Geotextiles and Geomembranes 38 (2013) 1–15

Contents lists available at SciVerse ScienceDirect

Geotextiles and Geomembranes

journal homepage: www.elsevier.com/locate/geotexmem

Large scale field tests on geogrid-reinforced granular fill underlain by clay soil

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ARTICLE INFO

Article history:
Received 1 June 2011
Received in revised form
11 April 2012
Accepted 22 May 2012
Available online 5 December 2012

Keywords:
Field test
Geogrid
Natural clay
Granular fill
Subgrade modulus
Bearing capacity

ABSTRACT

This study aims at experimentally explaining the potential benefits of geogrid reinforced soil footings using large scale field tests. A total of 16 field tests were carried out to evaluate the effects of replacing natural clay soil with stiffer granular fill layer and single-multiple layers of geogrid reinforcement placed into granular fill below circular footings. The large scale field tests were performed using different size of the circular footing diameters which have 0.30, 0.45, 0.60 and 0.90 m. The results of testing program are presented in terms of subgrade modulus and bearing capacity. These values were calculated for each test at settlements of 10, 20 and 30 mm. Based on the test results, it is shown that the use of granular fill and geogrid for reinforced soil footings (RSF) have considerable effects on the subgrade modulus and bearing capacity. Finally, the field test results are compared to the analytical methods proposed by different researchers including the statistical correlations.

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1. Introduction

In many cases of construction, shallow foundations are built on top of existing cohesive soils, resulting in low bearing capacity and/or excessive settlement problems. An economical treatment method is the use of reinforced soil foundation (RSF). This can be done by either reinforcing cohesive soil directly or replacing the poor soils with stronger granular fill, in combination with geosynthetics. In this technique, one or more layers of a geosynthetic reinforcement and controlled fill material are placed beneath the footing to create a composite material with improved performance characteristics. This technique is commonly used for unpaved roads, embankments, and large stabilized areas such as car parks or working platforms for oil drilling (Giroud and Noiray, 1981; Giroud et al., 1984; Rowe and Soderman, 1986; Love et al., 1987; Fannin and Sigurdsson, 1996; Miura et al., 1990; Ling and Liu, 2001; Rowe and Li, 2005; Hufenus et al., 2006). In comparison with other applications of geosynthetic-reinforced soil, relatively less emphasis has been placed on reinforced soil foundations. There have been some studies of shallow foundations on reinforced soil systems, most of them concentrating on sandy soil (Adams and Collin, 1997; Gabr and Hart, 2000; Fonseca, 2001; DeMerchant et al., 2002; Fukushima et al., 2003; Latha and Somwanshi, 2009).

Analytical models have been proposed for calculation of the bearing capacity of a compacted 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).

However, a limited number of experimental studies are available at the present time relating to the bearing capacity of shallow foundations on reinforced granular material of limited thickness overlying soft clay (Love et al., 1987; King et al., 1993; Ornek, 2009; Consoli et al., 2009; Mohamed, 2010). Love et al. (1987) studied the effectiveness of geogrid reinforcement, placed at the base of a layer of granular fill on the surface of clay by small-scale model tests in the laboratory. In the tests, only one geogrid layer was used at the interface between granular fill and clay soil. They showed that performance of reinforcement systems to be excellent even at small deformations, due to the significant change in the pattern of shear forces acting on the surface of the clay. King et al. (1993) carried out some laboratory model tests to evaluate the improvement of ultimate bearing capacity of shallow strip foundation supported by

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doi:10.1016/j.geotexmem.2012.05.007

a strong sand layer underlain by weak clay with a layer of geogrid at the sand–clay interface. Based on the model test results, they showed that the maximum benefit in increasing the ultimate bearing capacity by inclusion a layer of geogrid at the sand–clay interface is obtained at $H/B = 0.67$ (H, the thickness of sand layer; B, the size of strip footing) and the optimum width of the geogrid layer for improvement of ultimate bearing capacity is $6.0B$. However, this topic is still being researched and there is no an accepted design technique in the practice. This study aims to be given an attention and it can be an alternative improvement method for shallow spread footings with low load capacity. It is necessary for engineers to understand more fully their behavior in order to carry out safe and economical design and construction. This paper relates to some recent field test results which were conducted to determine the bearing capacity and settlement behavior of a circular footing supported by a reinforced stiffer granular layer of limited thickness over soft clay. Subgrade modulus was also defined to evaluate improvement performance of geogrid reinforced system.

2. Field testing program

It is sometimes difficult to accurately model the full scale behavior of reinforced soil using small scale laboratory test due to the scale effect (Abu-Farsakh et al., 2008). Main reason for performing the tests in field is to indicate properly how the bearing capacity and subgrade modulus is affected by footing size in unreinforced and granular fill with and without geogrid reinforcement. The field testing program was carried out in the Adana Metropolitan Municipality's (AMM) Water Treatment Facility Center (WTFC) located in west part of Adana, Turkey. A total of 24 reaction piles were constructed in WTFC test area for large scale field tests. The locations of the piles in test area are shown in Fig. 1. Each plate load test was conducted in field test pit measuring 2.8 m × 2.8 m in plan, and 2.0 m in depth as seen in Fig. 1. Four different model rigid footings fabricated from mild steel was used for all load tests. The diameters of the footings are 0.30, 0.45, 0.60 and 0.90 m. All the footings have 0.03 m of thickness. The footings were loaded using a hydraulic jack against a reaction frame. A total of 16 field tests were performed on unreinforced clay soil, granular fill reinforced clay soil and geogrid reinforced granular fill over clay soil, respectively.

