A New Material for Improving Bearing Capacity

Faisal Shamsher
Faculty of Engineering, University of Aden

Abstract

This paper presents the results of triaxial tests conducted on large diameter specimens to assess the improvement in bearing capacity of granular material due to geogrid reinforcements both with micro-mesh elements and circular discs.

1. Introduction

Provision of a sand layer over a soft clay as a foundation bed has long been known to be beneficial in terms of improved bearing, drainage and ease in construction. In the recent past,. further improvements in terms of reinforcing the sand bed have come into vogue. These reinforcements historically consisted of even tree branches and wooden logs, now comprise of strips or rods of metal , and more recently of geotextiles and geogrids, in order to improve the bearing pressure and reduce settlement. This paper presents result of a study carried to assess the improvement in bearing capacity arising in granular material due to geogrid reinforcement based upon result of large diameter triaxial tests.

2. Literature Review:

2.1 Reinforcements Materials:

Several research workers have studied the problems of shallow foundation improvement, viz.,soil strengthening by root, rope fibre materials (Akinmusurn & Akinbolade, 1981). Man-made fibres (Andersland & Khattak, 1979; Hoare, 1979;Leflaive,1982 and Gray & Ohashi, 1983). Recent research in the use of randomly distributed polymeric mesh elements in soil by Mercer et al.(1984) and McGown et al.(1985) have shown that polymeric materials can be placed in a soil to create a composite material with improved stress resistance. Also the meshes interlock with the soil particles produce a strengthening at the meso-scale. The principal advantage of using mesh elements is their interlock action. This occurs at two levels with ribs of individual mesh elements interlocking with groups of soil particles to form an aggregation of particles (Fig. 1a), then adjacent aggregation interlocking to form a coherent matrix (Fig. 1b)

2.2 Modes of Failure:

The geosynthetics are invariably provided along with granular fills so that the system may be termed as reinforced granular base or slab. The strengthening of soft soil by the reinforced slabs leads to an increase in stiffness with a consequent decrease of settlement and increase of the bearing capacity. Binquet and Lee(1979) identified three modes of failure of reinforced earth slabs, viz., (i)failure of soil above the top reinforcement(Fig.2a) (ii) failure by slip (Fig.2b) and (iii) tensile failure of reinforcement (Fig. 2c). Koerner (1985) has suggested yet another mode of failure due to long term (creep) settlement under sustained surface loads and subsequent stress relaxation (Fig.2d).

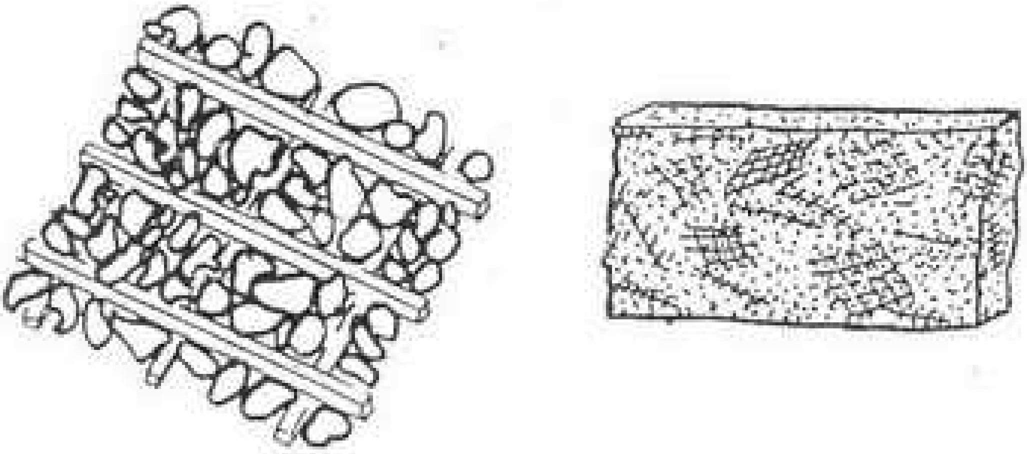

Fig.1: Interlock mechanism for mesh elements between
(a): groups of particles (b): adjacent aggregations

