Effect of Geogrid Reinforcement on Hyperbolic Stress Strain Behavior of Sand: An Experimental Investigation

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Abstract: ***This paper studies the effect of geogrid reinforcement on the hyperbolic stress-strain behavior of sand. A comprehensive set of laboratory triaxial compression tests was carried out on dry sand with and without geogrid. The layer configurations used are two, four and six horizontal reinforcing layers in a triaxial test sample. The influences of the number of geogrid layers and confining pressure on sample were studied and described. The results show that the hyperbolic equation (Kondner, 1963) can be used to represent the stress–strain relationship of both unreinforced and reinforced sand. It was also observed that the inclusion of geogrid increases the peak strength, axial strain at failure and hyperbolic parameters. Bulging between layers of reinforcement was observed.***

Keywords: Triaxial test, Geogrid, Sand, Soil reinforcement, Peak strength

1. Introduction

Reinforced soil is a composite material in which elements of high tensile resistance are implemented to increase the tensile resistance of the soil. Geosynthetics are the main materials used for increasing the resistance and stability of geotechnical structures all around the world. Among geosynthetics, geotextiles have received more attention because of their wide range of usage (Holtz, 2001).

One of the most important applications of geosynthitics is in the construction of reinforced slopes to increase the shearing resistance and allow for steeper slopes to be designed and constructed. The methods used to design reinforced slopes are based mainly on the limit equilibrium concept. Methods such as Jewell (1980; 1991), Reugger (1986), Schmertmann et al. (1987), Leshchinsky and Boedcker (1989), and Michalowski (1997) all use limit equilibrium analysis or limit analysis in the design of reinforced slopes. These studies used different methods in their analyses: the method of slices, two-part wedge and internal stability, variational limit equilibrium, and kinematics limit analysis, respectively.

Since the beginning of 1970s, several investigators have studied stress-strain and strength characteristics of reinforced soil using triaxial, direct shear, and plane strain tests. Extensive work has been performed on geotextile-reinforced sand. Some of these investigations are reviewed here to provide a reference to existing experimental data on the behavior of reinforced soils. Broms (1977) researched the mechanical behavior of geotextile-reinforced sand with monotonous grain size using a number of triaxial tests. Borms (1977) also studied the effect of distance between geotextile layers, sand density, and confining pressure on the strength of reinforced sand samples.

Holtz et al. (1982) conducted a number of long-term and short-term triaxial tests on dry sand reinforced by woven and nonwoven geotextiles. They also observed the influence of reinforcement on the creep of reinforced samples. Nakai (1992) investigated the stress-strain behavior of reinforced sand using triaxial tests and finite element analysis. Triaxial tests were performed on Toyoura sand, and reinforcement layers in the form of brass sheets were employed. Some finite element analyses were also performed under the experimental conditions with only a quarter of the triaxial samples being modeled. Haeri et al. (2000) studied the mechanical behavior of nonwoven geotextile-reinforced sand using triaxial apparatus. They conducted 160 triaxial tests on Unreinforced and reinforced Babolsar dry sand. They investigated the effect of some determining factors including geotextile layers, type and orientation of geotextiles and confining pressure. Two samples, with 38 and 100 mm diameters respectively, were tested to determine the influence of sample size on the mechanical behavior of unreinforced and reinforced sands.

All the above investigations studied the effect of geotextiles. Thus in current study the effect of geogrid on mechanical behavior of geogrid reinforced sand was investigated. Particularly effect of reinforcement on hyperbolic parameters of sand has been investigated.

2. Experimental Program

To investigate the effect of test parameters on the mechanical behavior of unreinforced sand, triaxial compression tests were performed. The test parameters included number of geogrid layers and confining pressure. The sample size in all the tests performed was kept 100mm diameter and 200mm height. A summary of these test parameters is given as under:

- Geogrid arrangement as shown in Figure1
- Four confining pressures (150, 250, 350 and 500) kN/m2.
- Sample size 100mm.