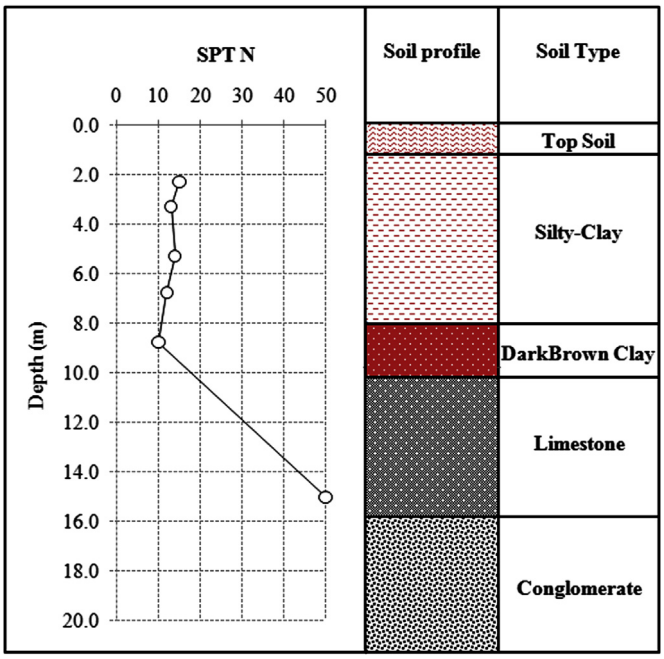


Fig. 2. Average SPT(N) values measured from borehole drillings.

3. Material properties

3.1. Site characterization

The soil conditions at the experimental test site (WTFC) were determined from a geotechnical site investigation comprised of both in-situ and laboratory tests (Laman et al., 2012). To define the soil profile of test site, two test pit excavations and four borehole drillings were performed. Ground water level was observed as 2.2 m from in the field. The general layout plan of the test area

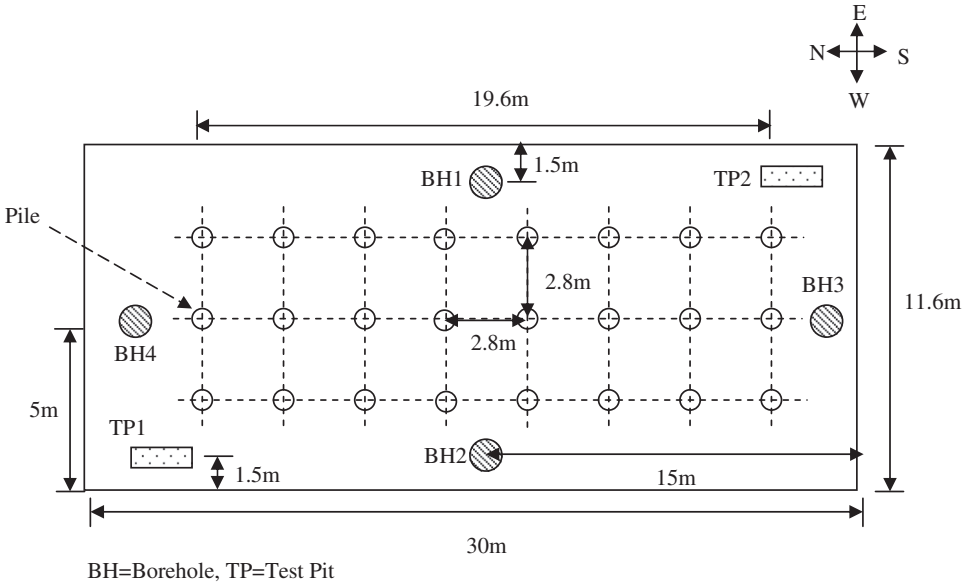


Fig. 1. Plan view showing piles, borings and test pits.

including test pits and boreholes locations is shown in Fig. 1. Five subsoil layers were clearly identified. First layer of 1.0 m depth observed as topsoil was removed before tests. The second layer between the depths of 1.0 m and 8.0 m shows silty clay stratum with high plasticity (CH) according to the Unified Soil Classification System (USCS). The soil layer below the silty clay layer was defined as dark brown clay (CH) between the depths of 8.0 m and 10.0 m. Soil profile was changed as limestone and conglomerate, respectively after 10.0 m depth as seen in Fig. 2. And also, Fig. 2 shows SPT results carried out during borehole drillings. From the laboratory tests, the clay content of the soil layers varies in the range of between 60 and 70%. The water content of the stratified soil layers varies between 20 and 25% depending on depth, and is almost the same as or greater than, the plastic limit. Specific gravity values 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 75 kN/m^2. The soil layers were classified as lightly overconsolidated soil (OCR = 1.0–2.7) from odometer tests. Some typical characteristics (clay content, specific gravity, water content and unconfined strength) of the natural clay deposits along the depths are shown in Fig. 3. Menard Modulus (Em) and net limit pressure changes along the depth obtained from the Pressuremeter Test (PMT) were also presented in Fig. 4.

3.2. Granular fill bed

Geogrid layer was placed in granular fill bed compacted on natural clay soil. Some laboratory conventional tests to obtain the index and engineering properties were conducted on the granular fill material which was obtained from the Kabasakal region situated northwest of Adana, Turkey. The granular fill material used in laboratory conventional and field test were obtained from natural granular material passing through 4.75 mm opening sieve in order to provide homogeneity in laboratory and field test conditions.

