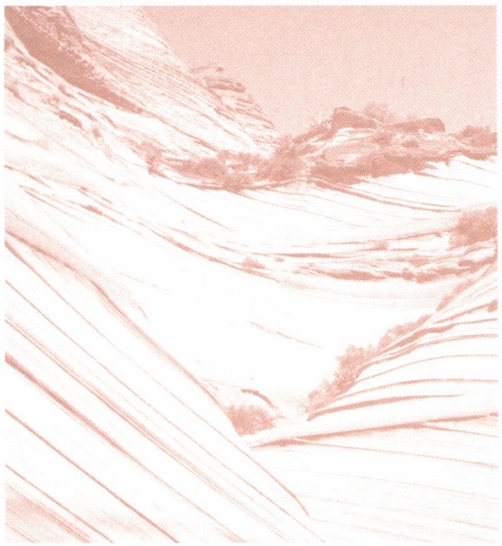


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Field Test of Circular Footings on Reinforced Granular Fill Layer Overlying a Clay Bed

ABSTRACT: The ultimate bearing capacity and settlement of a circular shallow rigid plate on compacted granular fill layer with and without geogrid reinforcement overlying on natural clay deposit exhibiting low bearing capacity and large settlement have been investigated. A total of 15 field tests were carried out using a circular model rigid plate with a diameter of 0.90 m. This study has been initially directed to evaluate the beneficial effects of the compacted granular fill layer on natural clay deposit for the shallow rigid plate performance. Then, the reinforcing effect of the top granular fill layer with horizontal layers of welded geogrid reinforcement on the bearing capacity and settlement has been studied. Parameters of the testing program include granular fill thickness, depth of first reinforcement, vertical spacing of reinforcement layers, and number of reinforcement layers. Bearing capacity ratio (BCR) and percentage reduction in settlement (PRS) were defined to evaluate improvement performance. Based on the test results, the effect of the granular fill and welded geogrid reinforcement on the bearing capacity and settlement are discussed. The results indicate that the use of granular fill layers over natural clay soils has considerable effects on the bearing capacity and settlement characteristics. The construction of granular fill layer with welded geogrid reinforcement over clay deposit helps in redistributing the applied load to a wider area. It has been observed that the use of welded geogrid reinforcement in granular fill layer provides additional improvement of bearing capacity and provides reduction in settlement of the rigid plate up to 80 and 60 %, respectively.

KEYWORDS: large scale, field tests, clay, granular fill layer, welded geogrid

Introduction

Lack of proper construction sites in city centers has led to the increased use of problematic areas where the bearing capacity of the underlying deposits is low. The construction of reinforced soil footing to support a shallow footing has considerable potential as a cost-effective alternative to conventional methods of support. Welded geogrid is one of the soil improvement materials and has been used efficiently to reinforce soil structures. Geogrids generally mobilise a high soil-reinforcement bond stress and have a high tensile stiffness (Laman and Yildiz 2003). Binquet and Lee (1975a, 1975b) were among the first to report a systematic study on bearing capacity of reinforced soil beds. Since then, a number of analytical and experimental studies on the subject have been conducted by several researchers (Akinmusuru and Akinbolade 1981; Guido et al. 1985; Huang and Tatsuaoka 1990; Abled-Baki et al. 1993; Khing et al. 1993; Omar et al. 1993; Yetimoglu et al. 1994; Adams and Collin 1997; Kumar and Saran 2003; Laman and Yildiz 2003; Chen 2007; Kumar et al. 2007).

However, there are limited studies that focused directly on the use of large-scaled model rigid plates in the literature, some case studies using large-scale rigid plates are Adams and Collin (1997), Consoli et al. (1998), Gabr and Hart (2000), Fonseca (2001), Hirofumi et al. (2003), and Chen (2007). Adams and Collins (1997) studied the effect of geosynthetic reinforcement on the ultimate bearing capacity of reinforced sands using square plates ranging in size from 0.30 to 0.91 m. The test results indicate that the use of geosynthetic reinforced soil foundations may increase the ultimate bearing capacity of shallow spread rigid plates by a factor of 2.5. Wayne et al. (1998) reviewed a complete overview of research work to provide a better understanding of the benefits that can be derived through the use of the ground modification technique. Some emphasis was placed on the examination of the factors which control the design of a geosynthetic reinforced foundation. They suggested design guidance by providing typical design parameters for geogrids used in reinforced foundations. Gabr and Hart (2000) reported the results of nine plate load tests on geogrid reinforced sand in terms of elastic modulus. It was reported that the elastic modulus decreased with increasing depth of the top geogrid layer. Hirofumi et al. (2003) reported that the plate loading test with different-sized loading plates is effective as a method for studying design constants in the design of spread footing by taking scale effects into consideration. Chen (2007) investigated the potential benefits of using the reinforcement to improve the bearing capacity and reduce the settlement of shallow footings by conducting a total of 117 tests. The investigated parameters include top layer spacing (u), number of reinforcement layers (N), vertical spacing between reinforcement layers (h), tensile modulus and type of reinforcement, embedment of the footing (Df), shape of footing, and type of soil. The test results showed that the inclusion of reinforcement can significantly improve the soil's bearing capacity and reduce the footing

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settlement. Insignificant strain measured in the geogrid beyond its effective length of 4.0*B*–6.0*B* indicated that the geogrid past this length provides negligible reinforcement effect. The optimum depth to the first reinforcement layer was estimated to be at about 0.33*B* below the footing for all soil types tested in this study. The performance of reinforced soil improves by increasing the reinforcement tensile modulus. The scale effect on the results of model footing tests was studied using the FEA program ABAQUS. Finite element analysis results indicate that the scale effect of reinforced soil foundation is mainly related to the reinforced ratio (*Rr*) of the reinforced zone. The failure mechanisms of reinforced soil foundation were proposed based on the literature review and experimental test results. Stability analysis including the effect of reinforcement has been conducted based on these proposed failure mechanisms.

This study focused on the ultimate bearing capacity and settlement of circular rigid plate resting on subsoil consisting of compacted granular fill layer with and without welded geogrid reinforcement overlying on natural clay deposit exhibiting low bearing capacity and large settlement. As known, the physical modeling can be carried out through either a full scale model or a small scale model in geotechnical engineering. Small scale laboratory tests are easy to perform and cost less when compared to large and full scale field tests. Small scale models are widely applied in geotechnical practice since the modeler has full control in using those to analyze geotechnical problems. Full scale modeling is generally performed with the real site conditions such as ground condition, loads, and stress levels. Hence, the results obtained from this modeling are more accurate than the other type of modeling. Despite their operational and financial disadvantages, large and full scale field tests give more reasonable results in simulating soil behavior and modeling of a geotechnical structure. With large scale field tests in this study, we tried to overcome the shortcomings of small scale laboratory model tests and to more accurately model the full scale behavior of reinforced soil footing. Soft clay deposits have been found in most of the populated areas in Turkey and around Cukurova, Adana. As more and more construction is concentrating on densely populated urban areas, there is an increasing need to construct on these soft subsoils. This will create new challenges for geotechnical design. Although there has been considerable development in both understanding the behavior of soft soils and in design methods, this knowledge is not routinely used in geotechnical design. Recently new techniques in soft soil construction have been adopted, such as soil reinforcement. Designs involving soil reinforcement techniques will require not only good knowledge about the behavior of soft soils but also on the granular fill material with and without reinforcement. Methodologies need to be developed on how to incorporate that knowledge into design. This study aims to address these issues. According to the author's knowledge, there are very few studies in literature that directly investigate the effect of the geogrid reinforced granular fill layer on the bearing capacity and settlement behavior of a large scaled rigid plate on natural clay deposits. Also it is thought that this study is considered to have provided a useful basis for further research leading to an increased understanding of the application of soil reinforcement to bearing capacity and settlement problems.