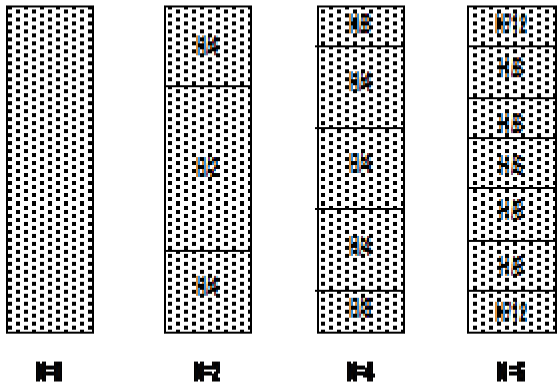


Figure 1: Sand samples with reinforcement layer arrangements tested in triaxial tests

2.1 Test Materials
2.1.1 Sand

The soil used in this investigation was dry sand collected locally from Ranipur village. The particle size distribution curve of the sand is shown in Fig. 2. All the tests were carried out in a medium dense state i.e. at a relative density of 55%. The sand used in the investigation was classified as poorly graded sand (SP). The other engineering properties of sand as determined in the laboratory are given in Table 1.

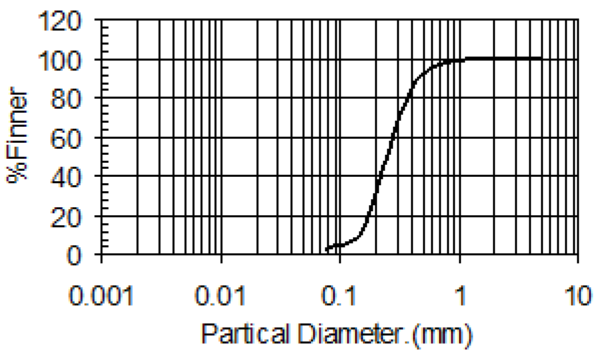


Figure 2: Particle size distribution of Ranipur sand

Table 1: Physical properties of Ranipur sand

S. No.	*Property*	*Value*
1	Soil type	SP
2	Effective Size (D10)	0.175
3	Uniformity Coefficient (Cu)	1.6
4	Coefficient of curvature (Cc)	0.83
5	Mean Specific Gravity, G	2.65
6	Maximum Dry Density γd max	17.5
7	(kN/m3)	15.3
8	Minimum Dry Density γd min (kN/m3)	55%
9	Relative density, Dr	15.8
	Unit weight of sand (kN/m3)	

2.1.2 Geogrid

The material used to reinforce the sand for performing tests was geogrid SG 150 as shown in Fig.3, supplied by M/s Strata Geosystems (India) Pvt.Ltd. It is a high performance geogrid constructed of high molecular weight and high tenacity polyester yarns. Yarns are precision knitted into a dimensionally stable, uniform network of apertures providing significant tensile reinforcement capacity. The physical and design properties of SG150 are given in Table 2, as supplied by manufacturer.