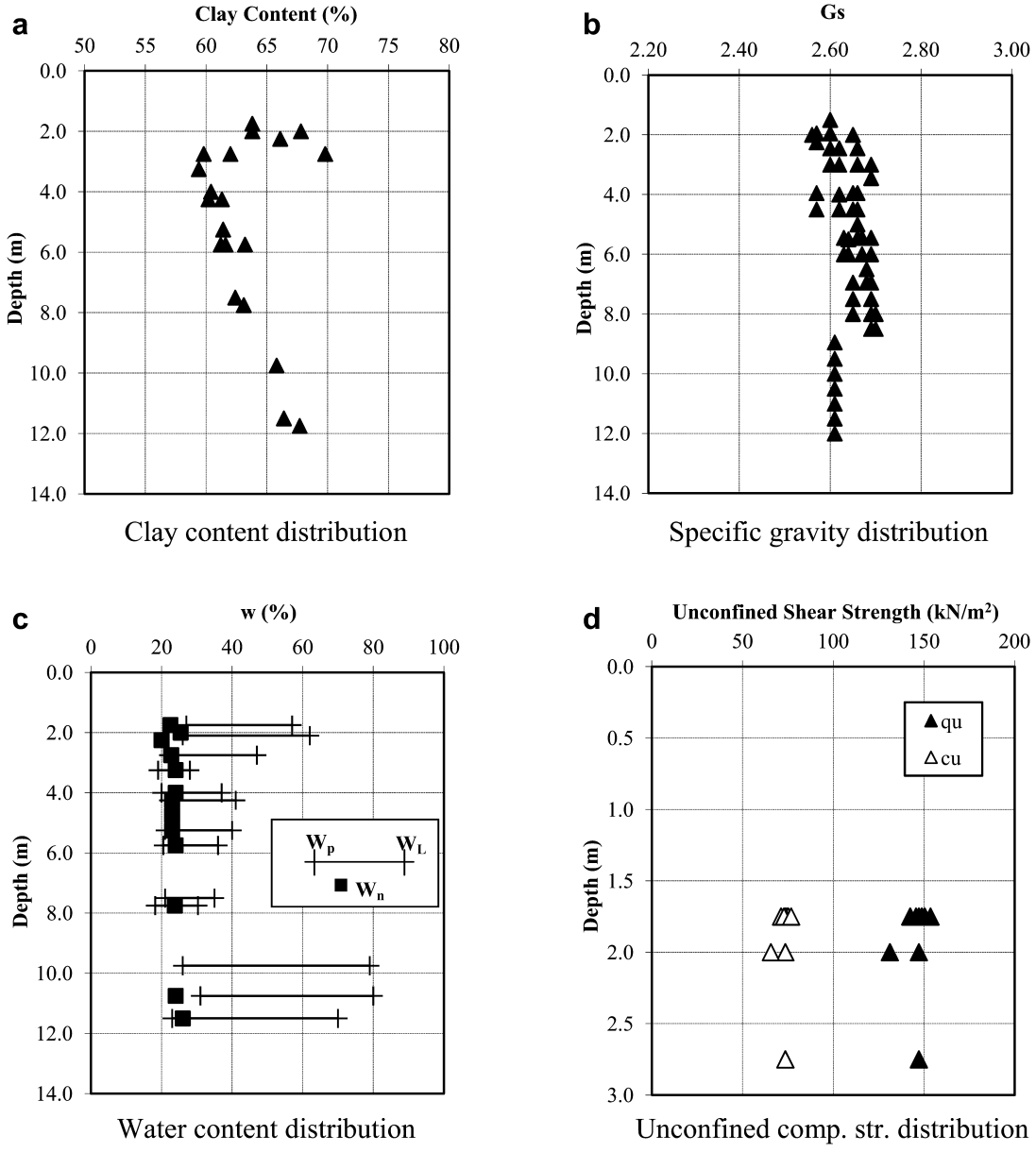


Fig. 3. Typical soil characteristics of natural clay soil.

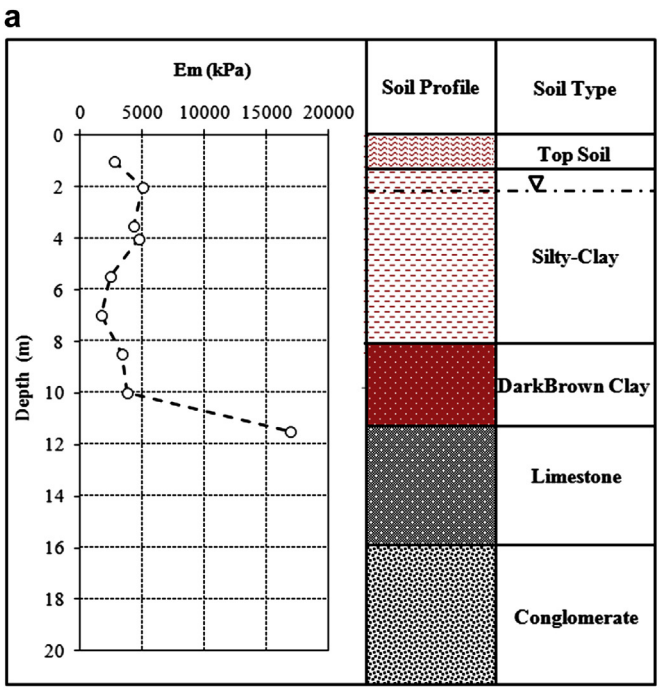


Fig. 4. PMT results on the field test area.

Granular fill bed was prepared at a value of optimum moisture content of 7% and was compacted so that the average becomes equal to a maximum dry unit weight of 21.7 kN/m^3 obtained from the Standard Proctor test. The values of internal friction angle and the cohesion of granular fill were obtained as 43° and 15.0 kN/m^2, respectively from direct shear tests. Specific gravity of the granular fill was obtained as 2.64. Typical soil characteristics of sieved granular fill material are given in Fig. 5.

3.3. 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 physical and mechanical properties of the geogrids as listed by the manufacturer are given in Table 1.

4. Test setup and procedures

For each reinforced test condition, the granular fill material was placed and compacted in layers. The amount of granular fill material and water needed for each layer was first calculated. Then, the granular fill material was compacted using a plate compactor to predetermined height to achieve the desired densities. When the desired reinforcement depth was reached, geogrid layer was placed and compaction was then continued until the desired granular fill height was reached. After that, the rigid model footings were placed at predetermined locations between the piles in the test area. The reaction frame was set onto piles. In each test a series of monotonically increasing static loads were applied to the reaction frame seated directly on the reaction piles. The load was applied using a hydraulic jack and maintained manually with a hand pump until the ultimate vertical deformation was obtained. The load and the corresponding footing settlement were measured by a calibrated pressure gauge and two LVDT's, respectively. The testing procedure was performed according to the ASTM D 1196-93 (ASTM, 1997), where the load increments applied and maintained until the rate of settlement was less than 0.03 mm/min over three consecutive minutes. To maintain the same density in the granular fill layer, the convenient compactive effort was applied on each layer of granular fill. Some tests were repeated twice to verify the repeatability and the consistency of the test data. The difference was considered to be small and neglected. Detailed information of the testing procedure can be found in Laman et al. (2009) and Ornek (2009). The general layout of the test setup is given in Fig. 6.

The effect of using stiffer granular fill with and without geogrid reinforcement on the subgrade modulus and bearing capacity was evaluated in this study. The research was conducted in three 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 of circular footings on the surface of natural clay deposit. Series II and Series III were performed on the footings settled on replacing natural clay with stiffer granular fill (without geogrid) and geogrid reinforced granular fill layer overlying by natural clay deposit, respectively. The thickness of granular fill was taken as 0.67D, corresponding to diameter of footing in Series II. Single layer or two layers of geogrid were used as a reinforcement material in Series III. In all the test series footing diameter was changed as 0.30, 0.45, 0.60 and 0.90 m. Table 2 summarizes the testing program where N is the number of the geogrid layer; D is the footing diameter; H is the granular fill thickness; u and h are the depth of first reinforcement layer and vertical spacing between two reinforcement layers beneath the footing, respectively.