Field Tests

It is known from the literature that most of the experimental stu ies on reinforced soils were conducted using small scale labor tory tests. Due to scale effect, it is sometimes difficult accurately model the full scale behavior of reinforced soil usi small scale laboratory tests (Abu-Farsakh et al. 2008). For this re son, large scale field tests were conducted on natural clay depos and compacted granular fill layers with and without welded ge grid layers overlying the natural clay deposit to evaluate the pe formance of clay deposits.

Site Characterization

A total of 15 large scale tests have been conducted in the Ada Metropolitan Municipality's (AMM) Water Treatment Facil Center (WTFC) located in the western part of Adana, Turkey. T soil conditions at the experimental test site (WTFC) were dete mined from a geotechnical site investigation comprised of bo field and laboratory tests. Two test pit excavations (TP1 and TP and four borehole drillings (BH1, BH2, BH3, and BH4) were pe formed in the WTFC test area. The test area has a dimension o 30 m (length) by 11.60 m (width). The test pits were excavated t derive visual inspection and to obtain disturbed and undisturbe soil samples for laboratory experiments. The dimensions of th excavation were 2.5, 4.0, and 2.0 m as depth, length, and widt respectively. The boreholes were drilled with diameters of 0.10 m and depths of 13 m. The borehole drilled on the south side wa 20 m. Ground water level was observed at 2.20 m from boreho drillings. The degree of saturation of the clay layer where the te was conducted was about 80 %. The locations of test pits an boreholes are shown in the layout plan in Fig. 1. Three subso layers were clearly identified by visual inspection and by unifie soil classification system. The top soil, which is about 0.80 m thick, contained highly organic partly decomposed vegetable ma ter. Therefore the top soil was removed from the test area befor the experiments. The intermediate layer between the depths o 0.80 and 7.0 m exhibits silty clay stratum with high plasticit (CH), with plastic and liquid limit values ranging from 20 to 22 % and 52 to 55 %, respectively. A silty clay layer with intrusion o sand (CL), with plastic and liquid limit values ranging from 20 t 22 % and 38 to 40 %, respectively, was observed in the botto layer up to 10.0 m depth.

Standard penetration tests (SPT) were carried out during dri ling of each borehole and distribution of SPT values with depth i shown in Fig. 2. Based on SPT test results the soil tested was cla sified as medium stiff clay.

Conventional laboratory tests were performed at the Geotech nical Laboratory of Civil Engineering Dept. at Cukurova Univ Adana, Turkey. The consolidation, laboratory vane, triaxial, an unconfined compression tests were conducted on undisturbed soi samples taken from the field. The clay content of the soil layer varies in a range of 60 to 70 %. The upper homogeneous laye where all loading tests were carried out is classified as high pla ticity clay (CH) according to the Unified Soil Classification Sys tem (USCS). The particle size distribution of the clay soil i presented in Fig. 3. The water content of the stratified soil layer

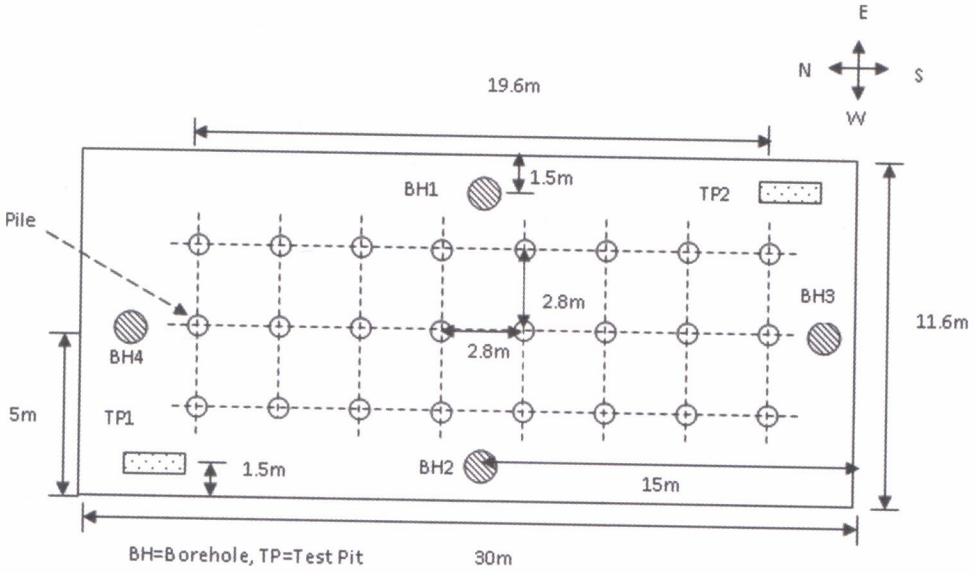


FIG. 1—*Plan view showing piles, borings, and test pits.*

varies between 20 and 25 % depending on depth, and is almost the same as or greater than the plastic limit. The specific gravity values (SG) of the clay soil vary from 2.60 to 2.65 along the depths. The values of undrained shear strengths c_u were determined by unconfined compression tests and obtained between 60 and 80 kN/m^2. The average value of undrained shear strength from unconsolidated undrained triaxial tests was obtained as 80 kN/m^2. The soil layers classified as normally consolidated from odometer tests.

Granular Fill

The granular fill material used in the model tests was obtained from the Kabasakal region situated northwest of Adana, Turkey. Some conventional tests (sieve analysis, moisture content, unit weight, direct shear, and proctor tests) were conducted on this material. Granular soil was prepared at an optimum moisture content of 7 % and maximum dry unit weight of 21.7 kN/m^3 obtained from the standard proctor test. The values of the internal friction angle and the cohesion of granular fill were obtained as 43° and 15 kN/m^2, respectively, from direct shear tests. The square shaped direct shear box has a width of 0.06 m. Specific gravity of the granular soil was obtained as 2.64. From the sieve analysis, granular soil was classified as well graded gravel-silty gravel, GW-GM according to the USCS. The particle size distribution of the granular fill material and the results from standard proctor test are shown in Fig. 4.

Geogrids

A white colored, Secugrid Q type geogrid with maximum tensile strength of 60 kN/m was used as reinforcing material in the field tests. The square shaped welded geogrid which has a length of 2.8 m ($\cong 3D$) was used in the field tests. The physical and

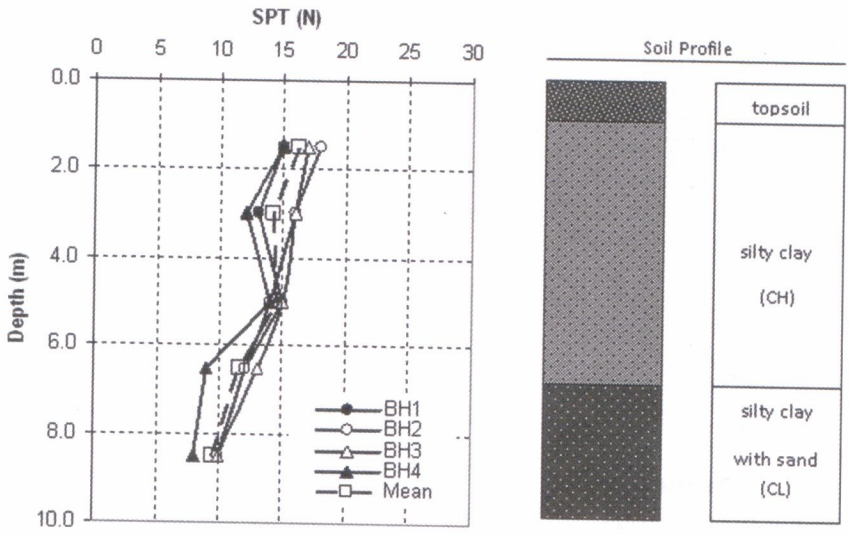


FIG. 2—*SPT(N) values measured from boreholes.*

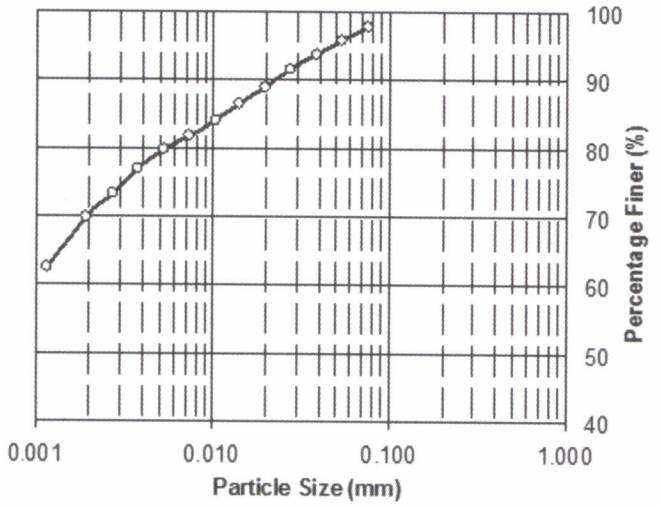


FIG. 3—*Particle size distribution of local clay soil.*

mechanical properties of the welded geogrids as listed by the manufacturer are given in Table 1.