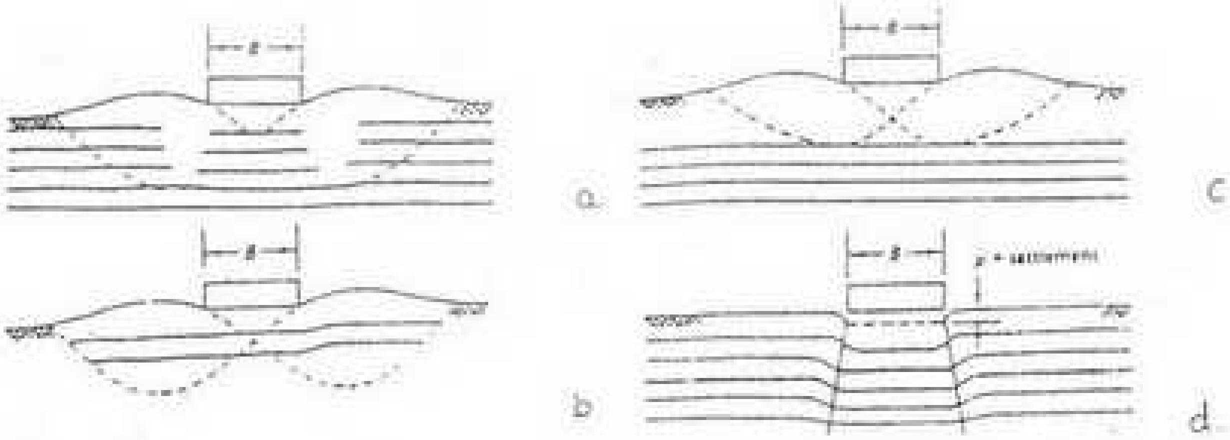

Fig.2: Failure modes

3. Experimental Programme

3.1 Materials Used

3.1.1 Soil

The soil used is stone dust, comprising of sub-angular particles ranging in size from 0.04 to 4.0 mm diameter with a uniformity coefficient of 3.3 .

3.1.2 Mesh Elements:

The geogrid CE121 manufactured by Netlon India has been used. The physical and mechanical properties of the grid are given in Table 1. Many possible mesh element sizes were subjected to some exploratory tests and from these 50 mm square elements were chosen as suitable.

Table 1 : Properties of Mesh Elements

Type	CE121
Polymer	Polyethylene
Aperture size	8x6 mm
Mass/unit area	730 g/m^2
Tensile strength	7.68 kN/m
Max. extension at max. load	20.2%
Load at 10% extension	6.8 kN/m
Elongation at peak strength	3.2%

3.2 Triaxial Tests:

Saturated stone dust alone and stone dust mixed with various percentages upto 1.4 (by weight) of mesh element were deposited in layers in water into a split mould forms. The specimens were 100 mm diameter and 200 mm high. Each layer was compacted to achieve required density by vibration at constant frequency. The achieved dry density of stone dust and stone dust - mesh mixtures varied from 17.6 to 18.0 kN/m^3. Cylindrical specimens have also been prepared with stone dust, at the same density but reinforced with circular discs of 100 mm diameter of the geogrid, the number of discs used were 1,2 and 7. Conventional consolidated drained triaxial tests were conducted on the specimens, with cell pressures of 50, 100 and 200 kPa at a deformation rate of 0.2 mm/minute.

4. Results:

Figure 3 presents stress-strain curves obtained with disc reinforced triaxial specimens at confining pressures of 50 and 200 kPa. The improvement in strength with reinforcement is amply clear at both the confining pressures, however it is more marked at lower pressure. Figure 4 gives the stress-strain curves for micro-mesh reinforced specimens at two different confining pressures. The behaviour in this case seems to be linearly (or gradually) improving with percentage reinforcement.

