REINFORCING SOIL SUBGRADES WITH GEOSYNTHETIC COMPOSITE ELEMENTS

L'ARMATURE DES HERISSONS AVEC DES ELEMENTS GEOSYNTHETIQUES COMPOSES

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SYNOPSIS : Geosynthetic composite elements which are suitable for significantly high improvements in virgin soils of medium stiff variety are evolved in the study. When a soil having CBR=8 was reinforced with these elements, its CBR value improved by 118 to 300% depending on constituents of the element employed. A series of tests conducted with gradually increasing cyclic load gave insight into elastic and permanent deformations of the soil. Material parameters which can represent these components of deformations are defined and constitutive relation of the reinforced soil system is postulated. The material parameters obtained by conducting specific laboratory test, when used in the constitutive relation, can predict performance of the actual subgrade which is reinforced with such composite elements. Another series of tests conducted by repeating loading and unloading at three different load levels gave idea of vibration dissipation capacity of such reinforced subgrades under repetitive type traffic loads of three different intensities.

INTRODUCTION

Geosynthetics are generally recommended for improvement of road subgrades having CBR less than 2 or 3 (Giroud and Noiray, 1981). But sometimes, high speed heavy traffic may demand improvement of subgrades having CBR of 4 or 6. Even soil with bearing capacity of 1 or 1.5 kg/sq.cm. may need improvement if it is below a railway track where stresses exceed 2.5 kg/sq.cm. (Azeem, 1992). In problems of road/railway subgrades, the stresses induced in the portion beneath the loading areas are many times more than the stresses in the adjacent areas in which the geosynthetic is anchored. A layer of nonwoven geotextile, although an excellent separator and drainage-cum-filtration medium, cannot withstand such differential stresses and may fail in a short time. In case of woven geotextile reinforcements, since surrounding medium stiff soils will experience low strains, only low strength of initial modulus would be mobilised. Geogrids allow cross movement of soil grains and do not derive benefits of confining the soils. It can be inferred that high strength improvement may require pad-like composite reinforcing element which may derive its strength by confining sand grains inbetween layers of high modulus geotextile. One such element, prepared by covering both faces of a thin layer of a selected variety of sand with a woven fibre-glass fabric gave optimum improvement of 100% at a settlement limited to 7.5% width of model footing tested on dense sand (ϕ = 37 degrees) (Patel, 1981).

Another composite reinforcing element, formed by placing a suitable geogrid in the middle of a thin layer of selected sand and by covering its one or both faces by a woven geotextile, gave improvement of 118 to 300% (Patel and Rana 1990, Patel and Gurappachari, 1991). This paper reports details of the laboratory static and cyclic load tests conducted on local clayey soil specimens reinforced with this geosynthetic composite element. The study gives insight into mechanism responsible for the reinforcing action of the element.

LABORATORY TEST DETAILS

CBR Load Tests

California bearing ratio test is widely adopted for testing subgrades and its design charts are well accepted for design of road pavements. Simplicity and accuracy are the added advantages. Hence this test is increasingly becoming popular for such studies (Mandal and Mohan, 1989, Murtaza et al., 1988).

Moreover, the CBR values are found at 2.5 mm and 5 mm penetrations which correspond to 5% and 10%of plunger diameter. These values of penetrations are comparable with the 7.5% to 8% value at which the reinforced sand bed begins to yield (Patel, 1981). Also the actual reinforced soil subgrade is more precisely simulated in CBR test than in the triaxial, direct shear and model footing tests on such soils.

However, if the CBR loading plunger is considered as a small circular footing, the standard mould of diameter equal to 3 times the plunger diameter is found inadequate for the reinforcing action of the geosynthetic composite elements (Patel and Rana, 1990). Hence, in the present study, the diameter of mould was increased to 5 times the plunger diameter. Other factors like thickness of specimen, compaction energy per unit volume, surcharge pressure etc. were kept same as in the standard test.

