991), "Behaviour of Granular Soils Reinforced with Discrete Randomly Oriented Inclusions", J. Geotextiles and Geomembranes, Vol.10 (4), pp.319333.

[12] Bauer, G.E. and Fatani, M.N. (1991), "Strength Characteristics of Sand Reinforced with Rigid and Flexible Element", Proc. Of 9th Asian Regional Conference on Soil Mech. And Foundation Engg., Bangkok (Thailand), Vol. 1, pp.471-474.

[13] Fletcher, C. S., and Humphries, W. K. (1991), "California Bearing Ratio Improvement of Remolded Soils by the Addition of Polypropylene Fiber Reinforcement", Transp. Res.Rec.1295, Transportation Research Board, Washington, D.C., pp.80–86.

[14] Gosavi, M., Patil, K.A., Mittal, S., and Saran, S. (1991), (1983), "Improvement of Properties of Black Cotton Soil Subgrade through Synthetic Reinforcement", Journal of institution of Engineers (India), vol. 84, pp.257-262.

[15] Guha, A. (1995), "Characteristic Physical Properties of Coir Fibres and their Compatability with Jute", M.Tech Thesis, IIT, Delhi.

[16] Michalowski, R.L., and Zhao, A. (1996), "Failure of Fibre-Reinforced Granular Soils", Journal of Geotechnical Geoenvironmental Engg., ASCE, Vol.122 (3), pp.226-234.

[17] Ranjan, G., Vasan, R.M., and Charan, H.D. (1996), "Probabilistic Analysis of Randomly Distributed Fibre-Reinforced Soil", Journal of Geotechnical Engineering, ASCE, Vol.122(6), pp. 419-426.

[18] Webster, S. L., and Santoni, R. L. (1997), "Contingency Airfield and Road Construction using Geosynthetic Fiber Stabilization of Sands", Tech. Rep.GL-97-4, U.S. Army Engr. Waterways Experiment Station, Vicksburg, Miss.

[19] Santoni, R.L., and Webster, S.L. (2001), "Engineering Properties of Sand-Fiber Mixture for Road Construction", J. Geotech. Geoenviron. Eng. (ASCE) 127(3), pp. 258–268.

[20] Ling, I., Leshchinsky, D., and Tatsuoka, F. (2003), "Reinforced Soil Engineering: Advances in Research and Practice", Marcel Dekker Inc.

[21] Gupta, P.K., (2004), "Behavior of Fiber Reinforced Sand", Indian Geotechnical Journal.

[22] Mitchell, James K., Kenichi., and Soga. (2005), "Fundamentals of Soil Behaviour ", 3rd Edition.

[23] Murray, J., Frost D., and Wang, Y. (2006), "The behavior of sandy soil reinforced with discontinuous fiber inclusions", Trans Res Rec 714, pp.9–17.

[24] Kumar, P., and Singh, S.P (2008), "Fiber-Reinforced Flyash subbase in Rural Roads", J. Transp. Eng. (ASCE) 134(4), pp.171–180.

[25] Ramesh, A. and Kumar, M. (2009), "An Experimental Investigation on Coir Fiber and Fly Ash in Stabilized Mechanistic Pavements", Proc. Indian Geotech. Conf. 1, pp. 201– 204.

[26] Rao, S., and Jayalekshmi, S. (2010), "Fibre Reinforcement of Soil Sub-grade Beneath Flexible Pavements", Proc. Indian Geotech. Conf. 1, pp.575–578 (2010).

[27] Dos Santos, A.P.S., Consoli, N.C., and Baudet, B.A. (2010), "The Mechanics of FiberReinforced Sand", Geotechnique 60(10), pp.791–799.

[28] Saran Swami (2010) "Reinforced Soil and its Engineering Applications", Second Edition, I. K. International Publishing House Pvt. Ltd.

[29] Maity, J., Chattopadhyay, B.C., and, Mukherjee, S.P. (2010), "Application of Geonaturals Spoils in Sub base for Road Construction", Proc. Indian Geotech. Conf. 1, pp.617–620.

[30] M. Pal, K., Majumdar, M., and Barman, D. Sarkar. (2010), "Study of Strength, CBR, Resistivity and Conductivity of Soil–Jute Mixture", Indian Highway, IRC 6, pp.53–62.

[31] Aggarwal, P., and Sharma, B. (2010), "Application of Jute Fibre in the Improvement of Subgrade Characteristics ".

[32] J. Maity, B.C. Chattopadhyay, B.C., ang Mukherjee, S.P. (2012), "Behaviour of different types of sand randomly mixing with various Natural Fibers", Proc J.Inst.Eng.India Ser.A Vol. 93(2), pp.97-104.

[33] Chegenizadeh, Amin., and Nikraz, Hamid. (2012), "CBR Test on Fiber Reinforced Silty sand", International Journal of Structural and Civil Engineering, ISSN : 2277-7032, Volume 1 Issue 3.

[34] Singh, H.P. (2013), "Effects of Cotton Fiber on CBR Value of Itanagar Soil", International Journal of current and Engineering and Technology, ISSN 2277 – 4106.

[35] Singh, R.R., and Er. Shellly Mittal. (2014), "Improvement of Local Subgrade Soil for Road Construction by the use of Coconut Coir Fiber", IJRET: International Journal of Research in Engineering and Technology eISSN: 2319-1163 | pISSN: 2321-7308.