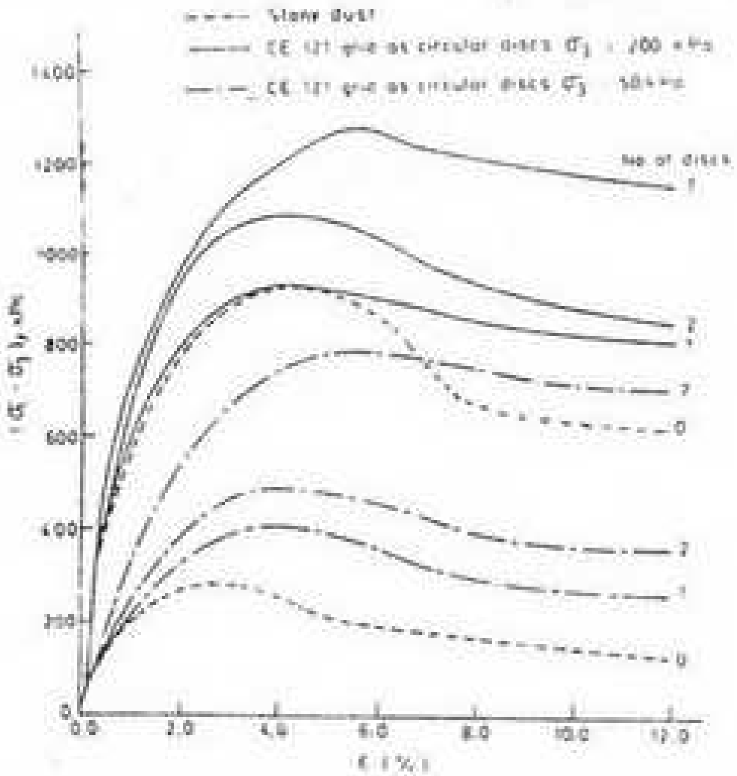

Fig. 3: Stress-Strain relationships for reinforced stone dust with geogrid discs

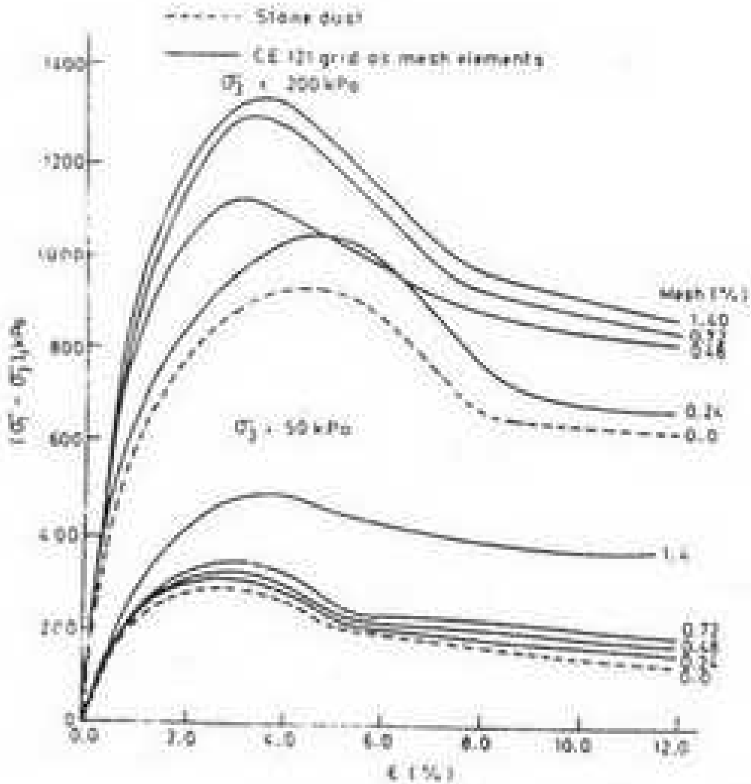

Fig. 4: Stress-Strain relationships for reinforced stone dust with micro-meshes

A comparison of the behavior with disc and micro-mesh reinforced specimens is presented in Fig. 5. It may be observed that the improvement in strength is marked at low confining pressure for disc reinforced case. This improvement decreases at the pressure of 100 kPa and the strength is nearly same at confining pressure of 200 kPa. It is also seen that the micro-mesh reinforced specimen is more stiff than the disc reinforced one.

It should however be borne in mind that the results for disc reinforced specimen cannot be directly utilised for design purposes, as it requires a discrete approach (Raju & Rao,1990). On the other hand one can use the results of the micro-mesh reinforced specimen in design by using the global approach.