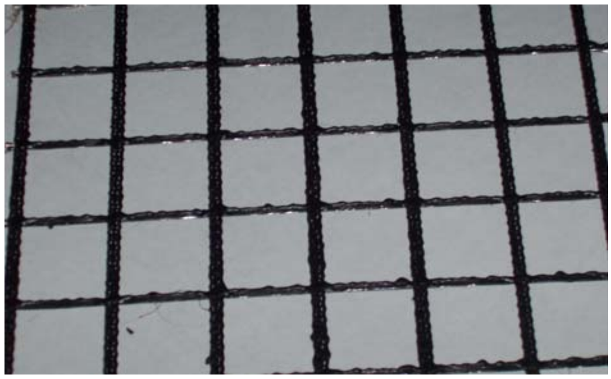


Figure 3: Strata geogrid SG – 150

Table 2: Physical and engineering properties of SG - 150

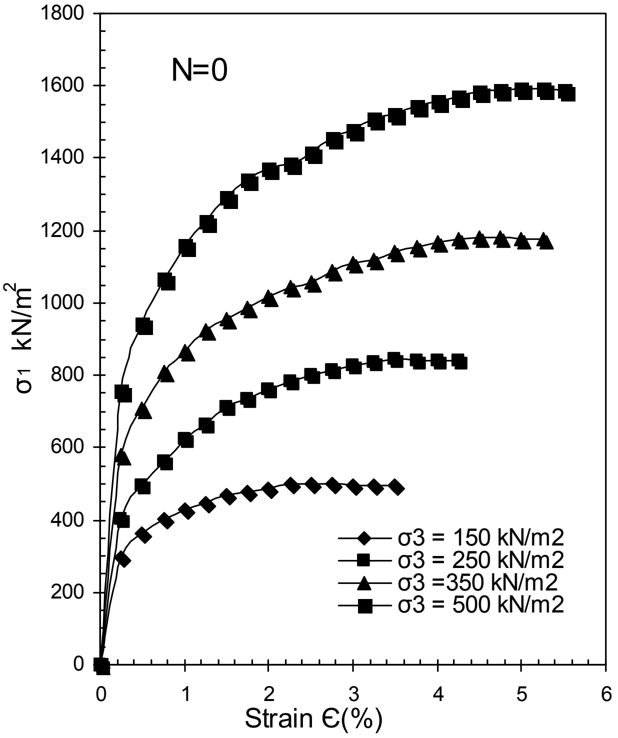
S.No.	*Property*	*Value*
1	Structure	Biaxial
2	Aperture shape	Square 22.9
3	Aperture size (mm x mm)	x 22.9
4	Roll Dimensions (Width(m) x Length(m))	1.8 x 55.7
5	Weight per Roll (kilograms)	15.9
6	Polyester type	SG - 150
7	Ultimate Tensile strength at 10% strain	27.5
8	(kN/m)	17.0
9	Creep Limited Strength (kN/m) MD	14.7
	Long – term Design Tensile Strength for	
	Sand, Silt & Clay (kN/m) MD	

2.2 Test Equipment and Procedure

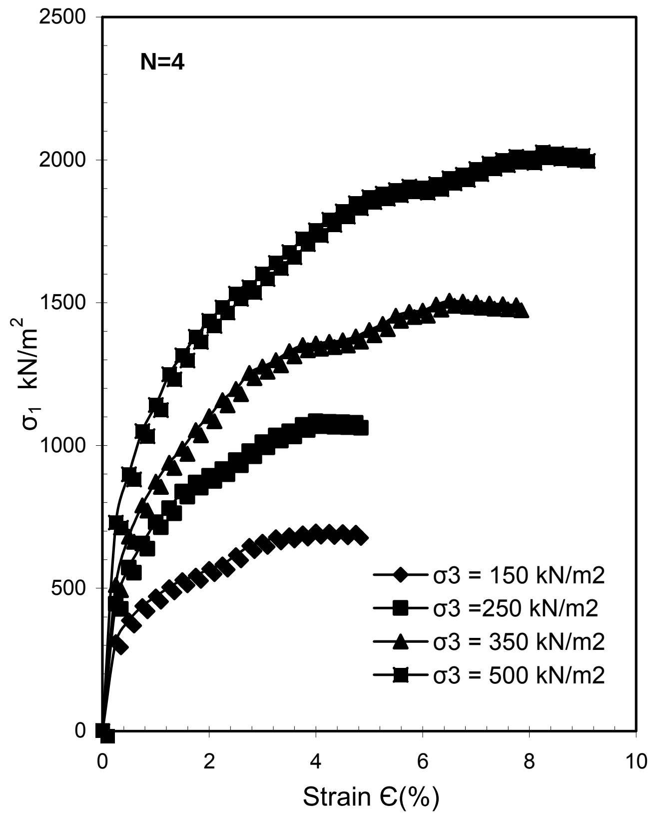
A standard triaxial apparatus was used for testing unreinforced and reinforced dry samples, which were cylinders with 100mm diameter and 200mm height. As several researchers observed (e.g. Lambe and Whitman,1979), the stress-strain behaviors of dry sand and saturated granular soil are analogous provided that the pore fluid can freely flow into and out of pores and no excess pore pressure can develop. A standard procedure for preparing dry cohesionless sample and testing with triaxial apparatus was adopted as recommended by Bishop and Henkel (1969), Ladd (1978) and Head (1986). The samples were compacted in several layers through tamping with a tamper consisting of a circular disk attached to a steel rod. The disk had a diameter slightly less than the mold. The relative density of the sand was maintained constant around 55% for all tests. After compacting and leveling each layer of sand, the reinforcement was placed horizontally in the specimen. The diameter of the reinforcement was slightly less than that of the sample. The specimen was compacted in ten layers. For all tests, a strain rate of 0.35% per minute was used. Most the tests were continued up to a strain level of 20%. Corrections such as membrane penetration, membrane force, cell compensation were not considered.