Materials

Local clayey soil adopted as virgin soil was of CI type, having W_L = 37, Wp = 17, fine sand = 30% and silt+clay = 47%. At Standard Proctor test energy, the soil had m.d.d. = 18 kN/cu.m., o.m.c.=14%, c=0.45 kg/sq.cm., ϕ = 26°-30' and CBR = 7.8.

Local Bhadarpur sand which was used in forming the reinfocing element had 18% coarse, 45% medium and 37% fine sand grains, uniformity coefficient = 2.8 and coefficient of curvature = 0.84. At the placement density of 17 kN/cu.m., it had I_D = 60% and Ø = 42 degrees.

Other materials used in forming the reinforcing element were Polystron Geomat (Poly G Mat), Netlon grid CE-121 and fibreglass woven rovings (FGWR). Their physical and mechanical properties are presented in Table-1.

Table-1 : Properties of Geosynthetics

Generic Name	CE-121	PolyGMat	FGWR
Form	Geogrid	Woven Geotext.	Woven Geotext.
Make	Netlon India	Gujarat Filament	CEAT Tyres
Basic Material	HD Polyethylene	Polypropylene	Glass(E) Fibres
Mass(gm/sq.m)	730	250	360
Tensile Strength (wide strip test) :			
max. P(kN/m)	9	28 to 40	V.high
%strain (at max.P)	17 to 27	17 to 21	3 to 10
av.modulus (per unit strain)	4620 kg	5000 kg	V.high
Friction angle (with Bhadarpur sand)	38°	36° - 50'	--

Test Programme

Following 3 series of CBR tests were conducted :

Series-A : with increasing static load,
Series-B : cyclic load, increased in equal increments, with one cycle of loading and unloading at each increment,
Series-C : cyclic load, applied and removed thrice, at load levels corresponding to 4,8 and 12 mm penetrations observed in corresponding static CBR tests.

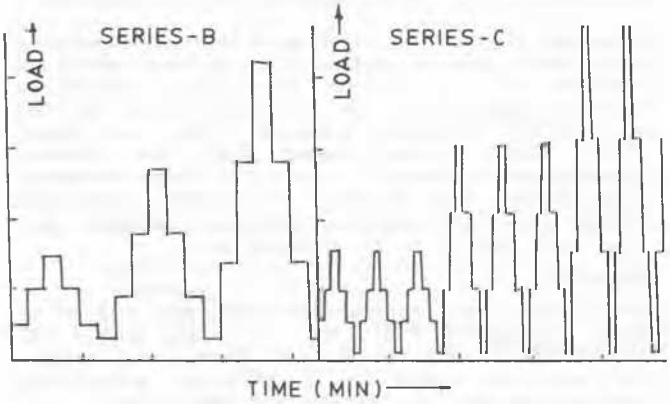

Fig.1 : Sequence of Loading and Unloading

Sequence of loading and unloading, as a function of time, used in the last two series is described in Fig. 1. Almost steady rate of loading and unloading was maintained.

These three series of tests were conducted on the unreinforced and 5 different types of reinforcing elements as per the following detail :

(1) Unreinforced soil specimen,
(2) Specimen reinforced with a layer of Bhadarpur sand + a layer of CE-121,
(3) as of (2) + a layer of Poly G Mat,
(4) as of (2) + 2 layers of Poly G Mat,
(5) as of (2) + a layer of FGWR,
(6) as of (2) + 2 layers of FGWR.

These tests were designated as follows :-

T1 to T6 - Series A static CBR tests,
C1 to C6 - Series B cyclic tests,
D1 to D6 - Series C cyclic tests.

RESULTS OF SERIES-A

Performance of Reinforcing Elements

Results of test T-1 conducted on unreinforced soil and of tests T-2 to T-6 conducted on the soil reinforced with 5 different geosynthetic composite elements are presented in Table-2.

