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Experimental Study of the Bearing Capacity of Stiff Clay Overlying Sand with and without Geotextile Inclusion

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ABSTRACT The objective of this study is to investigate the potential benefits of using reinforcement inclusion to improve the bearing capacity of stiff clay over very loose to medium dense sand. Model load tests were performed on two layered systems, namely stiff clay over very loose, loose, medium dense sand with and without geotextile inclusion between the two layers and on stiff clay only. The load-settlement curves were plotted from the experimental test results, and the ultimate bearing capacity was obtained using Log – Log (L-L), Tangent (TIM), 0.1B and Hyperbolic (HYP) methods. Theoretical approaches were used to compute the ultimate bearing capacities of the tests without and with reinforcement. The test results have shown an increase in the ultimate bearing capacities due to increase in the relative densities of the bottom sand layer. The bearing capacity increased significantly with the inclusion of geotextile layer. The bearing capacity ratio (BCR) for the case of very loose sand as bottom layer was the highest compared to loose and medium dense cases. Load - settlement curve of the pure clay test plots above or is identical to the load - settlement curve of stiff clay overlying medium dense sand with geotextile layer. The maximum benefit for the geotextile inclusion was gained at large strain when the sand was very loose. The analytical methods were generally in good agreement with the experimental model test results obtained by the 0.1B method.

Keywords: Circular footing, Clay, Geotextile, Sand.

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1. INTRODUCTION

Most of Khartoum city is covered with a blanket of stiff to very stiff potentially expansive silty clay soil having variable thickness and properties. This layer is underlain by alluvial poorly graded silty sand which ranges in density from very loose to dense. In many cases, shallow foundations are place on the very stiff clay soil which is often underlain by relatively weak sand resulting in reduction to the bearing capacity of the very stiff clay and in settlement problems. Conventional treatment methods were to place selected compacted fill below the foundation increase the dimensions of the footing, or a combination of both methods. An alternative and probably economical solution is the use of geosynthetics to reinforce the foundation soil and improve its bearing capacity.

For the last five decades, reinforced soil foundation (RSF) has been employed in engineering practice as a common technique to increase the ultimate bearing capacity of soils and decrease the settlement of footings. Soil alone is just able to carry compressive and shear stresses, however, through the use of a reinforcing elements, soil structures can be formed to resist tensile stresses. Reinforcing of soil can be achieved by embedding various types of materials such as metallic strips, geotextiles, geogrids and even fibers. The main concept of reinforced soil is based on the existence of tensile strength of the reinforcement elements and soil-reinforcement interaction due to the frictional, interlocking and adhesion properties. There have been some studies of shallow foundations on reinforced soil systems, most of them concentrating on sandy soil (Binquit and Lee, 1975a; Akinmusuru and Akinbolade, 1981; Fragaszy and Lawton, 1985; Guido et al., 1985; Guido et al., 1986; Huang and Tatsuoka, 1990; Khing et al.,1993; Das and Omer ,1994; Yetimoglu et al.,1994; Gabr et al., 1998). Analytical models have been proposed for calculation of the bearing capacity of a compact sand or gravel layer on soft clay (Chen and Davidson, 1973; Hanna and Meyerhof, 1980; Love et al., 1987; Florkiewicz, 1989; Michalowski and Shi, 1995; Lyons and Fannin, 2006; Sharma et al., 2009). Generally, several studies attempted to evaluate the benefits of using RSFs through bearing capacity ratio (BCR). Their efforts were aimed at investigating the parameters and variables that would contribute to the BCR value, such as the type of reinforcing materials, number of reinforcement layers, width of foundation etc.

A limited number of experimental studies are available at the present time relating the bearing capacity of shallow foundations on reinforced granular material of limited thickness overlying clay (S.K.Dash et al., 2003; A. Demir et al., 2011). However, less attention has been given to stiff clay soil overlying sand with geosynthetic inclusion. This investigation focuses on the latter situation using experimental laboratory model tests to estimate the improvements in bearing capacity for three cases: stiff clay overlying sand (very loose, loose and medium dense) with and without reinforcement.