TABLE 1—*Mechanical properties of geogrid.*

Parameter	Value
Structure	biaxial
Aperture shape	squared
Aperture size	30 mm × 30 mm
Mass per unit area	360 g/m^2
Raw material	polypropylene
Elongation at nominal strength	8 %
Tensile strength at 2 % elongation	22/22 (md/cmd) (kN/m
Tensile strength at 5 % elongation	48/48 (md/cmd) (kN/m

Experimental Setup and Test Program

After describing soil properties in the WTFC test area, a total of 24 piles were constructed. The locations of the piles in the test area are shown earlier in Fig. 1. Topsoil was removed before tests. Then, reaction piles were connected with a steel beam. The top surface of the test area was leveled and the rigid plate was placed on a predefined alignment such that the loads from the hydraulic jack and loading frame would be transferred concentrically to the rigid plate. A hydraulic jack against the steel beam provided downward load. The hydraulic jack and two linear variable displacement transducers (LVDT) were connected to a data logger unit and the data logger unit was connected to a computer. In each test a series of monotonically increasing static loads were applied to the reaction frame seate directly on the reaction piles. The load was applied using a hy draulic jack and maintained manually with a hand pump unti the ultimate vertical deformation was obtained. The load an the corresponding footing settlement were measured by a cali brated pressure gauge and two LVDT's, respectively. The tes ing procedure was performed according to the ASTM D1196-9 standard (ASTM D1196-93 1997), where the load increment were applied and maintained until the rate of settlement wa less than 0.03 mm/min over 3 consecutive minutes.

For each test section, the granular fill material was placed an compacted in layers. The compacted granular fill layer has a mois ture content of 7 % and a unit weight of 20.2 kN/m^3. The thickne of each layer is 0.05 m, depending on reinforcement spacing. Th amount of granular fill material and water needed for each laye was first calculated. Then the granular fill material was compacte using a plate compactor to a predetermined height to achieve th desired densities. To maintain the same density in the granular fi layer, the convenient compactive effort was applied on each laye of granular fill. During the installation of the welded geogrid i the test, considerable care was taken to place the welded geogri as flat as possible. When the desired reinforcement depth wa

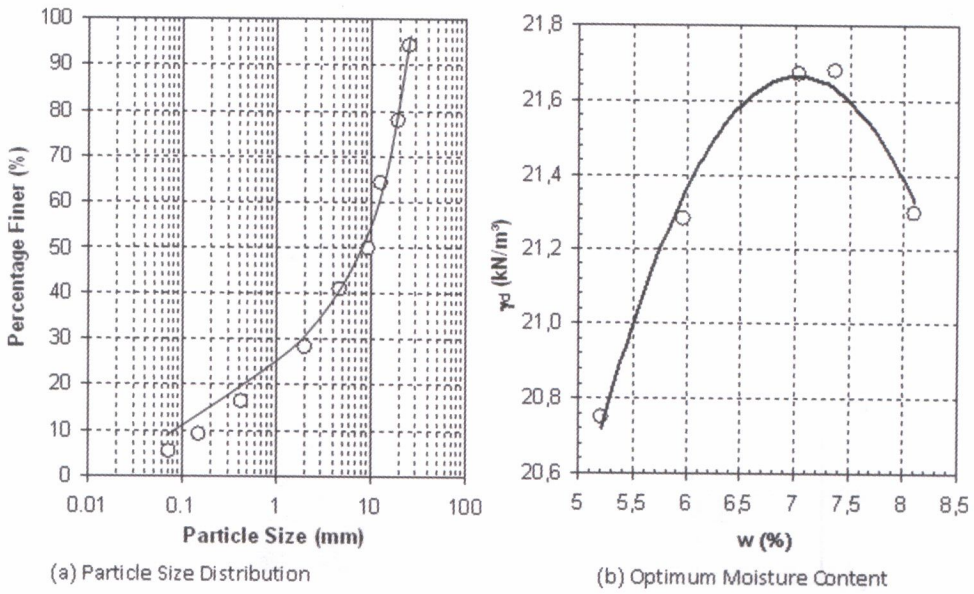


FIG. 4—*Typical soil characteristics of granular fill material.*

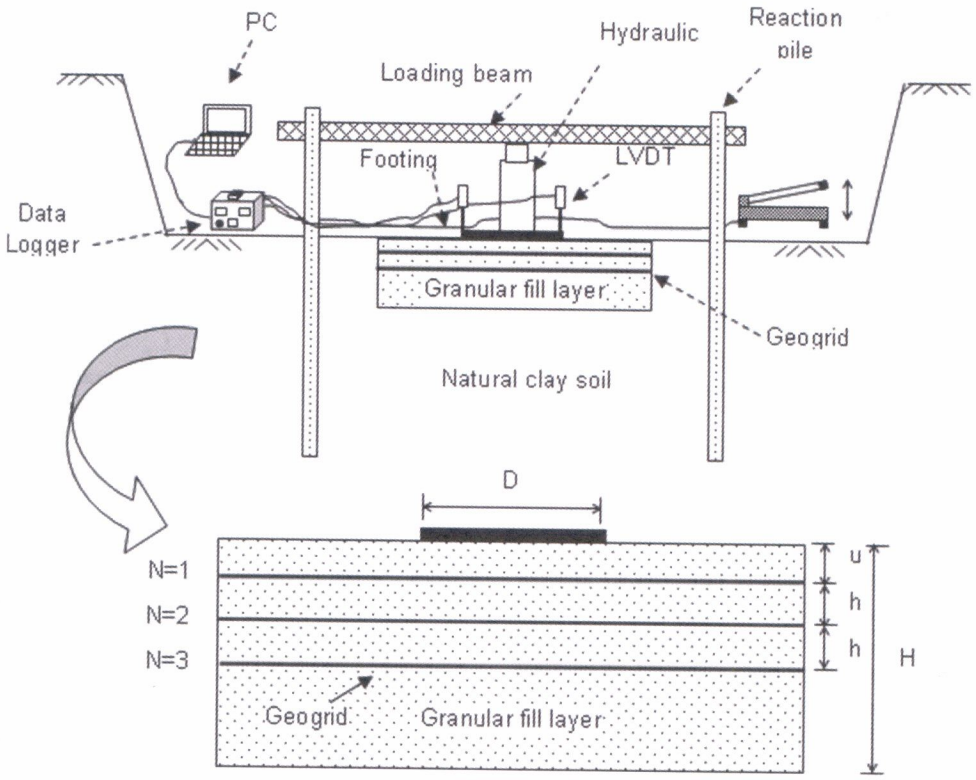


FIG. 5—*Schematic view of test setup, loading, reaction system, and typical layout of instrumentation.*

reached, the welded geogrid layer was placed and compaction was then continued until the desired granular fill height was reached.

The tests were carried out for a short time until the initial settlements were measured under undrained conditions. So long term settlement (consolidation) of clay was not measured in the testing program. After each test was completed, the granular fill layer was carefully excavated and the welded geogrid reinforcement visually inspected. Some tests were repeated twice to verify the repeatability and the consistency of the test data. The same pattern of load-settlement relationship with a difference of the ultimate load values less than 2.0 % were obtained. The difference was considered to be small and neglected. Detailed information of the testing procedure can be found by Laman et al. (2009) and Ornek (2009). The general layout of the test setup was given in Fig. 5.