3. Test Results and Discussion

The typical stress-strain curves for unreinforced and reinforced sample under confining pressure of 150, 250, 350 and 500 kPa with different number of geogrid layers have been shown in Figs. 4 a-d. These figures indicate that the reinforcement increases the deviatoric stress and shear strength of the samples considerably, compared with unreinforced samples. This matter is essentially due to the increase in confinement; geogrid layers cause an internal confinement in reinforced samples, which has been explained by an increased confinement concept by Yang [10]. It can be observed that, there were no pronounced failure points in stress-strain behavior; as increasing the number of reinforcement layers resulted in more ductility of the samples as clogging developed in shear band within specimens. The figures also show that the beneficial effect of geogrid to enhance the strength of reinforced samples appear in high strain. It means that, the high strain levels should be imposed to appear the effect of geogrid layers to increase the strength of samples.

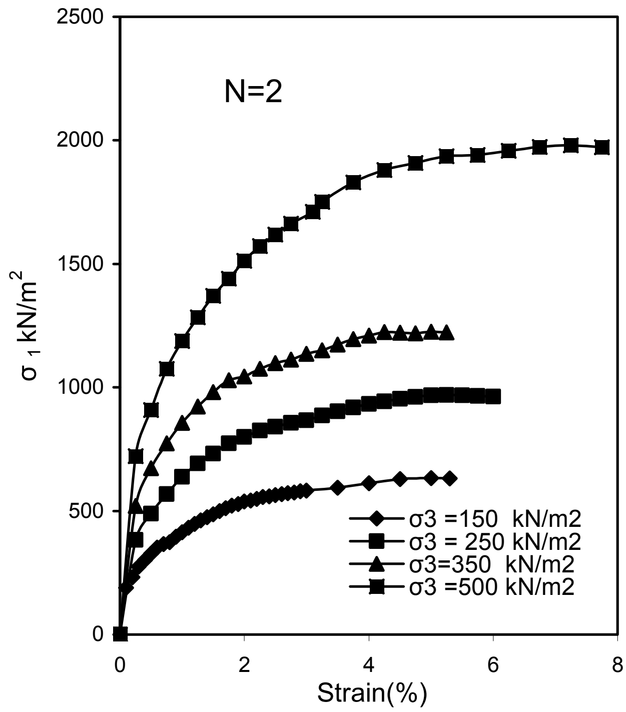


(a)



(c)

Figure 4: Stress–strain plots of unreinforced and reinforced sand samples (N-> Indicates the number geogrid layers)