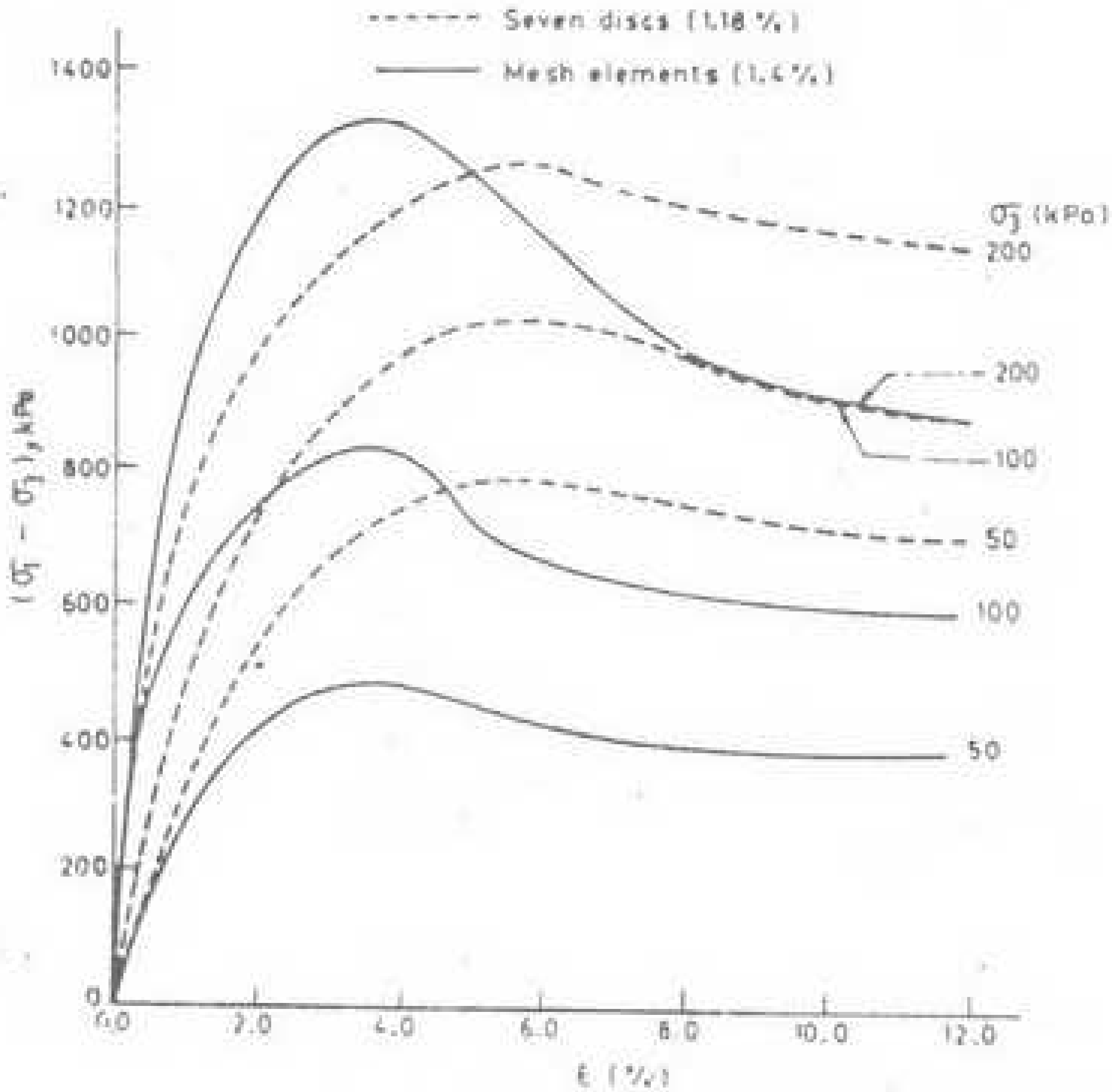

Fig. 5: Comparison of disc and micro-mesh for reinforced stone dust

The p-q plots for stone dust alone, stone dust mixed with mesh elements, unreinforced and reinforced with discs for different ranges of confining pressures are presented in Fig.6 and Fig.7. For different percentages of mesh elements and numbers of reinforcement layers.The failure envelopes are observed to be bilinear in nature for both type of reinforcement.These figures indicate that the range of confining pressure upto 100 kPa corresponds to initial linear portion, whereas the second linear portion corresponds to the range of 100-200 kPa. The values c' and φ' obtained are presented in Tables 2and 3. A study of these tables reveals the following:

- For unreinforced stone dust the values of c' is zero upto σ_3=200 kPa.
- For both types of reinforcement for σ_3 upto 100 kPa, c' is found to be zero. With increased percentage of mesh elements or number of circular disc, there is similar increase in φ'.
- For reinforced mesh element for σ_3 = 200 kPa, the values of φ' nearly remains constant at 40°, whereas the value of c' increases to a maximum of 65 kPa. In case of reinforced with circular discs, c' increase up to 152 kPa, and φ' show no stable value.

Typical calculations have been carried out using the data of Meyerhof(1974) except that the sand layer is replaced by reinforced stone dust. Typical improvement in bearing capacity ratio BCR (ratio of ultimate bearing capacity with and without reinforcement) using this data has been presented. The computation of ultimate bearing capacity (UBC) has been carried out using the approach suggested by Meyerhof (1974). Figs.8 and 9 present a summary of the computation. It is clear that the BCR improves with H/B and % of reinforcement.

Typical calculations of settlement have been also computed based on data obtained from experimental results. The computations of settlement have been carried out using the approach suggested by Bowels (1982). The summary of the computation is presented in Fig.10. It can be seen from this figure that with increase in mesh percentages the settlement decreases upto 0.8% mesh and beyond this it remains constant.