The research was conducted in five series. Each series of test was conducted to study the effect of one parameter while the other variables were kept constant. Series I consisted of testing a 0.90 m circular rigid plate on the surface of natural clay deposit. Series II was the same as series I except that the rigid plate were founded on a granular fill layer settled above the natural clay deposit. Series III, IV, and V were identical to series II except that welded geogrid reinforcement was used in these tests. Granular fill layers were designed in different thicknesses of 0.30, 0.60, and 0.90 m. The main purpose of series III, IV, and V is to investigate the most suitable welded geogrid configuration parameters such as u, h, and N that maximizes both bearing capacity and minimizes settlement of the rigid plate. Series III was conducted to evaluate the effects of different u/D ratios on bearing capacity and settlement characteristics. The depth of the first reinforcement beneath the rigid plate was changed to 0.15, 0.30, 0.45, and 0.60 m, respectively. Series IV was performed to identify the effects of different h/D values on rigid plate performance. Vertical spacing between two reinforcement layers was changed to 0.15, 0.30, and 0.45 m, respectively. Series V was conducted to evaluate the effects of number of reinforcement layers N (was changed from 1 to 4) on bearing capacity and settlement behavior. Table 2 summarizes the testing program and variables H/D, u/D, h/D, and N.

TABLE 2—*Details of the field test program.*

Test Series	Constant Parameters	Variable Parameters	Number of Tests Done
I		Unreinforced	1
II	...	$H/D = 0.33, 0.67, 1.00$	3
III	$H/D = 0.67, N = 1$	$u/D = 0.17, 0.33, 0.50, 0.67$	4
IV	$H/D = 0.67, N = 2, u/D = 0.17$	$h/D = 0.17, 0.33, 0.50$	3
V	$H/D = 0.67, u/D = h/D = 0.17$	$N = 1, 2, 3, 4$	4

Test Results and Discussion

Interpretation of Test Results

In field tests the relation of bearing capacity to settlement (q-s or q-s/D) for the following various arrangements of granular fill and welded geogrid reinforcement is obtained and discussed. In the tests the plot of q against s/D takes almost a linear shape and a peak value (ultimate bearing capacity) is not obtained clearly. So, the ultimate bearing capacity was defined at a specific settlement ratio $s/D = 3\,\%$. Similarly, settlement reductions due to granular fill and welded geogrids are evaluated at a specific loading pressure of 300 kPa. Although this technique is arbitrary, it (1) is convenient, (2) may actually be close to the average soil strain at failure, and (3) forces a fixed value at q_{ult} (Cerato and Lutenegger 2006).

The contributions from granular fill and welded geogrid reinforcement on bearing capacity and settlement characteristics are presented by the terms of bearing capacity ratio (BCR) and percentage reduction in settlement (PRS).

The term "bearing capacity ratio" (BCR) is commonly used to express and compare the test data of the reinforced and unreinforced soils. The following well-established definition (Binquet and Lee 1975a) is used for BCR:

$$\mathrm{BCR} = q_R/q_0 \tag{1}$$

where q_R and q_0 are the bearing capacity for the reinforced and unreinforced soils, respectively. The parameters investigated, including the settlement of rigid plate s, are normalized by the diameter of the rigid plate D (Laman and Yildiz 2003).

For comparison of test results, settlement ratio (SR) and percentage reduction in settlement (PRS) were used as previously described by Mandal and Sah (1992) and others,

$$\mathrm{SR} = S_R/S_0, \quad \mathrm{PRS} = (S_0 - S_R)/S_0 = 1 - (S_R/S_0) \tag{2}$$

where S_0 and S_R are the settlements for the unreinforced and reinforced soils, respectively.

The general type of bearing capacity-settlement curves obtained in the tests and evaluation of the terms BCR and SR are shown in Fig. 6. As seen from the figure the BCR is calculated by dividing the ultimate bearing capacity for reinforced soil to the ultimate bearing capacity for unreinforced soil. Similarly, the SR is calculated by dividing the settlement for reinforced soil to the settlement for unreinforced soil at a definite loading pressure q_D. In this study, the value of q_D is taken as 300 kPa. The bearing capacity obtained at $s/D = 3\,\%$ is used to calculate the corresponding BCRs and PRSs obtained at a loading pressure of 300 kPa and are presented. The settlement ratio (s/D) is defined as the ratio of rigid plate settlement (s) to rigid plate diameter (D). q_u, BCR, s, and PRS values were obtained from load-settlement curves of the tests.

Test Series I for Clay Soils, Without Reinforcement

The load-settlement behavior of natural clay soil in unreinforced condition was investigated in test series I. Bearing capacity of the rigid plate is defined as load (Q) divided by rigid plate area (A). Rigid plate settlements are normalized by dividing the rigid plate diameter 0.9 m. From the pressure-settlement curve (Fig. 6), the

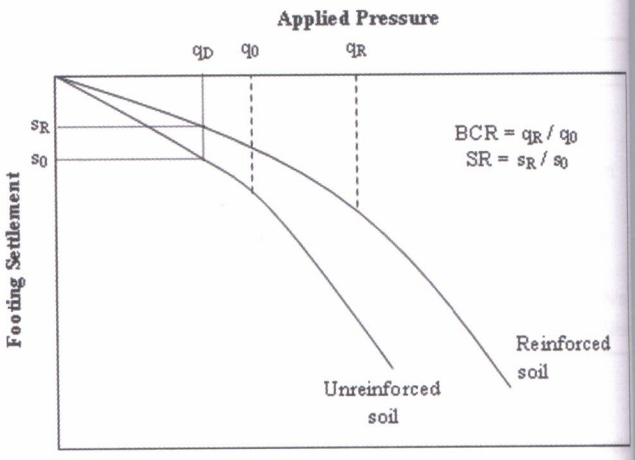


FIG. 6—*Definition of BCR and SR.*

ultimate bearing capacity was obtained as 295.7 kPa. As seen from the figure the load obtained is 181.1 kN for $s/D = 3\,\%$ and the settlement corresponding to 300 kPa is 0.029 m.

In undrained conditions the soil is often characterized by an undrained strength and an elastic stiffness. Plasticity theory is used to calculate bearing capacity. It is fairly realistic to assume that rigid plates on clay are loaded under undrained conditions and the soft clay can be accordingly modeled as a material with $c = s_u$ and $\phi = 0$, where s_u is the average undrained shear strength of the clay. The undrained bearing capacity of foundations on clay is normally calculated according to the following expression proposed by Meyerhof (1963):

$$q_u = s_c d_c N_c s_u \tag{3}$$

where:

q_u = the ultimate bearing capacity of the rigid plate,
s_c = a shape factor,
d_c = a depth factor, and
N_c = the bearing capacity factor.

Using Eq 3 the ultimate bearing capacity of the rigid plate was predicted as 370 kPa.

The bearing capacity value was obtained as the corresponding bearing capacity value of settlement ratio $s/D = 3\,\%$ of the experimental curve. In fact soil does not fail yet and has still got some strength to resist more at this stage of the settlement ratio. This explains why the results derived from the experiment and calculated from Eq 3 are different. Conventionally, the calculations for settlement in saturated clay are divided into two components (i) immediate settlements due to deformation taking place at constant volume, and (ii) the consolidation settlement accompanying the dissipation of pore water pressure. The consolidation settlement depends on time and is not taken into account in the field tests of the study.