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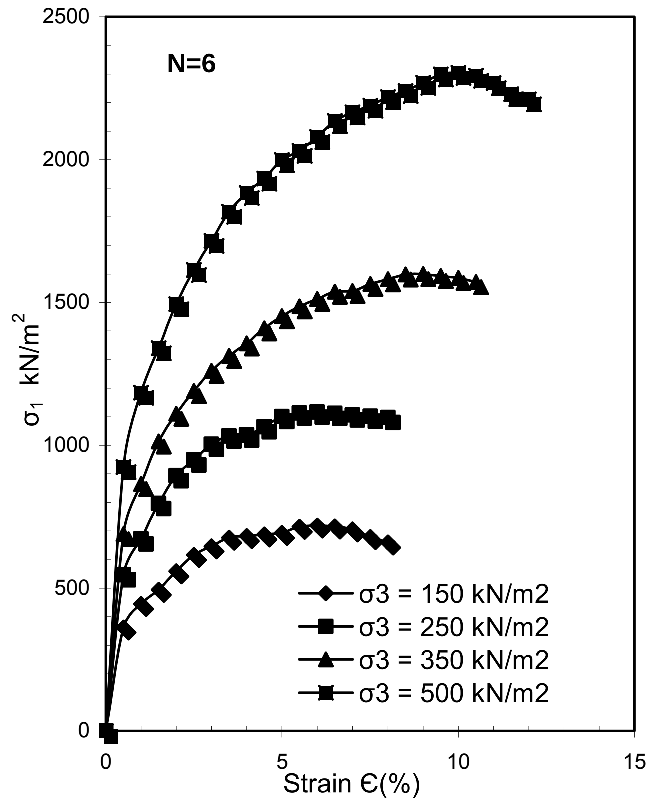
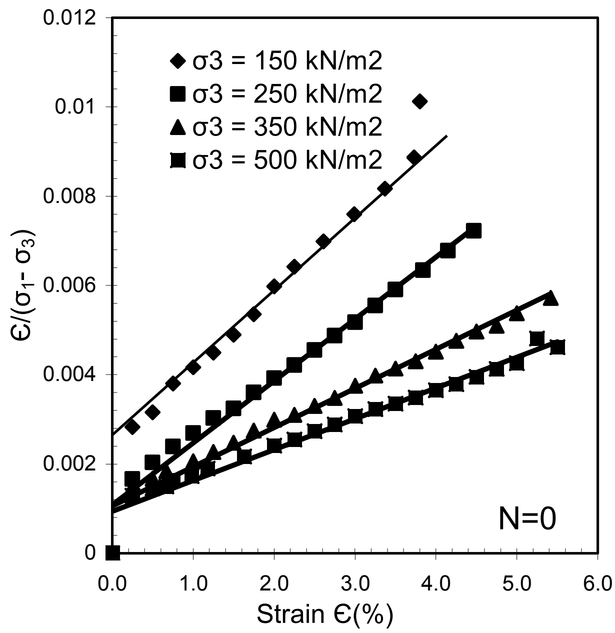
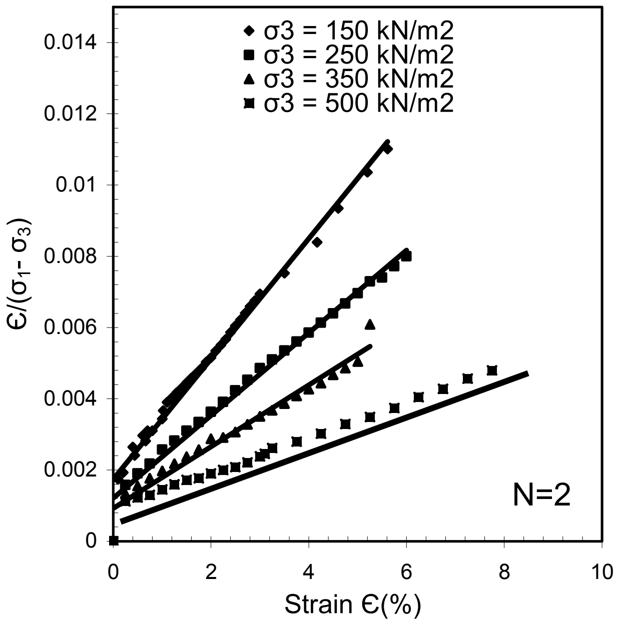


(d)

These comparisons indicate that the imposed strain level on the samples play an important role to increase the strength of the reinforced samples compared with unreinforced sample.