5. Test results and discussion

5.1. Interpretation of test results

Bowles (1996) indicated that the modulus of subgrade reaction is a conceptual relationship between soil pressure and deflection that is widely used in the structural analysis of footing members. In field tests, the applied pressure versus the resulting settlement data was plotted for each of the model footings used. From these plots, the subgrade modulus was determined as

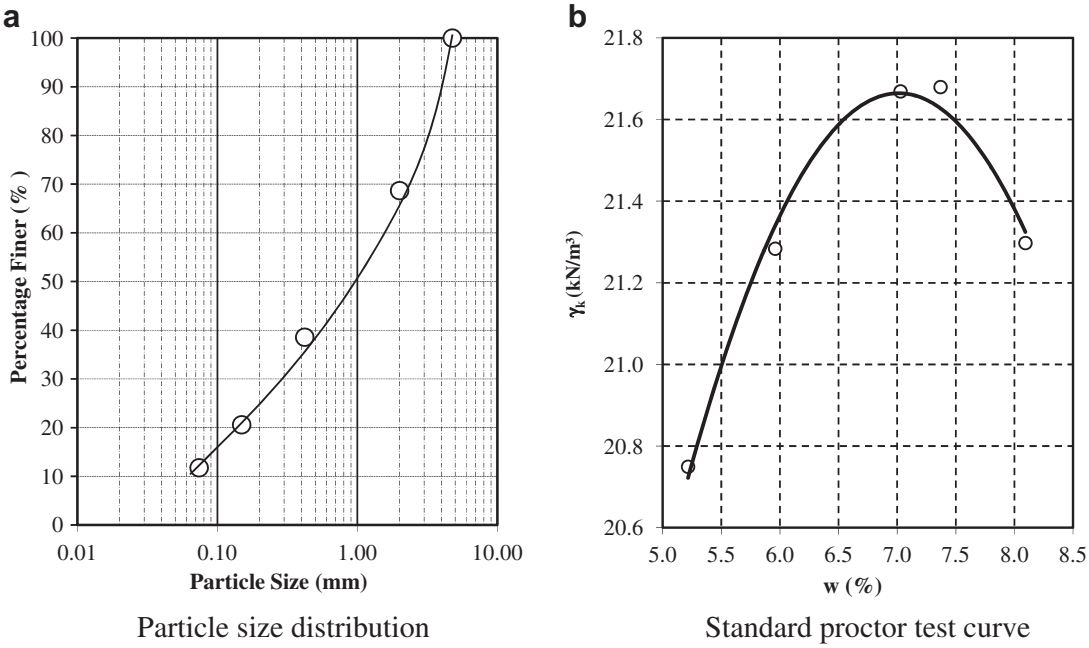


Fig. 5. Typical soil characteristics of sieved granular fill material. (a) Particle size distribution (b) Standard proctor test curve.

$$k = \frac{q}{\delta} \tag{1}$$

where k is subgrade modulus, q is the bearing capacity, and δ is the settlement. Determining the bearing capacity would have been more appropriate for the case where allowable settlement controls performance. Hence, subgrade modulus and bearing capacity values were calculated for each test for settlements of 10 (δ_{10}), 20 (δ_{20}) and 30 mm (δ_{30}).

The performance improvement due to the reinforced soil footing (RSF) in clay soil in terms of the reduction in footing settlement is quantified using the parameter called Percentage Reduction in Footing settlement (PRS). The PRS is calculated at different footing settlements (S_r) in case of RSF with respect to the footing settlement (S_o) of the unreinforced clay soils as shown below. The PRS is similar to the parameter defined by Mandal and Sah (1992) to quantify the reduction in footing settlements (settlement reduction factor, SRF) of the footing due to geogrid reinforcement in marine clay beds.

$$\mathrm{PRS} = \frac{S_o - S_r}{S_o} \times 100 \tag{2}$$

where S_o is the settlement of unreinforced clay soil at a given footing pressure and S_r is the settlement of RSF (with granular fill/geogrid reinforcement) at the same footing pressure. 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:

$$\mathrm{BCR} = \frac{q_R}{q_0} \tag{3}$$

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).

Table 1
Engineering properties of geogrid.

Parameter	Value
Structure	Biaxial
Aperture shape	Squared
Aperture size	30 mm × 30 mm
UV-resistance	>94%
Raw material	Polypropylene
Elongation at nominal strength	8%
Tensile strength at 2% elongation	22/22 (md/cmd)
Tensile strength at 5% elongation	48/48 (md/cmd)

5.2. Series I: natural clay (unreinforced)

Series I consisted of testing 0.30, 0.45, 0.60 and 0.90 m circular footings on the surface of natural clay. The Series I tests were to be the control (unimproved) tests with which to compare the RSF tests in Series II and III. Fig. 7 shows plots of applied pressure versus footing settlement for different large scale field tests on the unimproved natural clay. The curves in the figure have approximately the same trends. For the tests shown in Fig. 7, the settlement pattern generally resembles a typical local shear failure and the maximum load bearing capacity was not clearly well defined in each case. Hence, the values of bearing capacity were calculated in relation to footing settlement.