Table :2 Strength Parameters for Stone Dust Reinforced with Micro-mesh.

σ_3 kPa	Parameter	Stone-dust	Stone Dust Reinforced mesh CE121 mesh (50x50)% wt.			
			% 0.24	% 0.48	% 0.72	% 1.4
50-100	c'(kPa)	0	0	0	0	0
	φ' (deg.)	44.4	48.8	50.1	51.4	54.0
100-200	c'(kPa)	0	21.0	36.0	36.4	65.0
	φ' (deg.)	44.4	45.5	45.5	46.6	46.0

Table:3 Strength Parameters for Stone Dust Reinforced with Geogrid Discs.

σ_3 kPa	Parameter	Stone dust reinforced CE121 circular disc No. of Layers			
		0 % 0	1 % 0.17	2 % 0.34	7 % 1.18
50 - 100	c'(kPa) φ' (deg.)	0 44.4	0 51.4	0 57.0	0 60.0
100-200	C'(kPa) φ' (deg.)	0 44.4	166 29.0	138 34.5	152 37.8

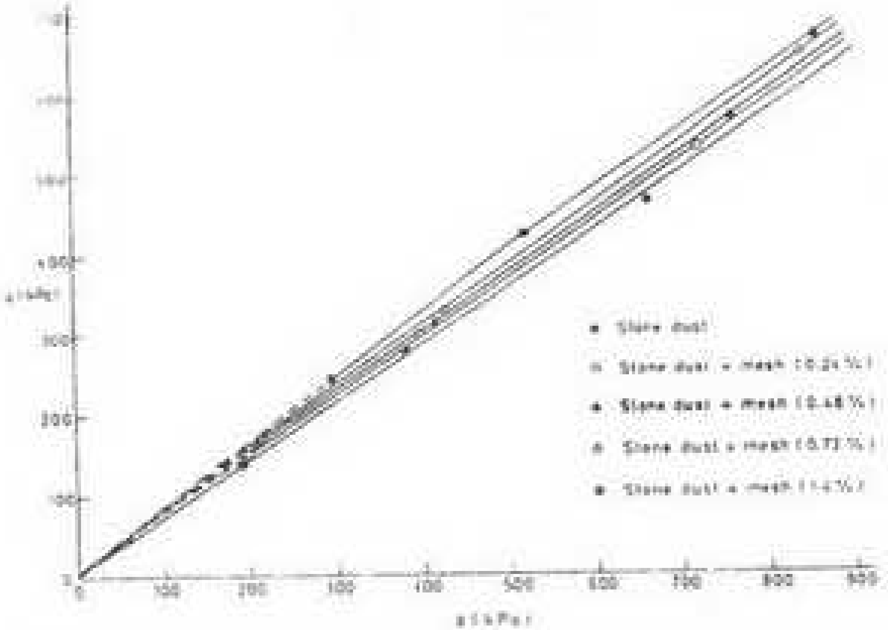

Fig. 6: p-q plots for stone dust reinforced with micro-mesh

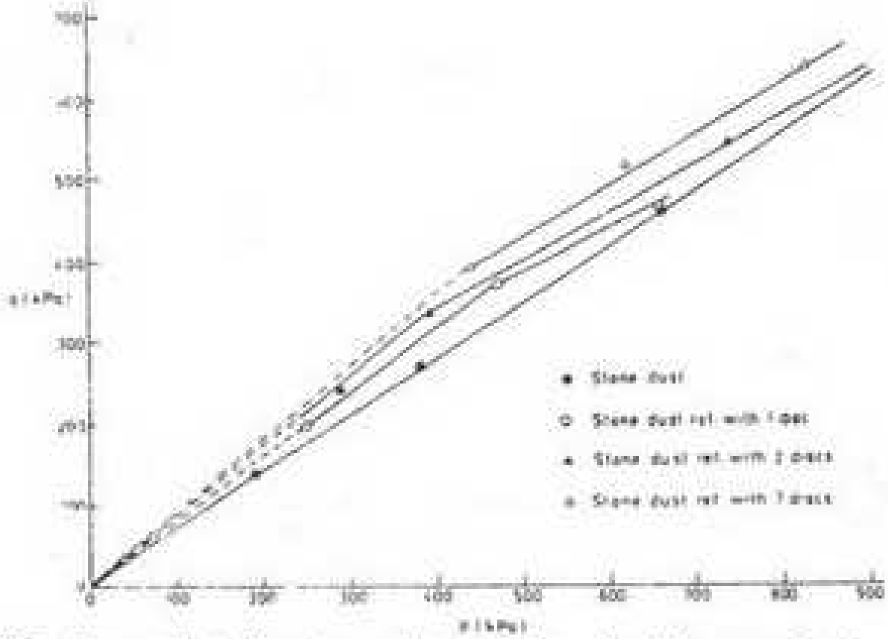

Fig. 7: p-q plots for stone dust reinforced with geogrid discs

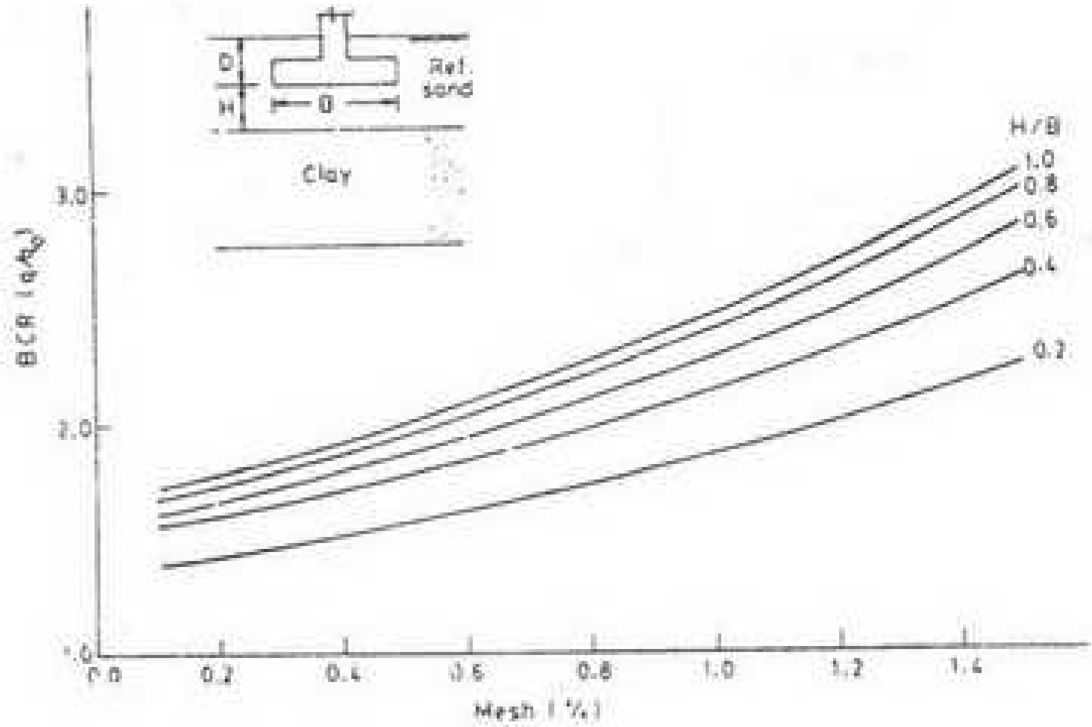