2. THE EXPERIMENTAL WORK

a. General

This experimental work was aimed at investigating the bearing capacity of a circular footing resting on stiff clay overlying sand, with and without geotextile inclusion between the two layers, for three major cases: stiff clay on very loose sand, stiff clay on loose sand and stiff clay on medium dense sand.

B. Materials used

Two types of soils were used as foundation soil in this experimental study namely sand and silty clay. Relevant properties were obtained by performing several laboratory tests in accordance with British Standard B.S.1377-1990. Non-woven polyester filament geotextile was used as reinforcement element in the present study.

1. The Sand

The sand was obtained from the flood plains of the Blue Nile in Tuti Island in Khartoum. It is medium to fine-grained air-dried sand that is classified as poorly graded sand with silt (SP - SM). The friction angle of the very loose, loose, and medium dense sand as determined from shear box tests was found to be 30^{o}, 33.40^{o} and 40.36^{o}, respectively. Maximum and minimum densities obtained by ASTM D2049-69 test method are 1.74g/cm3 and 1.38 g/cm3, respectively.

2. The Clay

The clay used in this investigation was obtained from University of Khartoum Brick Factory site in Soba south of Khartoum. Summary of the results of the laboratory tests performed on the clay sample is given in Table (1).

3. The Geotextile

A commercially available non-woven, continuous filament, needle-punched geotextile was used for reinforcement. It has a white color, maximum tensile strength of 32.05kN/m and was made from polyester.

TABLE 1: THE PROPERTIES OF CALY

Properties	Clay
Finer than 0.075 mm (%)	≈ 100
Range of Particles(mm)	Less than 0.125
Specific Gravity (units)	2.80
Liquid Limit (%)	51
Plastic Limit (%)	29
Plasticity Index (%)	22
Maximum Dry Density (g/cm^3)	1.43
Optimum Moisture Content (%)	24
Angel of Internal Friction (°)	20
Cohesion (KN/m^2)	110
Clay Content (%)	26
Silt Content (%)	73
Sand (%)	0
USCS Classification	CH

C. Tests program

In the present study seven plate load tests were conducted for the following cases: Case (I) stiff clay overlying very loose sand, Case (II) stiff clay overlying very loose sand with geotextile inclusion, Case (III) stiff clay overlying loose sand, Case (IV) stiff clay overlying loose sand with geotextile inclusion, Case (V) stiff clay overlying medium dense sand, Case (VI) stiff clay overlying medium dense sand with geotextile inclusion, and Case (VII) compacted homogenous silt clay soil. One circular steel plate size (D = 200 mm) was used as footing. The physical model for a shallow foundation resting on two-layered soil system was prepared. A circular tank, divided into two parts, was manufactured such that the two parts could be assembled to create a single tank (Fig 1). The sand was pulverized for Cases (I) and (II), compacted for Cases (III) to (VI) to the target densities in the lower tank to the top level to form the sand layer whereas the clay was prepared, in the upper part, placed on a steel plate, to the required density on its optimum moisture content. The upper part was then lifted carefully and placed on the lower one keeping good contact between the two layers. The above step was done for all the test cases without geotextile. As for the test cases with reinforcement, before the upper part was fixed, the geotextile was placed first on the top of the sand and then the upper part of the model was placed. Details of the test setup are given below:

1. The Model

The physical model consists of a tank and a reaction system. A 600mm diameter circular steel tank, formed of two parts, with plate thickness 2.8 mm was fabricated. It was strengthened with steel ring bars every 150 mm in height and stiffened at the bottom with platform to avoid any lateral yielding during sand compaction or when applying load at the model footings. The upper part and lower part heights are 150 mm and 600 mm, respectively. The two parts were fastened with steel bolts around the tank perimeter. The upper part of the model tank was used for preparation of the clay to the required placements conditions, i.e. density and moisture content, whereas the lower part was used for placing the sand. The reaction system is shown in Fig. 1.