Test Series II for Replacement of Clay Soils with Granular Fill Layers

The effect of granular fill layer thickness on bearing capacity and settlement behavior was investigated in tests series II. The relations of the loading pressure to rigid plate settlement for various

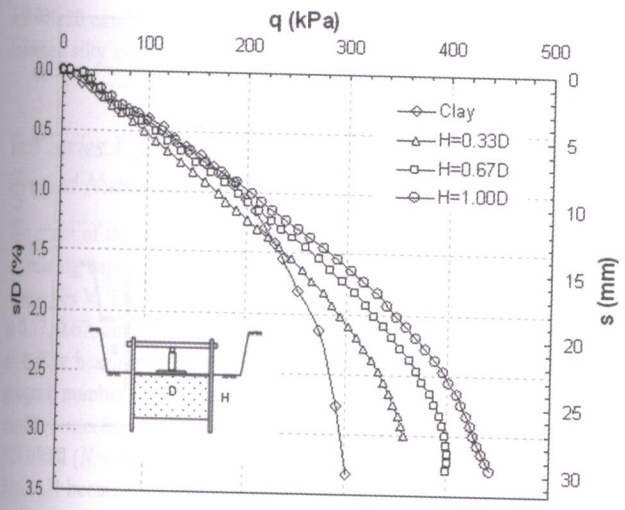


FIG. 7—*Curves of settlement against bearing capacity for tests series I and II.*

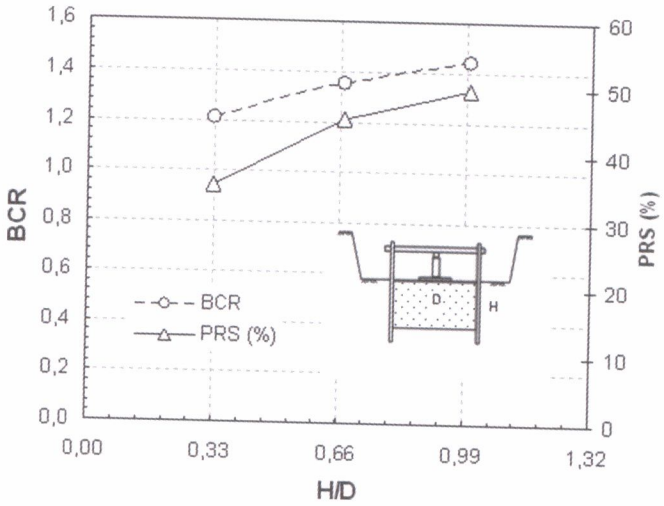


FIG. 8—*Effect of variation in H/D on BCR and PRS.*

values of H/D obtained from field tests are presented in Fig. 7 together with the result of unreinforced clay deposit (i.e., the result of the test series I). In the tests granular fill thickness was changed depending on the rigid plate diameter as $0.33D$, $0.67D$, and $1.00D$. The width of the granular fill layer was kept about $3.1D$ for all the reinforced soil conditions. As seen from Fig. 7, the bearing capacity keeps increasing with an increase in granular fill thickness. At settlement of $s/D = 3\,\%$, Fig. 6 shows that the bearing capacity increases from 295.7 (unreinforced clay deposit) to 425.7 kPa ($H = 1.00D$). Also, the applied load increases from 188.1 to 270.7 kN because of the contribution of granular fill layer on bearing capacity. It is also noted that the bearing capacity values increase significantly only after the s/D becomes greater than 1.5% at which it is believed that the effect of granular fill layer arising from partial replacement starts to be mobilized. The loading of diameter D is assumed to be spread through the granular fill to give an increased loading area on the surface of the natural clay. In test series II, the ultimate bearing capacity is a function of H/D (Madhav and Vitkar 1978; Hamed et al. 1986).

Figure 8 shows the relation of BCR to H/D and PRS to H/D ratios obtained from the tests series II (Fig. 7) by using the values of ultimate bearing capacities and settlement values correspond to 300 kPa, respectively. H/D is a ratio defined as the ratio of granular fill thickness (H) to rigid plate diameter (D). It is shown that BCR increases with an increase in the granular fill thickness. BCR values obtained are 1.21, 1.35, and 1.44 for $H = 0.33D$; $H = 0.67D$, and $H = 1.00D$, respectively. PRS values were obtained by using the settlement values corresponding to 300 kPa. It is seen that settlement decreases with an increase in granular fill thickness. PRS values obtained are 35, 45, and 50% for $H = 0.33D$; $H = 0.67D$, and $H = 1.00D$, respectively. The q_u, BCR, and PRS values calculated are given in Table 3.

Test Series III for Reinforced Clay, Effect of Depth of First Geogrid

The effect of the depth of the first welded geogrid reinforcement on bearing capacity and settlement behavior was investigated in tests series III. For the tests the values of H/D and N were kept constant at 0.67 and 1, respectively. In the tests the first reinforcement depth was changed depending on the rigid plate diameter as $0.17D$, $0.33D$, $0.50D$, and $0.67D$. The bearing capacity keeps increasing with the decrease in the first welded geogrid layer depth. Maximum performance on bearing capacity was obtained at $u = 0.17D$. For the same displacement ratio, the inclusion of the welded geogrid layer improves the performance of the rigid plate by increasing the bearing capacity and reducing the settlement of the system. At settlement of $s/D = 3\,\%$, the bearing capacity increases from 295.7 (unreinforced clay deposit) to 452.7 kPa ($u = 0.17D$). Also, the applied load increases from 188.1 to 287.9 kN because of the contribution of the granular fill layer reinforced with only one welded geogrid on bearing capacity. On removal of the rigid plate from the test area, the welded geogrid reinforcements directly below the rigid plate were visibly deflected downward in a shape conforming to the size of the rigid plate, suggesting that plunging failure had occurred.

TABLE 3—*Details of the field tests.*

Test Series	H/D	u/D	h/D	N	q_u, kPa	BCR	S, mm	PRS, %
I	…	…	…	…	295.7	1.00	28.98	0.0
II	0.33	…	…	…	357.4	1.21	18.72	35.4
	0.67	…	…	…	398.8	1.35	15.84	45.3
	1.00	…	…	…	425.7	1.44	14.58	49.7
III	0.67	0.17	…	1	452.7	1.53	13.23	53.3
	0.67	0.33	…	1	448.1	1.52	13.59	53.1
	0.67	0.50	…	1	433.5	1.47	15.48	46.6
	0.67	0.67	…	1	415.0	1.40	16.11	44.4
IV	0.67	0.17	0.17	2	463.0	1.57	13.41	53.7
	0.67	0.17	0.33	2	497.1	1.68	12.51	56.8
	0.67	0.17	0.50	2	508.5	1.72	11.79	59.3
V	0.67	0.17	…	1	452.7	1.53	13.23	53.3
	0.67	0.17	0.17	2	463.0	1.57	13.41	53.7
	0.67	0.17	0.17	3	514.0	1.74	12.87	55.6
	0.67	0.17	0.17	4	529.0	1.79	11.52	60.2