It is often helpful to present relationships in terms of dimensionless variables so that advantage can be taken of the scaling effects of continuum mechanics (Wroth, 1988). Normalization of the results of large scale field tests can establish general tendencies in relation to different footing diameters, the thicknesses of the granular fill. Such an approach has been used with data from Fig. 7. The first step toward normalization involves plotting the applied footing pressure against the settlement-to-diameter ratio (s/D) for each footing. The resulting plots for the tests on the surface of natural clay are shown in Fig. 8. It is expected that the results on saturated natural clay should be same theoretically since the water table is close to ground surface in the field. The obtained responses from the experimental results are found to diverge slightly in the large settlements. The reason for this may be due to the non-

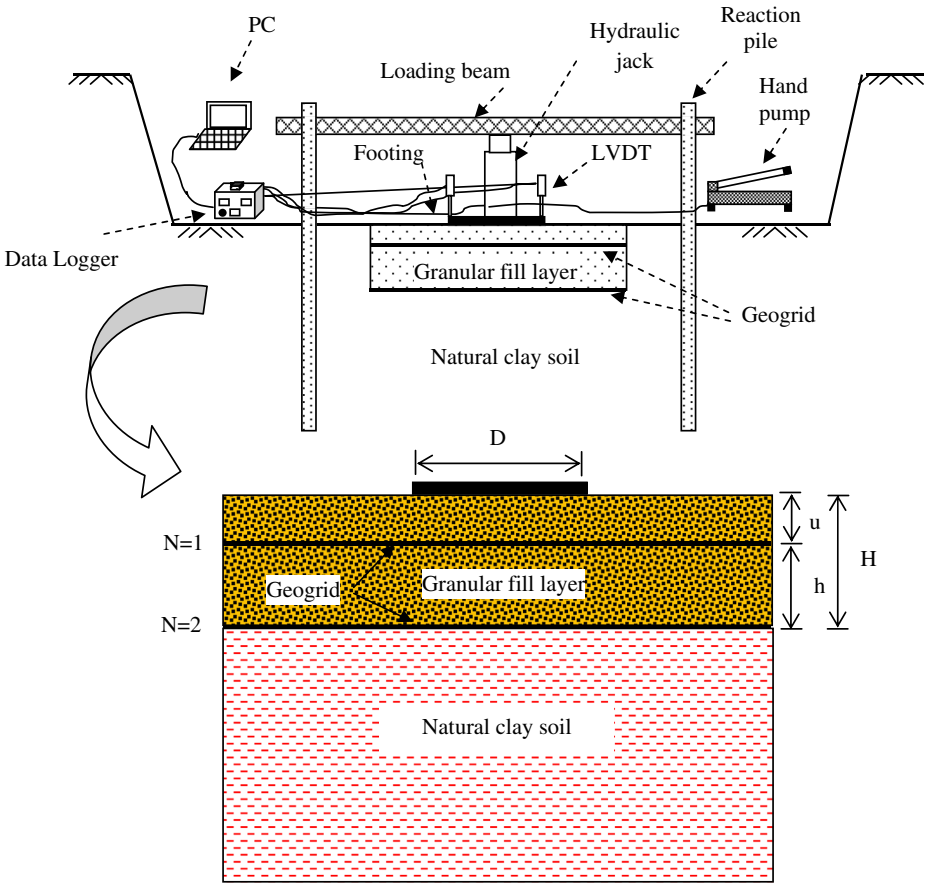


Fig. 6. Schematic view of test setup, loading and reaction system.

uniformity of the in situ soils and/or due to the load-capacity limitations of the reaction frame, it was not possible to completely reach the peak loads on footing with 0.90 m diameter. However, to the foundation engineer, the region of interest is the working load range, which is about one-third or at most one half of the ultimate value, and in those regions the results of the large scale field test show same behavior. To check the validity of the uniqueness of q and s/D in homogeneous soil profiles in order to disregard the effect of footing size outside the immediate environment in which tests were carried out, the writers considered it useful to identify other soils that might exhibit similar behavior. Three sets of results are presented in Fig. 9 for a homogeneous layer of dense compacted dune sand (D'Appolonia et al., 1968), a homogeneous upper layer of dense cohesionless wind blown sand with little silt (Ismael, 1985) and a residual homogeneous cohesive-frictional soil (Consoli et al., 1998). For each set of data a small amount of scatter was observed, but it is reasonable to say that data from D'Appolonia et al. (1968) plot as a linear relationship, and data from Ismael (1985) and Consoli et al. (1998) are nearly same. In these cases the behavior of pressure to settlement (q) to diameter ratio (s/D) seems to be independent from footing size on unimproved natural clay.

Table 2
Details of the field test program.

Test series	Test conditions	Number of tests
I	Unreinforced	4
II	$H/D = 0.67$	4
III	$N = 1$; $u/D = 0.67$; $H/D = 0.67$	4
	$N = 2$; $u/D = 0.17$; $h/D = 0.50$; $H/D = 0.67$	4

5.3. Series II: the effect of granular fill

Series II consisted of testing 0.30, 0.45, 0.60 and 0.90 m circular footings on the compacted granular fill layer of limited thickness over the natural clay. The thickness of compacted granular fill layers

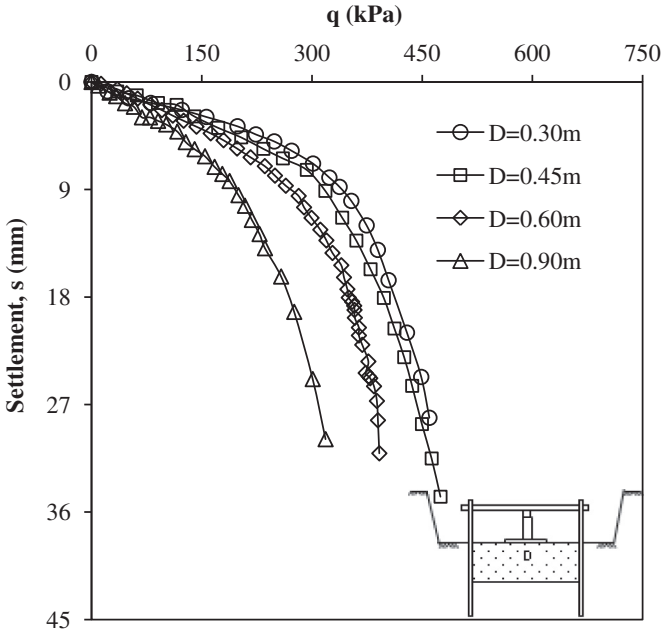


Fig. 7. Pressure versus settlement for Series I.