A circular steel plate of 200mm diameter and 15mm thick was selected to represent the footing in this experimental study based on the boundary condition of the test. The plate size was chosen in such a way that loads effects will not reach perimeter and bottom of the model tank.

Fig1: The steel tank and reaction frame

3. METHODOLOGY

The foundation soils were prepared in the model tank; the stiff clay in the upper part, and the sand in the lower part. The same sand was used in the bottom layer prepared at three different relative densities (very loose, loose and medium dense). Each test program was executed in two stages; the first stage was the sample preparation and the second one was the assembly of devices whereas the third was application of the load to failure. Test data was collected and the pressure–settlement curves were plotted and bearing capacities were computed. Each Laboratory model test took at least three days for preparation.

The sand with predetermined density and weight according to relative densities tests results was prepared in the bottom tank. It was poured for very loose sand from a suitable height above the bottom part of model in order to keep a constant density in the whole depth of tank. For loose and medium dense conditions, the sand was poured and compacted by manual compacter in 3 layers to the required densities which give relative density (Dr) equals 25% and 45% for loose and medium dense conditions, respectively. For tests with the geotextile inclusion, the filament geotextile layer in 600 mm diameter was prepared and placed at the top of the sand.

On the other hand, the air–dried processed silty clay soil was crushed and pulverized by using sieves no. 4 and no. 9 for more homogeneity. Five samples were used to calculate the natural moisture content. Using the results of the Standard Proctor test for the silty clay soil (maximum dry density & optimum moisture content) and the upper part of model tank dimensions, the amount of water and natural soil needed to achieve the required density were computed. The amount of clay soil needed to fill the upper plate was known and it was then divided into five parts, each part was mixed with the required water content, kept in plastic bag for 24 hours for proper moisture distribution. Specimens were taken for moisture content determination. The clay soil was compacted in the upper part of the tank in layers to the required density. This was controlled by compacting known weight of the moist clay to precomputed thickness [17]. Three steel pipe samplers were used to obtain samples for the unconfined compression test by pushing them carefully into the compacted stiff clay. The upper part of the tank, containing stiff clay, was carefully lifted and located on its position above the sand and fixed by bolts [17]. The steel plate was located on the stiff clay (at the center). The load cell was put on the center of the plate and the hydraulic jack was placed on it and in contact with the reaction beam of the loading frame. The load cell and displacement transducers were fixed and connected to the data logger. The load was applied by a hydraulic jack continuously at constant strain rate, as possibly as can be, until the failure took place. The unconfined compression test specimens were then extracted.

4. RESULTS AND DISCUSSION

A. The Ultimate Bearing Capacity

Fig 2 displays the load-settlement curves for all the tests conducted in this study. It is apparent from the curves that no distinctive failure point has been observed. For small strain (0.0 ~ 3%) the curves are very close to each other, i.e. the load is about the same for the same settlement value; however, the curves started to depart as the settlement increases until failure was attained. There is noticeable increase in the load carrying capacities of the tests due to increase in the relative densities of the bottom sand layer from very loose to loose to medium dense. Significant improvements were observed in the load bearing capacities of reinforced tests compared with unreinforced tests at large settlements. Comparing the load- settlement curves the results show that for the same settlement the ultimate bearing capacity increases due to the inclusion of geotextile. The load - settlements curve of the pure clay test gave stress - strain relationship about similar to those of stiff clay overlying dense sand with geotextile inclusion (test vi).