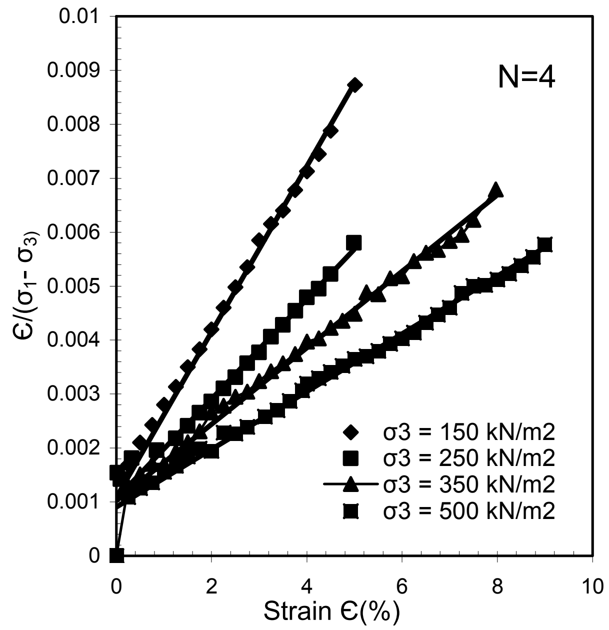


(a)



(b)

Figures 5 a-d shows the transformed plots of stress-strain behavior of unreinforced and reinforced sand sample. From these plots an attempt has been made to obtain the hyperbolic parameters of hyperbolic equation proposed by Kondner (1963). The values of stress at failure, parameters 1/a and 1/b are computed from the plots so drawn and tabulated in Tables 3, Table 4 and Table 5. From the values obtained for failure stress at peak, it is observed that the strength significantly increases for reinforcement layers up to four and there is no significant in peak failure stress when number of layers is increased to six.



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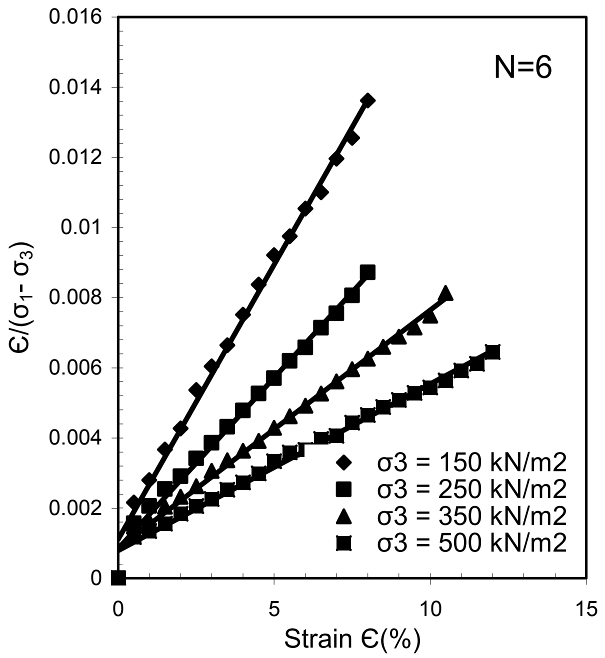


(d)

Figure 5: Transformed hyperbolic stress–strain plots for unreinforced and reinforced sand

Table 3: Failure Stress

σ_3 (kN/m^2)	*Failure stress at peak ($\sigma 1f$ kN/m^2)*			
	N=0	*N=2*	*N=4*	*N=6*
150	500.23	632.62	698.69	719.35
250	852.71	969.59	1085.32	1116.53
350	1180.97	1225.25	1506.55	1593.83
500	1595.06	1979.70	2026.25	2305.25

This indicates that optimum number of layers for better reinforcing effect may be restricted to four or five.

Table 4: Parameter 1/a

σ_3 (kN/m^2)	*Parameter 1/a (kN/m^2)*			
	N=0	*N=2*	*N=4*	*N=6*
150	$5.2*10^5$	$5.7*10^5$	$7.92*10^5$	$8.1*10^5$
250	$7.1*10^5$	$7.5*10^5$	$9.31*10^5$	$9.5*10^5$
350	$8.3*10^5$	$8.71*10^5$	$9.6*10^5$	$9.75*10^5$
500	$8.65*10^5$	$1.07*10^5$	$1.08*10^5$	$1.09*10^5$