Table-2 : Performance under static loading

Test No.	Element Description	CBR value	%-improvement
T-1	Unreinforced soil	7.80	(basic value)
T-2	+CE-121 + sand	15.60	100.0%
T-3	+CE-121+one PGM+sand	17.03	118.3%
T-4	+CE-121+two PGM+sand	22.87	193.2%
T-5	+CE-121+one FGWR+sand	23.60	202.6%
T-6	+CE-121+two FGWR+sand	31.38	302.3%

The results indicate improvement of 118% with composite element having one layer of Poly G Mat and of 193% with two layers of Poly G Mat. FGWR is much stiffer geotextile. Hence the improvement recorded with the composite element having one layer of FGWR was 202% and with two layers of FGWR, it was 302%.

Contribution of the Reinforcement

Load-penetration diagrams were plotted for the six tests conducted under this series. Difference of diagram of the reinforced specimen (any of T-2 to T-6) and of the unreinforced specimen (T-1) indicates contribution of the reinforcing element. A typical diagram (T4 - T1) which indicates contribution of the reinforcing element of test T-4 is shown in Fig.2. Similar diagrams were plotted for the other four composite elements. Their study gave the following graphical method of estimating the reinforcing contribution of the system :

1. Draw a tangent to the load-penetration diagram of the reinforced soil at a penetration equal to 7.5% plunger diameter, viz. at 3.75 mm. It may be noted that this penetration value corresponds to begining of yield of the reinforced soil bed (Patel, 1981) and the slope of curve-T4 in Fig.2 also changes significantly after this value of penetration.

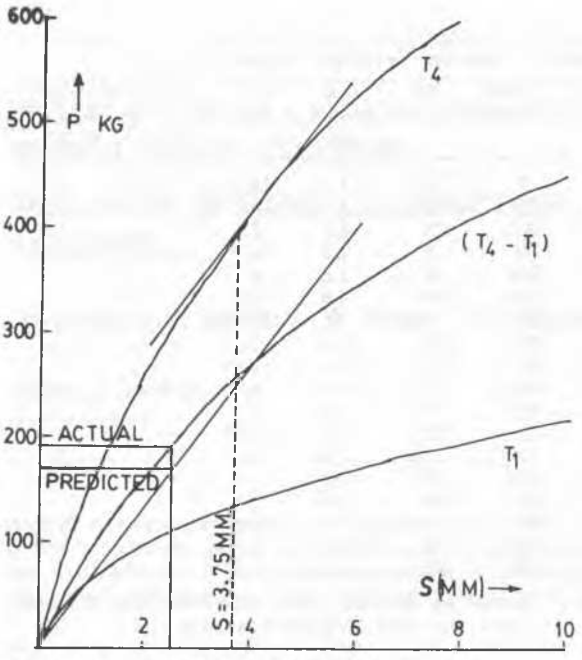

Fig.2 : Contribution of Reinforcing Element

2. Draw a line parallel to this tangent, passing through the origin.

3. Find load and CBR value corresponding to 2.5 mm penetration by this new line. It gives the estimation of reinforcing action in terms of increase in CBR value. This is an approximate method and it was found to give results with safe side error upto 20%.

RESULTS OF SERIES-B

A reversible but continuously increasing load applied in regular increments in this series made it possible to separate the recoverable-Se and nonrecoverrable-Sp components of penetration of the plunger at different load levels. A set of P vs Se and P vs Sp diagrams obtained from observations of cyclic load test C4 and P vs S diagram of corresponding static load test T4 are presented in Fig.3.

Constitutive Relationship

Fig.3 shows that plot of P vs Se is almost linear and that of P vs Sp is curvilinear. So if the former plot is represented by its slope Cu and later is represented by standard hyperbolic transformation (Duncan and Chang, 1970), then for a given load P, settlement of the reinforced soil system can be given by,

$$S = Se + Sp = \frac{P}{C_u} + \frac{a \times P}{1 - b \times P} \qquad (1)$$

Where C_u, a and be are material parameters whose meaning and values for test C4 are shown in Fig.3. Using these values in equation-(1), the performance is predicted and plotted by dotted line. The predicted performance is almost accurate.