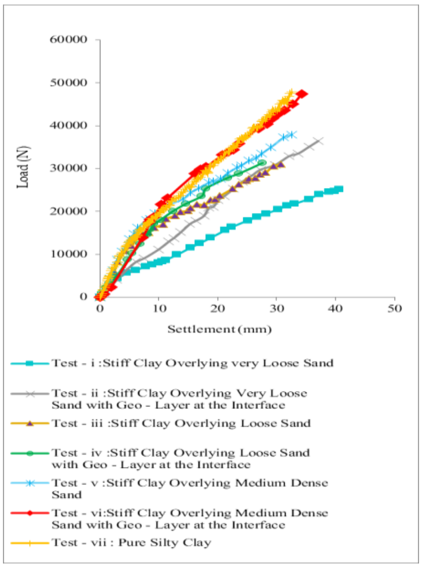

Fig 2: Load – Settlement Curves for all Tests

The theoretical ultimate bearing capacity (UBC) of the unreinforced tests was computed by using Meyerhof and Hanna (1978) equation given below for two - layered soil system for the general condition of strong soil over weak soil. According to the test conditions the top layer is described as strong and the bottom is described as weak therefore, the ultimate capacity of a plate on strong layer (top) overlying weak layer (bottom) can be computed by using Equations (1) and (2) for the bottom and top layers.

$$q_b = C_b N_{C_b} \lambda_{C_{Sb}} + \gamma_t \left(D_f + H\right) N_{q_b} \lambda_{q_{Sb}} + \frac{1}{2} \gamma_b B N_{\gamma_b} \lambda_{\gamma_{Sb}} \tag{1}$$

$$q_t = C_t N_{C_t} \lambda_{CS_t} + \gamma_t D_f N_{q_t} \lambda_{qs_t} + \frac{1}{2} \gamma_t B N_{\gamma_t} \lambda_{\gamma S_t} \tag{2}$$

Where,

$$C_b = c_b H \tag{3}$$

C_b is the adhesive force along two sides, c_b is the unit adhesion of soil along two sides, H is the thickness of the upper layer, D_f is the depth of the footing in the soil, B is the width of foundation, $\lambda_{C_{Sb}}, \lambda_{q_{Sb}}, \lambda_{\gamma_{Sb}}$ are the shape factors for the bottom soil layer, $N_{c_b}, N_{q_b}, N_{\gamma_b}$ are the bearing capacity factors, γ_b is the unite weight of the bottom soil layer, $C_t, N_{C_t}, N_{q_t}, N_{\gamma_t}, \lambda_{CS_t}, \lambda_{qs_t}, \lambda_{\gamma S_t}, \gamma_t$ are the parameter of the top layer.

Where, the ultimate bearing capacity for unreinforced two - layered system (top strong and bottom weak) was obtained by using the equation below.

$$q_u = q_b + \left[1 + \frac{B}{L}\right] \frac{2C_a H}{B} \lambda_a + \left[1 + \frac{B}{L}\right] \gamma_t H^2 \left[1 + 2\frac{D_f}{H}\right] \frac{K_S \tan \emptyset_t}{B} \lambda_S - \gamma_t H \leq q_t \tag{4}$$

L is the width of foundation, K_S is the punching shear coefficient, $\emptyset_t$ is the Angle of shearing resistance of the top (stronger) layer.

Whereas, the ultimate bearing capacity for two-layered system with geotextile inclusion is calculated using Wayne et al, (1998) equation.

$$q_{u(R)} = q_b + \frac{4 c_a S_a d}{B} + 2\gamma_t d^2 \left(1 + \frac{2D_f}{d}\right) \frac{K_s S_s \tan \emptyset_t}{B} + \frac{4 \sum_{i=1}^{N} T_i S_T \tan \delta}{B} - \gamma_t d \tag{5}$$

(S_a, S_s, S_T) are the Shape factors for punching shear resistance for circular footings, d is the depth of reinforcement, N is the number of reinforcement, δ is equal to friction angle of the reinforced layer $\emptyset_t$, T_i is the Tensile force of reinforcement.

Note: The definition of ultimate bearing capacity in the analytical solution is at 10% settlement values (Ahmet et al, 2011; Imran Akond, 2012), because the magnitude of the improvement in the load bearing capacity is observed at large settlements for reinforced tests and relatively large strain is required to mobilize the strength of the reinforcement layer.