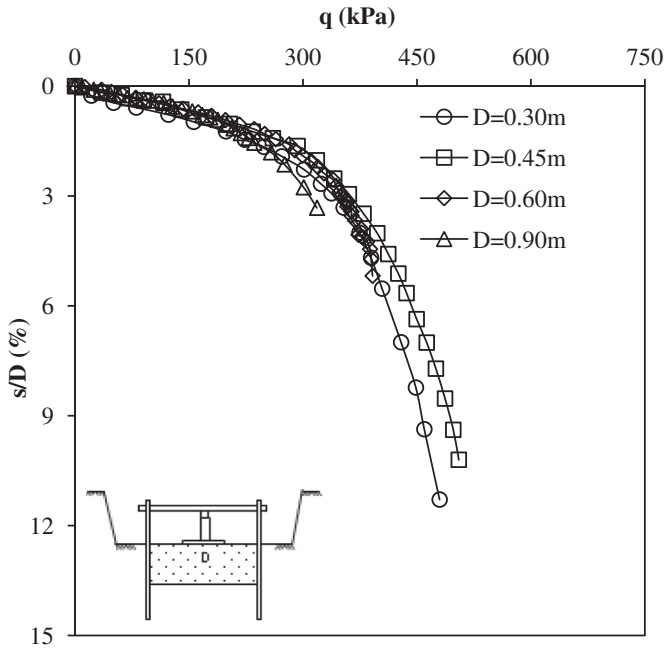


Fig. 8. Pressure versus settlement (s) to diameter (D) ratio for Series I.

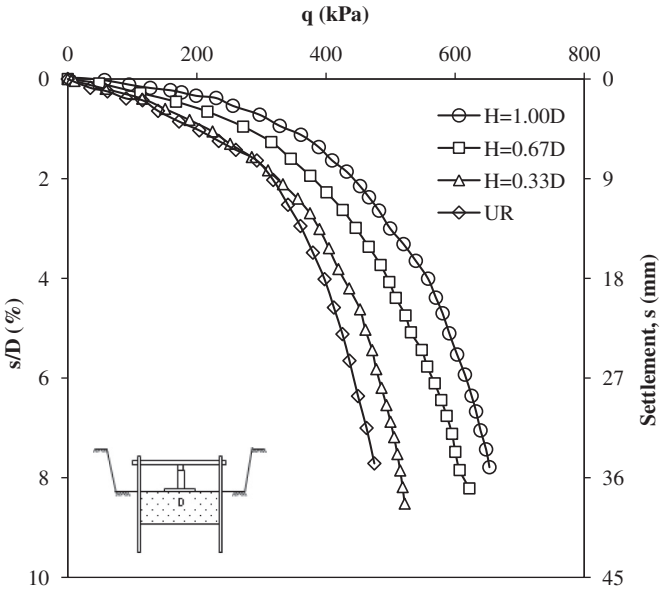


Fig. 10. Pressure–settlement curves of compacted granular fill layer ($D = 0.45$ m).

was initially investigated for the footing with 0.45 m diameter. Fig. 10 shows plots of applied pressure versus footing settlement for different thickness of compacted granular fill layers. It can be seen from the figure that bearing capacity increases with an increase in thickness of granular fill layer. Based on the results, the thicknesses of the granular fill layer were selected as 0.67D according to the footing diameter as suggested by King et al. (1993). Fig. 11 shows the results for all footing diameters on the compacted granular fill layer of limited thickness ($H = 0.67D$) over the natural clay. The values of bearing capacity and subgrade modulus were calculated for each test at settlements of δ_{10}, δ_{20} and δ_{30} and presented in Table 3. It is shown that the granular fill layer increases the load bearing capacity of the footing and decreases the settlement since the granular fill layer is stiffer and stronger than the natural clay.

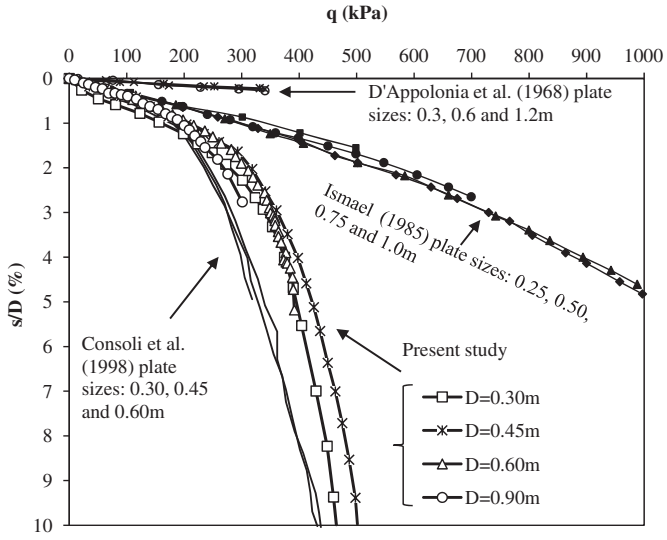


Fig. 9. Comparison of curves of pressure versus settlement (s) to diameter (D) ratio for Series I.

For this reason, bearing capacity can be improved while footing settlement is reduced. It is observed that the load–settlement curve is rounded and becomes steeper and takes almost a linear shape. An ultimate bearing capacity is never observed and no definite failure point can be established. The mode of failure can be described as local shear failure.

5.4. Series III: the effect of reinforcement with geogrid

The tests in this series were conducted to determine the effect of geogrid reinforcement within the compacted granular fill layer on bearing capacity and subgrade modulus. The thickness of granular fill layer, H was kept constant as 0.67D. Series III was divided into two sets. The first set of Series III tests consisted of a single geogrid layer. Results of a limited number of experimental studies are available in the literature (e.g., Love et al., 1987; King et al., 1993) relating to the bearing capacity of shallow foundations on a strong fill layer of limited thickness overlying a natural clay. In these studies, a single geosynthetic layer was generally placed at the interface of strong layer and soft clay. Hence, in the present study, a single geogrid layer was initially placed at the interface ($u = 0.67D$). For a single layer of reinforcement, the results do not show any substantial improvement. This may be due to the fact that the depth of the geogrid layer of reinforcement is too deep ($u = 0.67D$) and it does not work efficiently. Lateral soil shear is believed to have occurred above the geogrid layer of reinforcement. King et al. (1993) also found similar findings that the magnitudes of the bearing capacity with and without the presence of geogrid at the granular fill–clay interface were practically the same. Hence, in the second set of Series III tests, additional geogrid layer was placed at a depth of 0.17D from the footing base. Thus, it was aimed to examine the effect of 1 versus 2 reinforcement geogrid layer. Fig. 12 shows the loading pressure–settlement curves for all the tests performed in Series I, II, and III, respectively. As seen that the most effective location of the top geogrid layer should be close to the footing base. Alternatively, additional geogrid layer should be placed within the compacted granular fill layer. The results in Fig. 12 clearly show that a further improvement is achieved by placing second geogrid layer within the compacted granular fill. The beneficial effect of a double layer of geogrids, one at the