Fig. 8: Bearing Capacity ratio variation with micro-mesh percentages

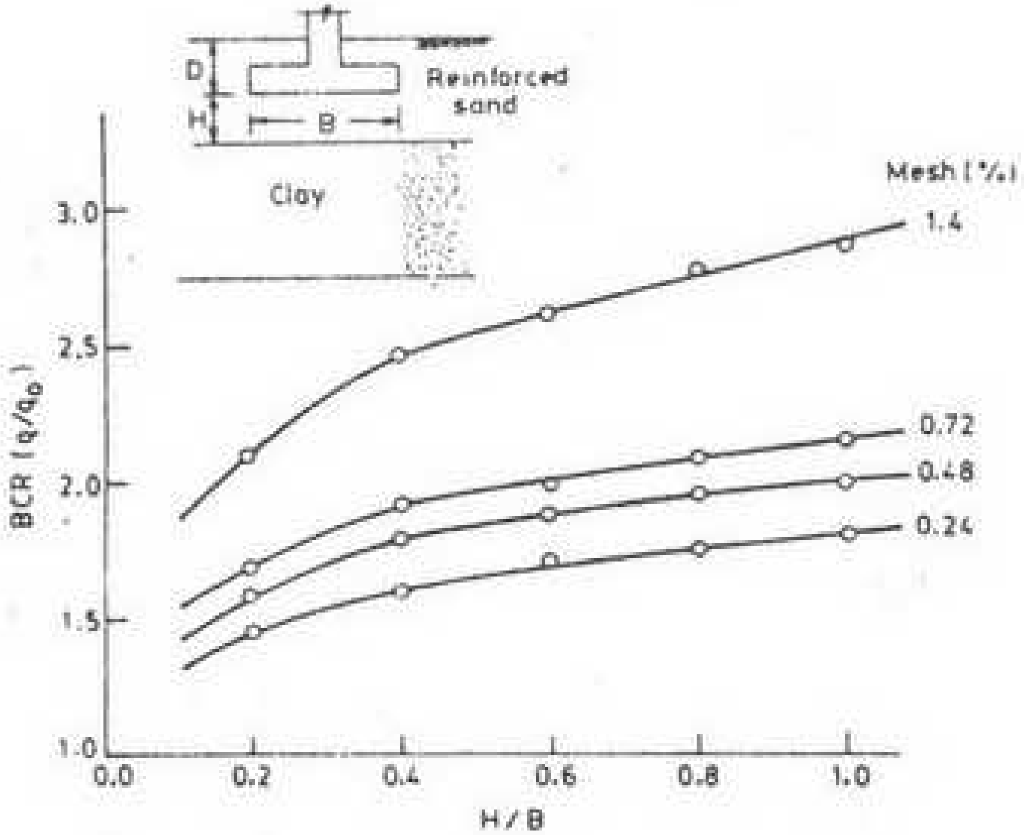

Fig. 9: Bearing Capacity ratio variation with H/B

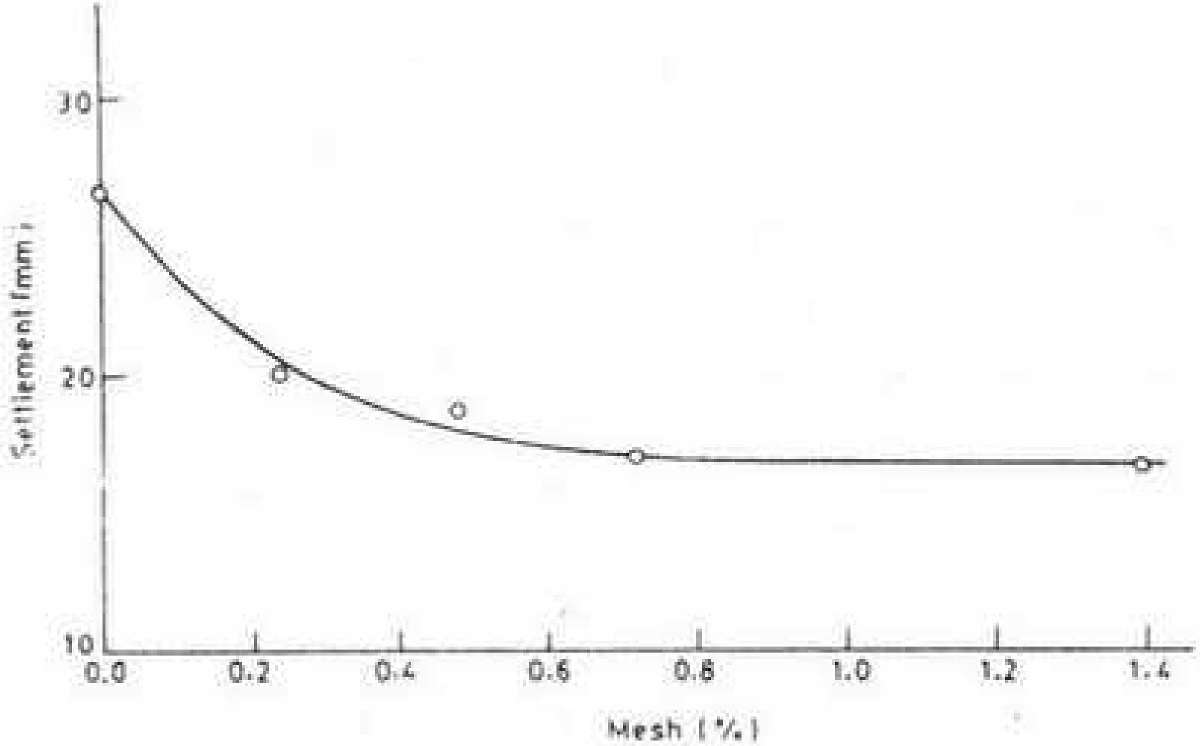

Fig. 10: Variation of settlement with micro-mesh percentages

5. Conclusions:

The use of randomly distributed polymeric micro-mesh in stone dust has been shown to yield significant improvement in stress resistance.