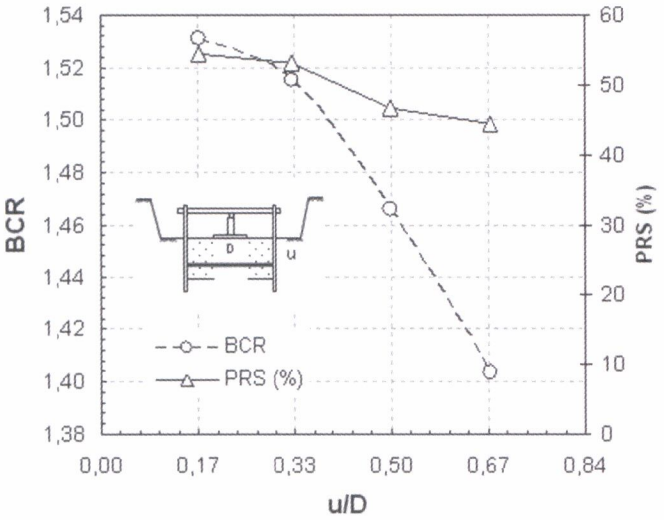


FIG. 9—*Effect of variation in u/D on BCR and PRS.*

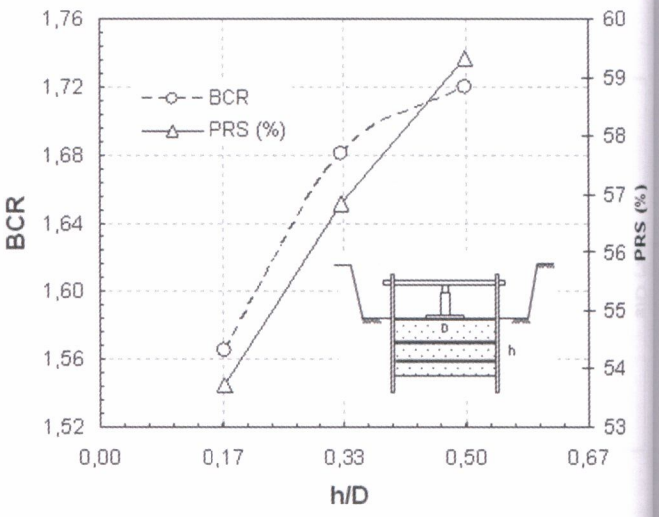


FIG. 10—*Effect of variation in h/D on BCR and PRS.*

The values of BCR at a settlement of $s/D = 3\,\%$ and PRS values at a rigid plate pressure of $q = 300$ kPa are presented as a function of the top layer spacing ratio (u/D) in Fig. 9. Figure 9 shows that the values of BCR generally decrease with an increase in the first welded geogrid depth ratio (u/D). The BCR values obtained are 1.53, 1.52, 1.47, and 1.40 for $u = 0.17D$, $u = 0.33D$, $u = 0.50D$, and $u = 0.67D$, respectively (Table 3). It is obvious that the inclusion of the reinforcement would reduce the immediate settlement significantly. The PRS values obtained are 54, 53, 47, and 44 % for u values of $0.17D$, $0.33D$, $0.50D$, and $0.67D$, respectively. This finding is similar to that reported by Sakti and Das (1987) and Shin et al. (1993). Sakti and Das (1987) reported that the maximum value of BCR was obtained at a depth of $0.35B$ for strip footing on both single and multilayer geotextile reinforced clay. Shin et al. (1993) indicated that the optimum location of the top layer was at a depth of about $0.4B$ for strip footing on clay with four layers of welded geogrid. On the other hand, based on a 0.04 -m-diameter circular footing on clay reinforced by one layer of reinforcement, Ramaswamy and Purushothaman (1992) obtained a maximum BCR at $u/B = 0.5$. Mandal and Sah (1992) indicated that the optimum top layer spacing was about $0.175B$ for 0.10 -m-wide square footing on clay with a single layer of welded geogrid. Chen (2007) reported that the maximum value of BCR was obtained at a depth of $0.33B$ for square and rectangular footing on both welded geogrid and geotextile reinforced clay. These discrepancies on the optimum location of the top layer of reinforcement may be attributed to the different properties of soil and reinforcement used by different researchers. Experimental studies have also shown a reasonable increase in the bearing capacity of the reinforced system at small displacement, and a further improvement at large displacement, which is attributed to a tensioned-membrane effect in the welded geogrid layers beneath the footing (Love et al. 1987).

Test Series IV for Reinforced Clay, Effects of Vertical Spacing of Geogrids

Test series IV was conducted to determine the influence of the vertical spacing between welded geogrids on the bearing capacity and settlement behavior of clay deposits. For the tests, the values of H/D, u/D, and N were kept constant at 0.67, 0.17, and 2, respectively. Vertical welded geogrid spacing ratio (h/D) is defined as the ratio of vertical spacing between two welded geogrids (h) to the rigid plate diameter (D). The vertical spacing of reinforcement varied from $0.17D$ to $0.50D$. The placement of the second welded geogrid increases bearing capacity. The bearing capacity reaches its maximum value when h equals to $0.50D$. At settlement of $s/D = 3\,\%$, the bearing capacities of the reinforced soil increase to 463, 497.1, and 508.5 kPa, respectively, as the vertical spacing between welded geogrids (h) are $0.17D$, $0.33D$, and $050D$.

Figure 10 depicts the relation of BCR to h/D and PRS to h/D ratios obtained from the tests series IV by using the values of ultimate bearing capacities and settlement values corresponding to 300 kPa, respectively. It is obvious that the BCR values decrease with an increase in vertical spacing of reinforcement layers with maximum BCR at $h = 0.50D$. The maximum value of BCR obtained was 1.72 (Table 3). It is seen that settlement decreases with a decrease in vertical spacing between welded geogrid reinforcements. PRS values obtained are 54, 57, and 59 % for h values of $0.17D$, $0.33D$, and $0.50D$, respectively. But, it is clear that the location of the second reinforcement does not significantly effect the bearing capacity and settlement behavior so much in the range of h values used in this study.

It is understood from the literature and experimental study that the effect of vertical spacing is not independent. Instead, it is a function of top layer spacing (u) and number of layers (N) and may also be a function of reinforcement modulus and size (Chen, 2007). Guido et al. (1985) indicated that it is difficult to fully understand the effect of vertical spacing on bearing capacity separately without considering other influencing factors. Singh (1988) indicated that the optimum vertical spacing of reinforcement layers for square footings on welded geogrid-reinforced sands varied between $0.15B$ and $0.25B$, depending on the size of the reinforcement. Yetimoglu et al. (1994) showed that the optimum vertical spacing was about $0.20B$ with rectangular footings. Chen (2007) reported that the value of

$h/B = 0.20$ can be a reasonable value to use in the design of reinforced silty clay.

Test Series V for Reinforced Clay, Effects of Geogrid Number

The effect of the number of welded geogrid reinforcement layers on bearing capacity and settlement behavior was investigated in tests series V. The values of u/D, H/D, and h/D were kept constant at 0.17, 0.67, and 0.17 while the values of N were changed from 1 to 4. The bearing capacity increases with an increase in welded geogrid number. At the settlement of $s/D = 3\,\%$, the bearing capacity increases from 295.7 (unreinforced clay deposit) to 529.0 kPa ($N = 4$). Also, the applied load increases from 188.1 to 336.4 kN because of the contribution of multiple welded geogrid reinforcements in the granular fill layer. The shapes and slopes of curves of reinforced clay deposits are very similar to those of unreinforced clay soil condition for the settlement (s/D) less than 1% and that reinforcing effect starts to mobilize when the s/D exceeds the threshold value of 1 %.

Figure 11 shows the relation of BCR to N obtained from the tests series V by using the values of ultimate bearing capacities. It is shown that the BCR value increases with an increase in number of welded geogrid reinforcements and remains relatively constant for $N > 3$. BCR values obtained are 1.53, 1.57, 1.74, and 1.79 for $N = 1$, 2, 3, and 4, respectively (Table 3). Figure 11 also depicts the variation of the PRS factors as a function of N at a specified surface pressure. It is obvious that the inclusion of reinforcement would reduce the immediate settlement significantly. With four layers of welded geogrids, the settlement can be reduced by about 60% at a rigid plate pressure of 300 kPa. Studies conducted by other researchers have also shown that increasing the number of reinforcement layers would increase the BCR of reinforced soils (e.g., Binquet and Lee 1975a; Huang and Tatsuoka 1990; Omar et al. 1993; Yetimoglu et al. 1994; Adams and Collin 1997, Chen 2007). Similar to this observation, the test results presented by Adams and Collin (1997) for large square footing on reinforced sand indicated that the benefit increases with increasing settlement. A study conducted by Binquet and Lee (1975a) for strip footing on reinforced sand indicated that BCR is constant and independent of the settlement (s/B).