B. Improvements in Ultimate Bearing Capacity

The estimation of the ultimate bearing capacity from the load – settlement curves (Fig 2) was done using the Log-Log method (L-L), Tangent Intersection method (T-I), 0.1B method, Hyperbolic method (HYP) (Kolay et al., 2013).

The increase or improvement in bearing capacity due to the introduction of geotextile is explained by the bearing capacity ratio (BCR) which is the ratio of bearing capacity of the reinforced case to that of the unreinforced case for the same relative density of the sand layer. Here, the bearing capacity was computed from the test results using the given methods and consequently the BCR was obtained for each method (Table 2).

TABLE 2: Comparisons between Unreinforced and Reinforced Tests

Test designation	**Ultimate bearing capacity (kN/m²) (no geotextile inclusion)**				**Ultimate bearing capacity (kN/m²) (with geotextile inclusion)**				**Ultimate Bearing capacity Ratio (BCR)**			
	L-L	**T-I**	**0.1BM**	**HYP**	**L-L**	**T-I**	**0.1BM**	**HYP**	**L-L**	**T-I**	**0.1BM**	**HYP**
Case I	200	255	471	520	300	382	732.5	882	1.50	1.50	1.56	1.70
Case II	400	439	732.5	1400	455	510	860	1722.2	1.13	1.16	1.17	1.23
Case III	580	605	880.6	1800	650	688.8	1019	2143	1.12	1.14	1.16	1.19

The initial portions of load-settlement curves are closely spaced at small settlements, however the curves depart when the settlement exceeds about 2.0mm. Minor contribution of geotextile for small settlements is noticed. According to the Tangent Intersection method the ultimate bearing capacity for test (i) was 255 KN/m² at settlement equals to (≈ 0.025 B), whereas the ultimate bearing capacity for test (ii) was 382 KN/m² at settlement equals to (≈ 0.04 B), and that was because the frictional resistance forces between soils and reinforced layer mobilize at large settlement in the tests with geotextile inclusion. It is observed that BCR for Case (I) is the highest compared with the other two cases. The reason for that is the contribution of tensile forces in the soils – geotextile system when very loose sand was used as a bottom layer (case I) more than loose sand and medium dense sand (case; II and III). The large settlement causes large strain for reinforcement layer and enhance the mobilization of frictional forces. It is also noticed that there is slight improvement in (BCR) from case (II) to case (III), despite the increase in the ultimate bearing capacities between the unreinforced and reinforced tests of these cases due to high strength of the bottom sand layer (Figure 3). On the other hand, the stiffness of the bottom layer reduces settlement and therefore minimizes the contribution of the reinforcement. Significant improvement has been observed in the load bearing capacity of the tested plates with increase in relative density, for the unreinforced tests. From Table 2 when the 0.1B method is used, the ultimate bearing capacity for the unreinforced tests increased by 56 % with increase in relative density of the bottom layer from very loose to loose (test (i) and test (iii)) and by 20 % from loose to medium dense (test (iii) and test (v)), whereas the ultimate bearing capacity for reinforced tests increased by 17 % from very loose to loose (test (ii) and test (iv)) and by 19 % from loose to medium dense (test (iv) and test (vi)). It is apparent that the percentage increase in bearing capacity for the reinforced tests, i.e. from very loose sand as a bottom layer to loose sand is less than the percentage increase in B.C for unreinforced tests. The test results show very good contribution of the geotextile to bearing capacity for the case of very loose sand. Also, the percentages increase in B.C for the unreinforced tests for loose sand and medium dense reinforced is 20 % approximately equals to that of the reinforced tests (~18%). The reason for that was the minor contribution of geotextile for medium dense case. The bearing capacity of “the full depth” silty clay, i.e. silty clay only, is considered as reference to compare with the bearing capacities of all other unreinforced and reinforced cases. From the experimental tests the load – settlement curve of the silty clay only plots above those of the other tests and is almost identical to the load – settlement curve of medium dense sand with geotextile layer (test vi), (Figure 2).