Values of parameters C_u, a and b obtained for the reinforcing systems studied under this series are given in Table-3.

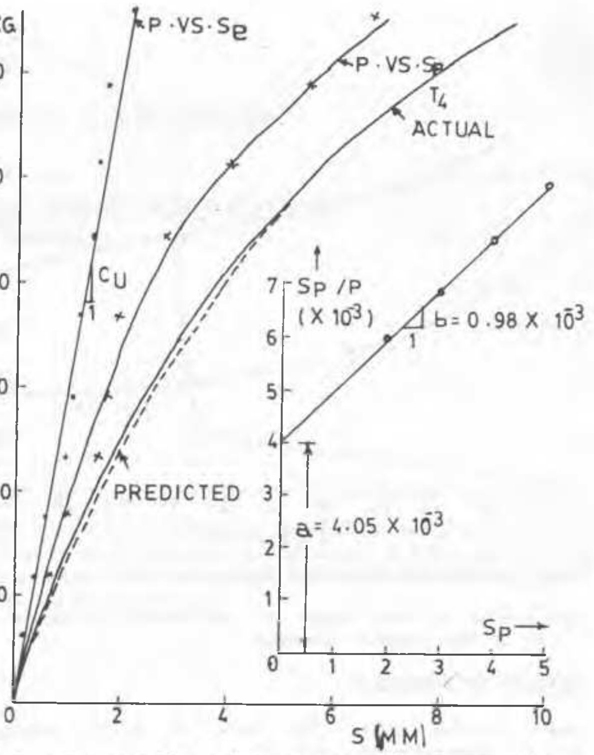

Fig.3 : Recoverable and Nonrecoverable Components

Table-3 : Results of Series-B

Test	C_u kg/mm	a($\times10^{-3}$) mm/kg	b($\times10^{-3}$) kg^{-1}	Eip kg/mm	Pp-ult kg
C1	217	6.9	4.12	144.9	243
C2	267	4.7	1.70	212.8	588
C3	280	4.5	1.55	222.2	645
C4	294	4.1	1.00	243.9	1000
C5	856	3.7	0.93	270.3	1075
C6	1077	3.0	0.83	333.3	1205

The inverse of parameter-a which indicates initial plastic modulus Eip and inverse of the parameter-b which indicates ultimate load corresponding to plastic component of penetrations Pp-ult are also given in Table-3. If C_u can be regarded as elastic modulus, comparison of its values with the corresponding values of Eip indicate that reinforcing the soils with stiffer variety of composite elements improve elastic modulus more significantly in comparison to that of Eip. Values of Pp-ult are almost proportionally increasing with the increase in CBR values of the reinforced systems.

Variation in values of the parameters a, b and C_u with the corresponding CBR values of the soil specimens are shown in Fig.4. The inferences are clearly explained in the figure.

Thus the constitutive relation of Eqn (1) alongwith values of material parameters obtained by conducting laboratory tests or assumed values can be used to predict performance of actual reinforced soil subgrades under vehicular loads. Recoverable and nonrecoverable components of settlement of actual subgrade can also be estimated by this approach.