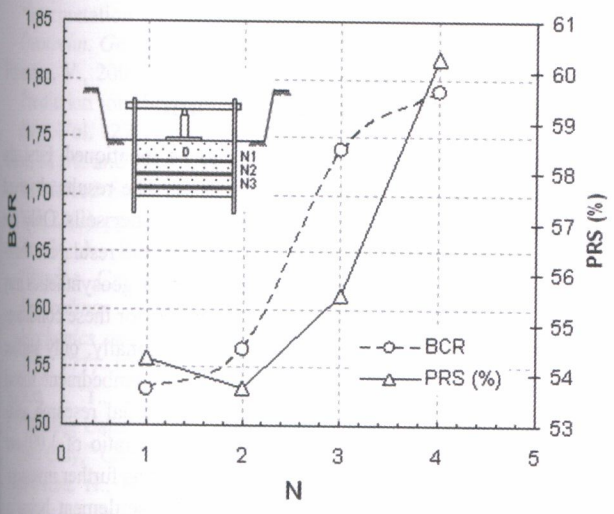


FIG. 11—*Effect of variation in N on BCR and PRS.*

The results of the field tests for unreinforced and reinforced natural clay soil are summarized in Table 3. Table 3 summarizes the testing program and variables q_u (ultimate bearing capacity), BCR (bearing capacity ratio), s (settlement), and PRS (percentage reduction in settlement) obtained from load-settlement curves of the tests.

Based on literature and the results of the study, it can be concluded that the soft soils having low load bearing capacity can be improved by replacing them with inadequate soft soils with a granular base material being compacted in layers. It is also possible to further improve the bearing capacity by reinforcing the fill material with welded geogrid reinforcement. For example, comparing the results of $H/D = 0.67$ with and without reinforcement, the values of BCR are 1.35 without reinforcement (series II) and 1.79 with four layers of welded geogrid (series V). The reinforced granular fill layer consists of one or more layers of a welded geogrid reinforcement and controlled fill placed below the circular rigid footing to create a composite material with 79% improved performance. From the comparisons of settlement response of $H/D = 0.67$ with and without reinforcement, the values of PRS are 45.30% without reinforcement (series II), 53.30, 53.70, 55.60, and 60.20% with 1, 2, 3, and 4 layers of welded geogrids (series V), respectively. It can be said that the insertion of reinforcing elements enables a transfer of tensile strength by developing friction as well as interlocking with the granular fill to produce a more rigid composite base below the rigid footing. As seen for a constant H/D ratio composite layer including the granular fill with welded geogrid can lead to enhanced settlement and improved bearing capacity performance.

The thickness of granular fill layer can be reduced by placing one or more layers of geogrid reinforcement. From the comparisons of the results of $H/D = 1.00$ without reinforcement and $H/D = 0.67$ with reinforcement of up to four welded geogrids, although the granular fill thickness is reduced by approximately 33%, the higher bearing capacity and settlement improvements can be obtained with inclusion of the reinforcement. The values of PRS and BCR are varying 49.7–60.2% and 1.44–1.79, respectively, depending on the reinforcement arrangement.

Typical design parameters presented by Wayne et al. (1998) based on a comprehensive study of experimental data, literature review, and economical considerations are compared with measured field test data. Table 4 shows the comparison of the parameters related to geometrical conditions of the test and improvement rates on bearing capacity and settlement characteristics. In this study, the typical values for the values of H, u, h, and N were selected considering the feasibility, applicability, and economy on the soil improvement facilities. Some of the test configurations discussed earlier in this paper; for example $H = 1.00D$; $h = 0.50D$, were chosen for research purposes. The fourth geogrid layer is placed at the base of granular fill layer for series V. In this case, the granular fill layer could only interlock from one side of the geogrid reinforcement layer. As seen, the typical design parameters suggested in this study approximately fall within the ranges proposed by Wayne et al. (1998) after a vast amount of literature

TABLE 4—*Typical design parameters for welded geogrids.*

	This Study (circular footing, welded geogrid)			Wayne et al. (1998) (strip and square footings, punched and drawn geogrid)		
Parameter	Typical Value	BCR	PRS	Typical Value	Recommended Value	BCR
H	$0.67D$	1.35	45 %	$0.5B$ to $1.0B$	$<2.0B$	1.5 to 2.5 (wide strip footings)
u	$0.17D$	1.53	53 %	$0.15B$ to $0.3B$	$<0.5B$	1.5 to 1.8 (square footings)
h	$0.17D$ to $0.33D$	1.56 to 1.68	54–57 %	$0.15B$ to $0.3B$	$<0.5B$	
N	2 to 4	1.56 to 1.79	54–60 %	2 to 4	<5	
b	$2.80D$	…	…	$2B$ to $3B$	$<4.0B$	
Δl	$0.90D$	…	…	$0.5B$ to $1.0B$	$<2.0B$	
a	variable	…	…	$0.1B$ to $0.2B$	$<0.3B$	

Note: D: footing diameter; B: footing width; H: thickness of the granular fill layer; u: depth of first welded geogrid reinforcement; h: vertical spacing of reinforcement layers; N: number of welded geogrid layers; b: width of the granular fill layer; Δl: length of welded geogrid extended beyond each end of footing; a: distance from lowest welded geogrid to bottom of reinforced fill; BCR: bearing capacity ratio; and PRS: percentage reduction in settlement.

review. Although Wayne et al. (1998) developed this table for punched and drawn geogrids and also developed it for square and rectangular footings, this study shows its use is appropriate for circular footings with a welded geogrid as well. In spite of the optimum depth to the first reinforcement layer obtained by Chen (2007) about $0.33B$ below the footing for all soil types tested in his study, the results of Wayne et al. and this study showed that the optimum depth to the first reinforcement layer was varying between 0.15 and 0.30.

Conclusions

A total of 15 large scale model plate tests were conducted to evaluate potential benefits of welded geogrid reinforced granular fill layer constructed above natural clay deposits. Based on the results from this investigation, the following main conclusions can be drawn.

The bearing capacity of a circular footing can be improved and footing settlement can be reduced by partially replacing natural clay soil with the compacted stiffer granular fill layer. The bearing capacity increases up to 44 % and settlement reduction obtained 50 % when granular fill thickness equals $1.0D$ (series II).

Maximum benefits in bearing capacity and settlement were obtained at the depth of the first reinforcement layer of $u = 0.17D$. BCR and PRS obtained at $u = 0.17D$ were 1.53 and 53.3 %, respectively (series III).

The inclusion of second reinforcement in granular fill layer increases bearing capacity up to 72 % and decreases settlement up to 59.3 % at $h = 0.50D$ (series IV).

The addition of more than two welded geogrid layers of reinforcement contributes to the bearing capacity and settlement improvement; thus the BCR and PRS values are obtained as 1.79 and 60.2 % when the number of welded geogrid layers is four (series V).

The settlement behavior can be further improved by placing welded geogrid layers within the granular fill layer, depending on the reinforcement arrangement. For example, comparing the results of $H/D = 0.67$ with and without reinforcement, the values of PRS are 45.3 % without reinforcement (series II) and 60.2 % with four layers of welded geogrid (series V). For the case of bearing capacity, comparing the results of $H/D = 0.67$ with and without reinforcement, the values of BCR are 1.35 without reinforcement (series II) and 1.79 with four layers of welded geogrid (series V). As such, for similar H/D ratios the geogrid composite layer can lead to enhanced settlement and improved bearing capacity performance.

Comparing the results of $H/D = 1.00$ without reinforcement and $H/D = 0.67$ with different reinforcement configurations up to four welded geogrids, although the necessary granular fill thickness is reduced by approximately 33 %, the better bearing capacity and settlement improvements can be obtained in the reinforced case. The values of PRS and BCR are varying 49.7–60.2 % and 1.44–1.79, respectively, depending on the reinforcement arrangement.

The typical design parameters suggested in this study approximately fall within the ranges proposed by Wayne et al. (1998) after a vast amount of literature review. Although the parameter range is also developed for square and rectangular footings with punched-drawn geogrids, this study shows its use is appropriate for circular footings with welded geogrid as well.

Nevertheless, the investigation is considered to have provided a useful basis for further research leading to an increased understanding of the application of soil reinforcement to ultimate bearing capacity and settlement problems.