Table 5: Parameter 1/b

σ_3 (kN/m^2)	*Parameter 1/b (kN/m^2)*			
	N=0	*N=2*	*N=4*	*N=6*
150	579.18	632.66	753.96	777.80
250	853.33	997.75	1100.00	1118.00
350	1227.27	1285.71	1550.00	1600.00
500	1678.32	1981.00	2100	2500.00

The values of hyperbolic parameters 1/a and 1/b are shown in Tables 4 and 5. As is seen in Table 4, the values of 1/a for different number of reinforcement layers has marginal effect and as the confining pressures increase the values significantly change. Similarly the values of 1/b in Table 5 show significant increase with increase in number of reinforcing layers. Also as the confining pressures increase there is increase in values of 1/b.

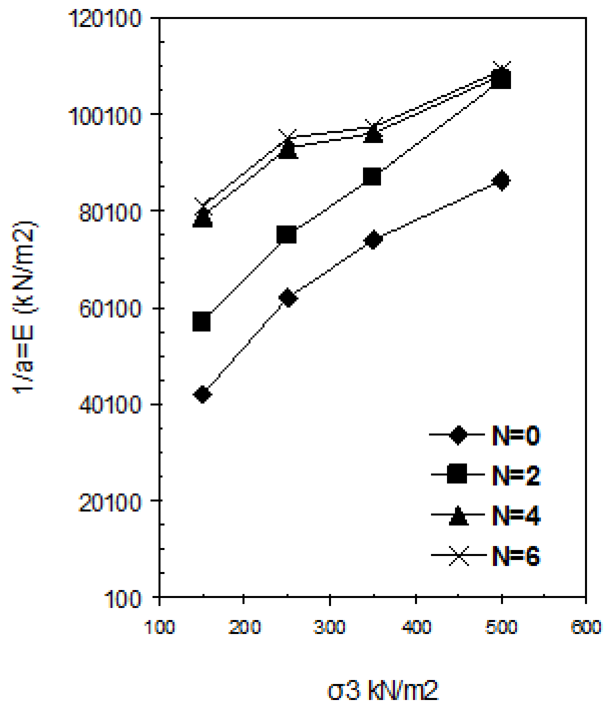
Figs. 6 a-b shows the influence of confining pressure on the hyperbolic parameters 1/a and 1/b. Both parameters show an increase in the value as the confining pressure increases. By linear regression of the values obtained for different reinforcement layers and different confining pressures, The following relationship holds good for tests performed on Ranipur sand both reinforced and unreinforced.

$$\frac{1}{a} = k_1 + A_1\sigma_3 \qquad \text{------------------} \quad (1)$$

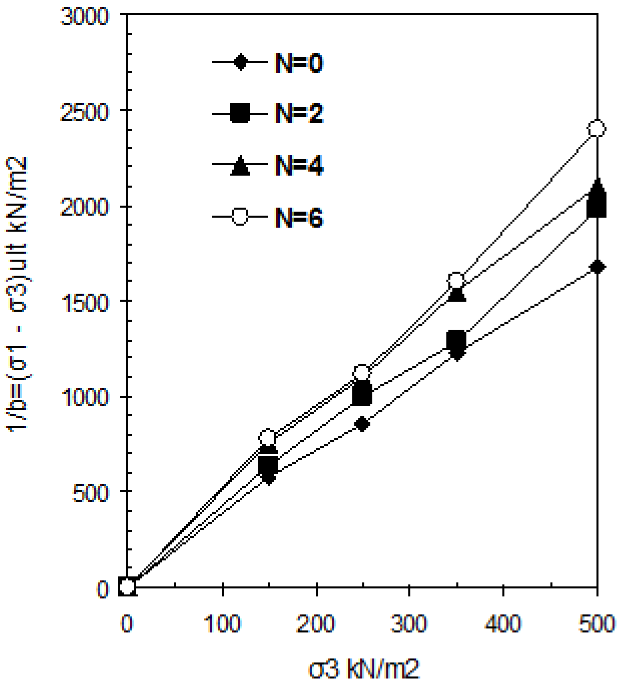
$$\frac{1}{b} = k_2\sigma_3 \qquad \text{--------------------} \ (2)$$

k_1, A_1 and k_2 are constants obtained from analysis of triaxial data and given in Table 6; σ_3 is the confining pressure in kN/m^2.