Addition of recoverable and nonrecoverable components of settlements in Eqn. (1) is found to give accurate

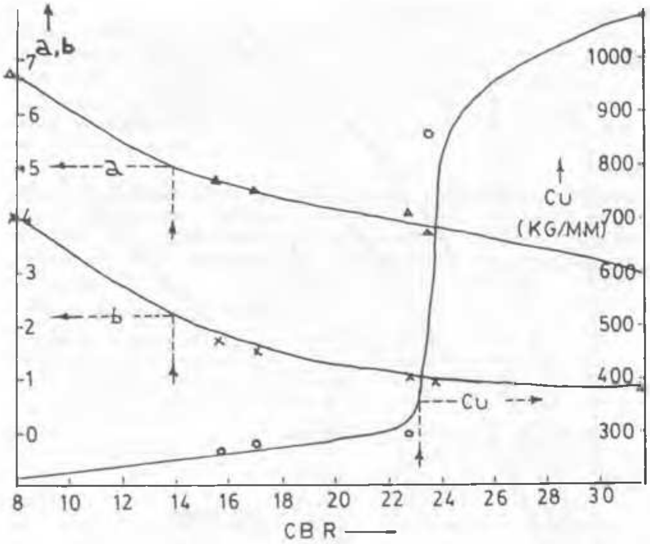

Fig.4 : CBR Values related to a, b & c_u

predictions in the range of settlements equal to 1% to 15% of the plunger diameter.

RESULTS OF SERIES-C

Based on the static CBR tests T1 to T6 conducted on the unreinforced and the other five reinforced soil specimens, load levels corresponding to 4 mm, 8 mm and 12 mm penetrations were decided. Then in the series-C, for each of the corresponding tests D1 to D6, after gradually loading the specimen upto the stage of 4 mm penetration, loading and unloading was repeated thrice. Similarly, next at load level of 8 mm and finally at load level of 12 mm penetrations, loading and unloading was repeated thrice.

When load-penetration diagrams were plotted, at each of the load levels, repetition of loading and unloading gave a hysterisis loop. Corresponding to a particular load level, average area of the 3 hysterisis loops was designated as ΔE and total area under load -penetration diagram upto the first unloading measured from the origin was designated as Eo.

The ratio $\Delta E/Eo$ is designated as damping capacity since it indicates the vibration dissipation capacity of the subgrade under repetitive traffic loads. Damping capacity-Dc found for the tests are presented in Table-4. Also the Dc-values, as compared with those at P=243 kg of test D-1 and at P=344 kg of test D-2 are indicated in the same table. Dc-values at load levels of 243 and 344 kg were obtained by linearly interpolating from the adjacent values observed at the load levels of the experiments.

Comparison made in the last two columns of the table indicated that the damping capacity of the reinforced soil subgrade improves with the stiffness of the geosynthetic composite element.

CONCLUSIONS

1. The proposed geosynthetic composite elements can improve medium stiff soil subgrades by about 118% to 300% depending upon the number of layers and strength modulus of the geotextile.

2. The constitutive relation relating load and penetration values in terms of material parameters c_u, a and b can be used for predicting performance of the actual subgrade reinforced with the proposed geosynthetic composite element. The material parameters can be determined by conducting cyclic CBR test of series-B type in the laboratory.

3. The damping capacity of the soil subgrade can be improved remarkably by reinforcing it with the proposed geosynthetic composite elements.

Table-4 : Damping Capacity Values

Test	Load Level	Eo	ΔE (aver-age)	Dc % (E/Eo)	% improvement in Dc	
					P=243	P=344
D-1	81	8	1.2	14.3	2.1	—
	162	46.2	4.7	10.2		
	243	223	4.7	2.1		
D-2	172	9	1.2	13.3	9.75	4.7
	344	37	1.8	4.7		
	516	176	4.4	2.5		
D-3	188	9.4	1.1	11.7	9.92	6.64
	376	43	2.4	5.6		
	596	201	3.1	1.54		
D-4	240	9.1	0.8	8.8	8.8	8.33
	485	34	2.6	7.7		
	725	198	3.1	1.57		
D-5	245	9.8	0.8	8.2	8.2	7.1
	490	39	2.1	5.4		
	735	188	2.8	1.5		
D-6	291	6.2	0.9	14.5	16.9	12.9
	584	38	2.2	5.8		
	847	177	3.5	1.98		

Note : Areas of Eo and ΔE measured for different tests are with different scales.

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