A comparison of the behaviour of disc and micro-mesh reinforced specimens revealed that the improvement in strength is marked at low confining pressure for disc reinforcement, whereas at higher confining pressure the strength is nearly the same.

The contribution of micro-mesh in improving the BCR increases with increase in its percentage and reduction in H/B ratio. Settlement decreases with increase of mesh percentage upto 0.8% mesh, beyond this it remains constant.

References:

Akinmusuru, J.O.& Akinbolade, J.A.(1980). "Stability of Loaded Footings on Reinforced Soils". ASCE J. Geo.Engg. Div.(107), GT6.16320, June pp. 819-827.

Andersland, O.B. & Khattak, A.S.(1979)."Shear Strength of Kaolinite/Fibre Soil Mixtures" , Int. Conf. On Soil Reinforcement, Reinforced Earth and Other Techniques, Paris,Vol.1,pp.11-16.

Binquet, J. & Lee ,K.L.(1975). "Bearing Capacity Tests on Reinforced Earth Slabs" ASCE, J. Geo. Engg. Div.(101). GT12, Dec,pp.1241-1255.

Bowles ,J.E.(1982) "Analysis And Design of Foundation". Second Edititon. International Student Edition. McGraw-Hill.

Gray, D.H. & Ohashi, H. (1983). "Mechanics of Fibre Reinforcement in Sand" ASCE J.Geo. Engg. Div. 109, March pp. 355-353.

Hoare, D.J.(1979)," Laboratory Study of Granular Soils Reinforced with Randomly Oriented Discrete Fibres - A Laboratory Study", Int. Conf. on Soil Reinforcement : Reinforced Earth And Other Techniques, Paris. Vol.1 pp.47-52.

Koerner, R.M.(1985) "Designing with Geosythetics", Prentice Hall, Englewood Cliffs,N.J.

Leflaive, E.(1982). "The Reinforcement of Granular Materials with Continuous Fibres". Proc. 2nd Int. Conf. on Geotextile. Las Vegas. Vol.3.pp.721-726.

McGown , A. Andrawes , K.Z., Hytiris, N., Mercer, F.B.(1985). "Soil Strengthening Using Randomly Distributed Mesh Elements", Proc. of 11th ICSMFE, San Francisco, pp. 1735-1738'

Mercer, F.B.Andrewes, K.Z., McGown,A. & Hytiris, N. (1984). " A New Method of Soil Stabilization", Proc.Sym. Polymer Grid Reinf. in Civil Engg., London , Paper 8.1, pp.244-249.

Meyerhof, G.G.(1974). "Ultimate Bearing Capacity of Footing on Sand Layer Overlying Clay", Canadian Geotechnical Journal,Vol.11 No. 2, pp. 223-229.

Raju Suryanarayana,G.V.S & Rao, Venkatappa G. (1990). "Reinforcement with Geotextiles" Engineering with Geosynthetics, Tata McGraw Hill, New Delhi, pp. 197-236.

مواد جديدة لتحسين مقاومة التربة

فيصل شمشير

كلية الهندسة – جامعة عدن

الملخص

تعتبر تقنية تسليح التربة من التقنيات الحديثة المستخدمة في مجال تحسين خواص التربة في عدة دول بالعالم. حيث انتشرت هذه التقنيات انتشارا واسعا خلال العقدين الماضيين في مجال المنشآت الترابية مثل الجدران الترابية الواقية المسلحة، طبقات التربة المسلحة لتحمل أساسات المباني وفي طبقات تربة الطرق. تعتبر هذه التقنية واحدة من أنجح التقنيات المستخدمة حديثا وذات سرعة فائقة في التنفيذ خلافا للطرق التقليدية المتبعة في تحسين وظيفة التربة.

اعتمدت الدراسة المقدمة في هذه الورقة على النتائج المخبرية التي اجريت على تربة خشنة مسلحة بواسطة مادة الجيوجريد، حيث سلحت التربة بواسطة دوائر وبشكل قطع صغيرة مربعة خلطت بنسب مع التربة بشكل عشوائي في قالب مكون من (قطر ١٠٠ مم وارتفاع ٢٠٠مم) تمت التجارب بواسطة جهاز الإسقاط الثلاثي، استخدمت النتائج المخبرية لتساعد في الجانب النظري / التطبيقي على طبقات التربة الضعيفة حيث وضعت التربة الطينية أسفل والتربة المسلحة بواسطة شرائح الجيوجريد فوقها وكذلك مسلحة بواسطة القطع الصغيرة المربعة الموزعة عشوائيا فوق الطبقة الطينية. وقد أثبتت النتائج أن مقاومة التربة تزداد بازدياد كمية التسليح .