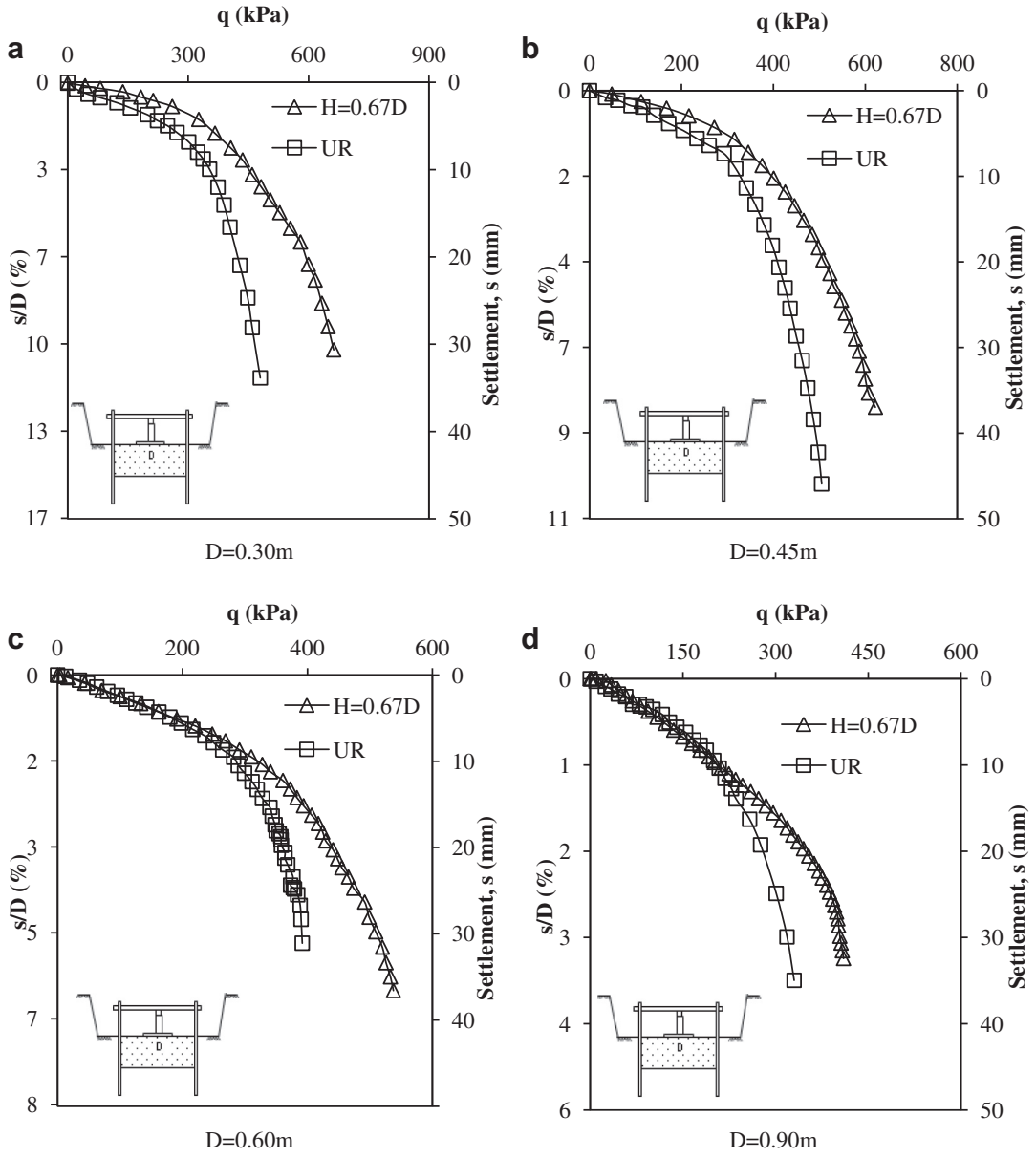


Fig. 11. Effect of granular fill in Series II.

interface and the other at 0.17*D* depth, is much greater than that of a single layer as far as settlements are concerned.

The values of bearing capacity and subgrade modulus were calculated for each test at settlements of δ_{10}, δ_{20} and δ_{30} presented in Table 4. The bearing capacity ratios calculated based on Tables 3 and 4 have been plotted against the footing settlements of δ_{10}, δ_{20} and δ_{30} in Fig. 13. It is also seen in Fig. 13 that a single layer of reinforcement at the interface of the compacted granular fill and

Table 3
Comparison of values of bearing capacity and subgrade modulus for Series I and II.

Test conditions	Footing diameter, *D*(m)	Bearing capacity, *q* (kPa)			Subgrade modulus, *k* (MN/m^3)		
		$s = 10$ mm, (δ_{10})	$s = 20$ mm, (δ_{20})	$s = 30$ mm, (δ_{30})	$s = 10$ mm, (δ_{10})	$s = 20$ mm, (δ_{20})	$s = 30$ mm, (δ_{30})
UR	0.30	350	420	460	35.0	21.0	15.3
	0.45	330	420	460	33.0	21.0	15.3
	0.60	290	360	395	29.0	18.0	13.2
	0.90	205	280	320	20.5	14.0	10.7
$H/D = 0.67$	0.30	450	595	660	45.0	29.8	22.0
	0.45	400	510	585	40.0	25.5	19.5
	0.60	320	440	510	32.0	22.0	17.0
	0.90	210	350	405	21.0	17.5	13.5