Limitations of Research Program

There are several limitations that should be mentioned. First, the tests were conducted on only one soil type. The results observed from this test program may be different for other soils. Only one type of geogrid reinforcement was evaluated; the results are therefore specific to the reinforcement tested. Other geosynthetics may perform very differently. The trends observed for these conditions should be verified for larger footings. Additionally, only surface footings were tested. The effect of footing embedment should also be included in future works. The additional research using different granular fill thickness (such as H/D ratio of 1.00) and reinforcement configurations is recommended as further investigations to improve the understanding of the settlement behavior comprehensively, which may lead to developing a design method

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References

Abled-Baki, S., Raymond, G. P., and Johnson, P., 1993, "Improvement of the Bearing Capacity of Footings by a Single Layer of Reinforcement," *Proceedings of Geosynthetics'93 Conference*, Canada, pp. 407–416.

Abu-Farsakh, M., Chen, Q., Sharma, R., and Zhang, X., 2008, "Large Scale Model Foundation Tests on Geogrid Reinforced Footing and Marginal Embankment Soils," *Geotech. Test. J.*, Vol. 31(5), pp. 413–423.

Adams, M. and Collin, J., 1997, "Large Model Spread Footing Load Tests on Geosynthetic Reinforced Soil Footings," *J. Geotech. Geoenviron. Eng.*, Vol. 123(1), pp. 66–72.

Akinmusuru, J. O. and Akinbolade, J. A., 1981, "Stability of Loaded Footing on Reinforced Soil," *J. Geotech. Eng.*, Vol. 107(6), pp. 819–827.

ASTM D1196-93, 1997, "Standard Test Method for Onrepetitive Static Plate Load Tests of Soils and Flexible Pavement Components for Use in Evaluation and Design of Airport and Highway Pavements, *Annual Book of ASTM Standards*, ASTM International, West Conshohocken, PA, pp. 112–113.

Binquet, J. and Lee, K. L., 1975a, "Bearing Capacity Tests on Reinforced Earth Slabs," *J. Geotech. Eng. Div.*, Vol. 101(GT12), pp. 1241–1255.

Binquet, J. and Lee, K. L., 1975b, "Bearing Capacity Analysis on Reinforced Earth Slabs," *J. Geotech. Eng. Div.*, Vol. 101(GT12), pp. 1257–1276.

Cerato, A. B. and Lutenegger, A. J., 2006, "Bearing Capacity of Square and Circular Footings on a Finite Layer of Granular Soil Underlain by a Rigid Base," *J. Geotech. Geoenviron. Eng.*, Vol. 132(11), pp. 1496–1501.

Chen, Q., 2007, "An Experimental Study on Characteristics and Behavior of Reinforced Soil Footing," Ph.D. Thesis, LA State Univ., Baton Rouge, LA.

Consoli, N. C., Schnaid, F., and Mititsky, J., 1998, "Interpretation of Plate Load Tests on Residual Soil Site," *J. Geotechn. Geoenviron. Eng.*, Vol. 124(9), pp. 857–867.

Fonseca, V., 2001, "Load Tests on Residual Soil and Settlement Prediction on Shallow Footing," *J. Geotechn. Geoenviron. Eng.*, Vol. 127(10), pp. 869–883.

Gabr, M. A. and Hart, J. H., 2000, "Elastic Modulus of Geogrid Reinforced Sand Using Plate Load Tests," *Geotech. Test. J.*, Vol. 23(2), pp. 245–250.

Guido, V. A., Biesiadecki, G. L., and Sullivan, M. J., 1985, "Bearing Capacity of a Geotextile Reinforced Footing," *Proceedings of the 11th International Conference on Soil Mechanics and Footing Engineering*, San Francisco, CA Vol. 3, pp. 1777–1780.

Hamed, J. T., Das, B. M., and Echelberger, W. F., 1986, "Bearing Capacity of a Strip Foundation on Granular Trench in Soft Clay," *Civil Eng. Practising Design Eng.*, Vol. 5(5), p. 359.

Hakushima, H., Nishikawa, J., and Tomisawa, K. T., 2003, "A Study on the Scale Effect of Ultimate Bearing Capacity on Rock Fill," *Mon. Rep. Civil Eng. Res. Inst.*, Vol. 600, pp. 21–28.

Huang, C. C. and Tatsuoka, F., 1990, "Bearing Capacity Reinforced Horizontal Sandy Ground," *Geotext. Geomembr.*, Vol. 9, pp. 51–82.

Khing, K. H., Das, B. M., Puri, V. K., Cook, E. E., and Yen, S. C., 1993, "The Bearing Capacity of a Strip Foundation on Geogrid Reinforced Sand," *Geotext. Geomembr.*, Vol. 12(4), pp. 351–361.

Kumar, A., Ohri, M. L., and Bansal, R. K., 2007, "Bearing Capacity Tests of Strip Footings on Reinforced Layered Soil," *J. Geotech. Geol. Eng.*, Vol. 25(2), pp. 139–150.

Kumar, A. and Saran, S., 2003, "Bearing Capacity of Rectangular Footing on Reinforced Soil," *J. Geotech. Geol. Eng.*, Vol. 21(3), pp. 201–224.

Laman, M. and Yildiz, A., 2003, "Model Studies of Ring Footings on Geogrid-Reinforced Sand," *Geosynth. Int.*, Vol. 10(5), pp. 142–152.

Laman, M., Yildiz, A., Ornek, M., and Demir, A., 2009, "Geogrid Reinforcement on Soft Clay Deposit," TUBITAK Scientific Research Project (No: 106M496), Ankara, Turkey, 528p.

Love, J. T., Burd, H. J., Milligan, G. W. E., and Houlsby, G. T., 1987, "Analytical and Model Studies of Reinforcement of a Granular Layer on a Soft Clay Subgrade," *Can. Geotech. J.*, Vol. 24, pp. 611–622.

Madhav, M. R. and Vitkar P. P., 1978, "Strip Footing on Weak Clay Stabilized with a Granular Trench or Pile," *Can. Geotech. J.*, Vol. 15(4), p. 605.

Mandal, J. N. and Sah, H. S. 1992, "Bearing Capacity Tests on Geogrid-Reinforced Clay," *Geotext. Geomembr.*, Vol. 11(3), pp. 327–333.

Meyerhof, G. G., 1963, "Some Recent Research on the Bearing Capacity of Foundations," *Can. Geotech. J.*, Vol. 1(1), pp. 16–36.

Omar, M. T., Das, B. M., Yen, S. C., Puri, V. K., and Cook, E. E., 1993, "Ultimate Bearing Capacity of Rectangular Footings on Geogrid-Reinforced Sand," *Geotech. Test. J.*, Vol. 16(2), pp. 246–252.

Ornek, M., 2009, "Geogrid Reinforcement on Soft Clay Deposits," Ph.D. Thesis, Univ. of Cukurova, Turkey, 318 p. (in Turkish).

Ramaswamy, S. D. and Purushothaman, P., 1992, "Model Footings of Geogrid Reinforced Clay," Proceedings of the Indian Geotechnical Conference on Geotechnique Today, Vol. 1, pp. 183–186.

Sakti, J. and Das, B. M., 1987, "Model Tests for Strip Footing on Clay Reinforced with Geotextile Layers," *Transportation Research Record No. 1153*, National Academy of Sciences, Washington, D.C., pp. 40–45.

Shin, E. C., Das, B. M., Puri, V. K., Yen, S. C., and Cook, E. E., 1993, "Bearing Capacity of Strip Footing on Geogrid-Reinforced Clay," *Geotech. Test. J.*, Vol. 16(4), pp. 534–541.

Singh, H. R., 1988, "Bearing Capacity of Reinforced Soil Beds," Ph.D. Thesis, Indian Institute of Science, Bangalore, India.

Wayne, M. H., Han, J., and Akins K., 1998, "The Design of Geosynthetic Reinforced Foundations," *Geosynthetics in Foundation Reinforcement and Erosion Control Systems*, Geotechnical Special Publication No. 76, J. J. Bowders, H. B. Scranton, and G. P. Broderick, Eds., Vol. 76, pp. 1–18.

Yetimoglu, T., Wu, J. T. H., and Saglamer, A., 1994, "Bearing Capacity of Rectangular Footings on Geogrid-Reinforced Sand," *J. Geotech. Eng.*, Vol. 120(12), pp. 2083–2099.