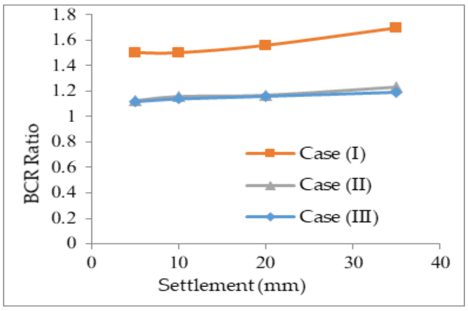

Fig 3: Bearing Capacity Ratios

C. *Comparison Between the Analytical Method and Experimental Results*

Table 3 presents the bearing capacity values obtained using the analytical method, i.e. Meyerhof and Hanna (1978) for the unreinforced tests and Wayne et al, (1998) for the reinforced tests. The values are compared with those which were determined from the experimental tests, 0.1B method. It is apparent that the ultimate bearing capacities for unreinforced and reinforced tests by using the analytical solution have given results close to those of the experimental tests when obtained by 0.1B method.

According to the 0.1B method there exists no constant trend for the ultimate bearing capacity values when compared with the analytical ones for all the tests (Table 3). For example, the percentage difference between the ultimate bearing capacities obtained by 0.1B method and analytical methods were - 25 %, 9 %, 1.6 %, 12.8 %, - 8.13%, 1.78 %, and - 33 % for the tests from (i to vii). However, the observed results computed by both methods are relatively close. Also the bearing capacity ratio for case (I - II - III) of the tests is 1.56, 1.17 and 1.16 for 0.1B method and 1.067, 1.057 and 1.044 for analytical methods, therefore there is a good approximation between the two approaches and each one achieved improvement in the ultimate bearing capacity.

TABLE 3: Ultimate Bearing Capacity & BCR for Experimental and Analytical Test

Test Designation	Ultimate bearing capacity (kN/m^2)		Error (%)	Bearing capacity Ratio(BCR)	
	0.1BM	**Anal**	**0.1BM &Anal**	**0.1BM**	**Anal**
i	471	630.09	- 25.25	1.00	1.00
ii	732.5	672.11	9	1.56	1.067
iii	732.5	721.195	1.60	1.00	1.00
vi	860	762.59	12.80	1.17	1.057
v	880.6	958.5	- 8.13	1.00	1.00
vi	1019	1001.01	1.78	1.16	1.044
vii	1020	1530	-33		

5. CONCLUSION

Model load tests were carried out on two layered soil system: stiff clay overlying very loose, loose and medium dense sand with and without geotextile inclusion layer and one test on homogenous pure clay. Based on the model test results, the following conclusions are drawn.

1- The inclusion of reinforcement generally increased the ultimate bearing capacity of soil and reduced the footing settlement.

2- The load carrying capacity increased with increase in the relative densities of the bottom sand layer from very loose to loose sand and to medium dense for constant H/D (the thickness of top layer to the diameter of footing.

3- For the same amount of settlement, the ultimate bearing capacity increased with the inclusion of geotextile.

4- It was observed that BCR for the very loose sand as bottom layer (case I) represented the highest values compared with the other two cases, whereas there was slight improvement in BCR when loose sand was used as a bottom layer for case (II) and as medium dense for case(III)

5- The important notice is that the load–settlement curve of pure clay was higher or almost identical to the load–settlement curve of stiff clay overlying medium dense sand with geotextile layer.

6- The ultimate bearing capacity for homogenous clay layer computed from analytical solution by Meyerhof and Hanna, (1978) represents the highest value for Log-Log, Tangent, and 0.1B methods, except for the HYP method which gave lower value of B.C.

7- The computed bearing capacities of unreinforced and reinforced soil foundation using the methods of Meyerhof and Hanna, (1978) and Wayne et al, (1998) are in good agreement with the laboratory test results obtained by 0.1BM, specifically.

8- The findings from the present study will provide general guidelines for the developments of a rational design methodology. However, the extrapolation of the results from these model tests to field cases can be done making use of a suitable scaling law with careful consideration of different references

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