Thus if we know the number of layers and the confining pressure, we can obtain the constitutive parameters of the situation under consideration. Complete description and discussion is presented in Shah (2008).



(a)



(b)

Figure 6 Variation of 1/a and 1/b with confining pressure.

Table 6: Parameters of constitutive laws

D_r	*No. of Reinforcing Layers, N*	k_1	A_1	k_2
55%	0	27307	125.3	3.5293
	2	37692	150.19	3.9013
	4	69957	77.178	5.3292
	6	72290	75.673	5.7129

4. Conclusion

Results of triaxial compression tests carried out on dry beach sand reinforced with three commercially available geogrid provided the following main conclusions:

- Geogrid inclusion enhances peak strength, axial strain at failure and reduces post-peak loss of strength. The progress is more effective with a higher number of geogrid layers.
- Failure of reinforced sand was observed by bulging between geogrid layers. The values of hyperbolic stress-strain behavior of sand are significantly affected by presence of reinforcing layers.
- There is no significant increase in peak strength as the number of layers increases from four to six. Same trend is observed in values of 1/a and 1/b.
- The investigation demands much more elaborate experimental study taking into account all the possible influencing factors.

References

[1] Holtz, R.D., 2001. Geosynthetics for Soil Reinforcement. The Ninth Spencer J. Buchanan Lecture, College Station Hilton, TX 77840, p.1-19.
[2] Jewell, R.A., 1980. Some Effects of Reinforcement in the Mechanical Behavior of Soils. PhD Thesis, University of Cambridge, UK.
[3] Jewell, R.A., 1991. Revised Design Charts for Steep Reinforced Slopes Reinforced Embankment-theory and Practice.
[4] Thomas Telford, London, p.1-30.
[5] Reugger, R., 1986. Geotextile Reinforced Soil Structures.Processing of the Third International Conference on Geotextiles, Vienna, Austria, p.453-458.
[6] Schmertmann, G.R., Chourey, V.E., Johnson, R.D., Bonaparte, R., 1987. Design Charts for Geogrid Reinforced Soil
[7] Slopes. Processing of Geosynthetics, New Orleans, p.108-120.
[8] Leshchinsky, D., Boedcker, R.H., 1989. Geosynthetic reinforced soil structure. Journal of Geotechnical Engineering 115(10):1459-1478.[doi:10.1061/(ASCE)0733-9410(1989) 115:10(1459)]
[9] Michalowski, R.L., 1997. Stability of uniformly reinforced slopes. Journal of Geotechnical Engineering, 126(3): 546-556. [8] Broms, B.B., 1977. Triaxial Tests with Fabric-reinforced Soil. Proceedings of the International Conference on the Use of
[10] Fabric in Geotechnics, Ecole Nationale des Ponts et Chaussees, Paris, p.129-134.
[11] Holtz, R.D., Tobin, W.R., Burke, W.W., 1982. Creep Characteristics and Stress-strain Behavior of Geotextile Reinforced. Proceeding of the Second International Conference on Geotextiles, Las Vegas, USA, p.805-809.
[12] Nakai, T., 1992. Fundamental Investigation of Behavior of Reinforced Sand by Experimental and Numerical Methods. Proceeding of the Practice, Balkema, Rotterdam, p.135-140.
[13] Haeri, S.M., Noorzad, R., Oskoorouchi, A.M., 2000. Effect of geotextile reinforcement on the mechanical behavior of sand. Geotextiles and Geomembranes, 18(6):385-402. [doi:10.1016/S0266-1144(00)00005-4]
[14] Lambe, T.W., Whitman, R.V., 1979. Soil mechanics. Wiley Eastern Limited, New Delhi.
[15] Bishop, A.W., Henkel, D.J., 1969. The Measurement of Soil Properties in the Triaxial Test. William Clowes and Sons Limited, London and Beccles.

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