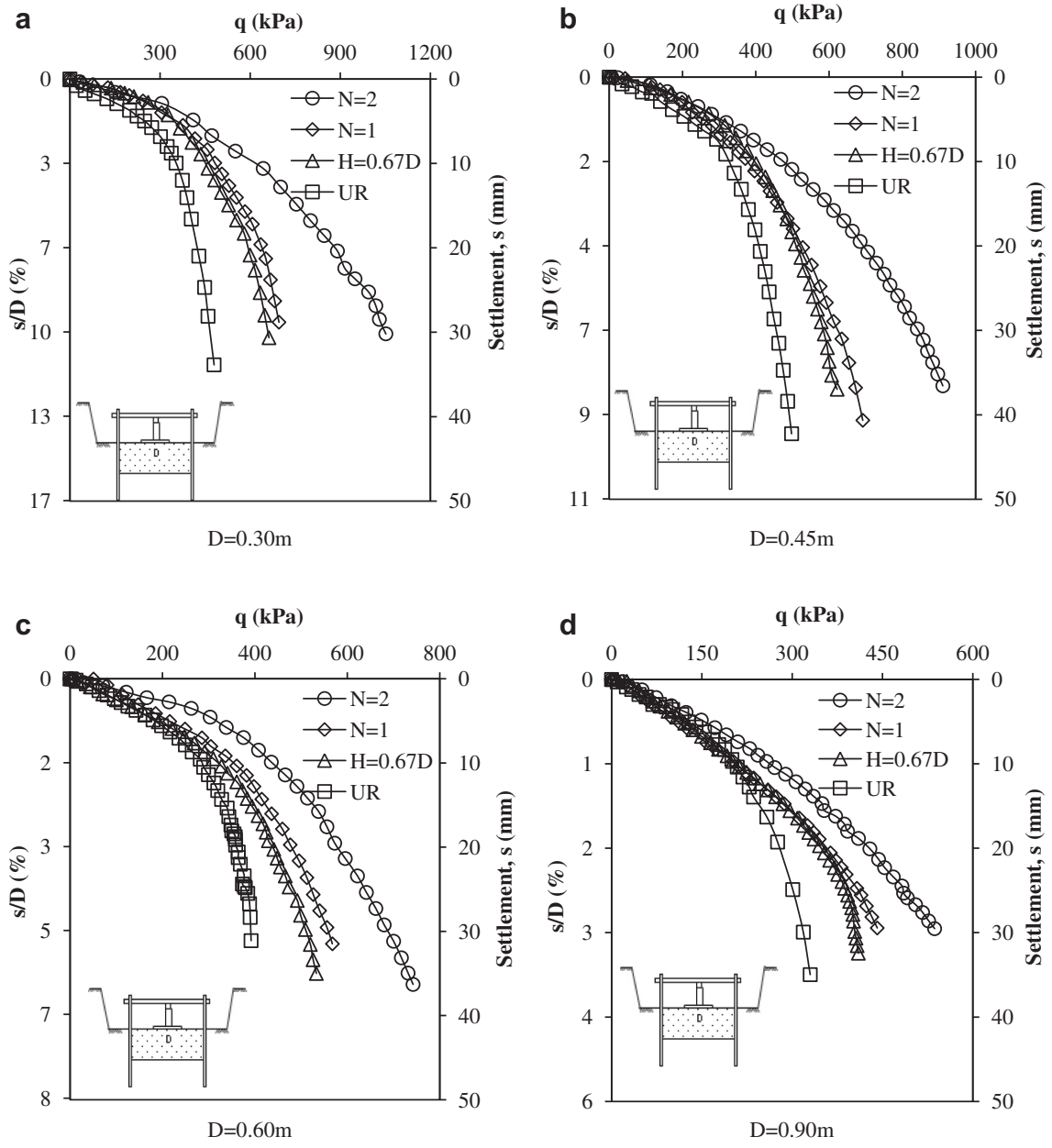


Fig. 12. Effect of reinforcement in Series III.

natural clay does not bring further improvement on BCR values. However, the values of BCR clearly increase with two layers of geogrid reinforcement. It is seen that the initial part of the load–settlement curves practically are the same at low settlement levels less than δ_{10}. However, the test curves are started to diverge from that of unreinforced test at the large settlements. The curves obtained for double layer of geogrids are observed to diverge from the other results in especially the final range. The important improvement in bearing capacity is observed at settlements larger than δ_{10}. The compacted granular fill with two geogrid layers improved the bearing capacity of the footings by approximately 130% (i.e., $1.7 \leq \mathrm{BCR} \leq 2.3$) at large settlements ($\delta_{30}$) while the bearing capacity is improved by approximately 70% (i.e., $1.3 \leq \mathrm{BCR} \leq 1.7$) at low settlements ($\delta_{10}$). The reason for this is that the geogrid layers are mobilized at large strains and that relatively large strains are required to mobilize the reinforcement layers. This improved performance can be attributed to an increase in shear strength in the reinforced granular fill from the inclusion of double geogrid reinforcement since the transfer of footing loads to greater depths through the geogrid layers and interlock between the geogrid and the granular fill reduce lateral and vertical displacements below the footing. The interaction between the geogrid and the soil is dominated by the interlocking of grain particles within the geogrid cells (Huang and Tatsuoka, 1990; Michalowski, 2004). So, the soil-geogrid system forms a composite material that inhibits development of the soil-failure wedge beneath footings. Fig. 12 shows also the beneficial effects of two geogrid reinforcement by the change in the shape of the load-settlement curve. This may be due to the fact that failure mechanism changes from local failure in compacted granular fill to a punching mechanism which appears gradually an increasing without catastropic consequences.

Table 4
Comparison of values of bearing capacity and subgrade modulus for Series I and III.

Test conditions	Footing diameter, D(m)	Bearing capacity, q (kPa)			Subgrade modulus, k (MN/m^3)		
		$s = 10$ mm, (δ_{10})	$s = 20$ mm, (δ_{20})	$s = 30$ mm, (δ_{30})	$s = 10$ mm, (δ_{10})	$s = 20$ mm, (δ_{20})	$s = 30$ mm, (δ_{30})
UR	0.30	350	420	460	35.0	21.0	15.3
	0.45	330	420	460	33.0	21.0	15.3
	0.60	290	360	395	29.0	18.0	13.2
	0.90	205	280	320	20.5	14.0	10.7
$N = 1$	0.30	480	640	700	48.0	32.0	23.3
	0.45	400	530	645	40.0	26.5	21.5
	0.60	355	475	555	35.5	23.8	18.5
	0.90	215	360	440	21.5	18.0	14.7
$N = 2$	0.30	620	880	1040	62.0	44.0	34.7
	0.45	480	700	840	48.0	35.0	28.0
	0.60	440	580	690	44.0	29.0	23.0
	0.90	260	430	540	26.0	21.5	18.0

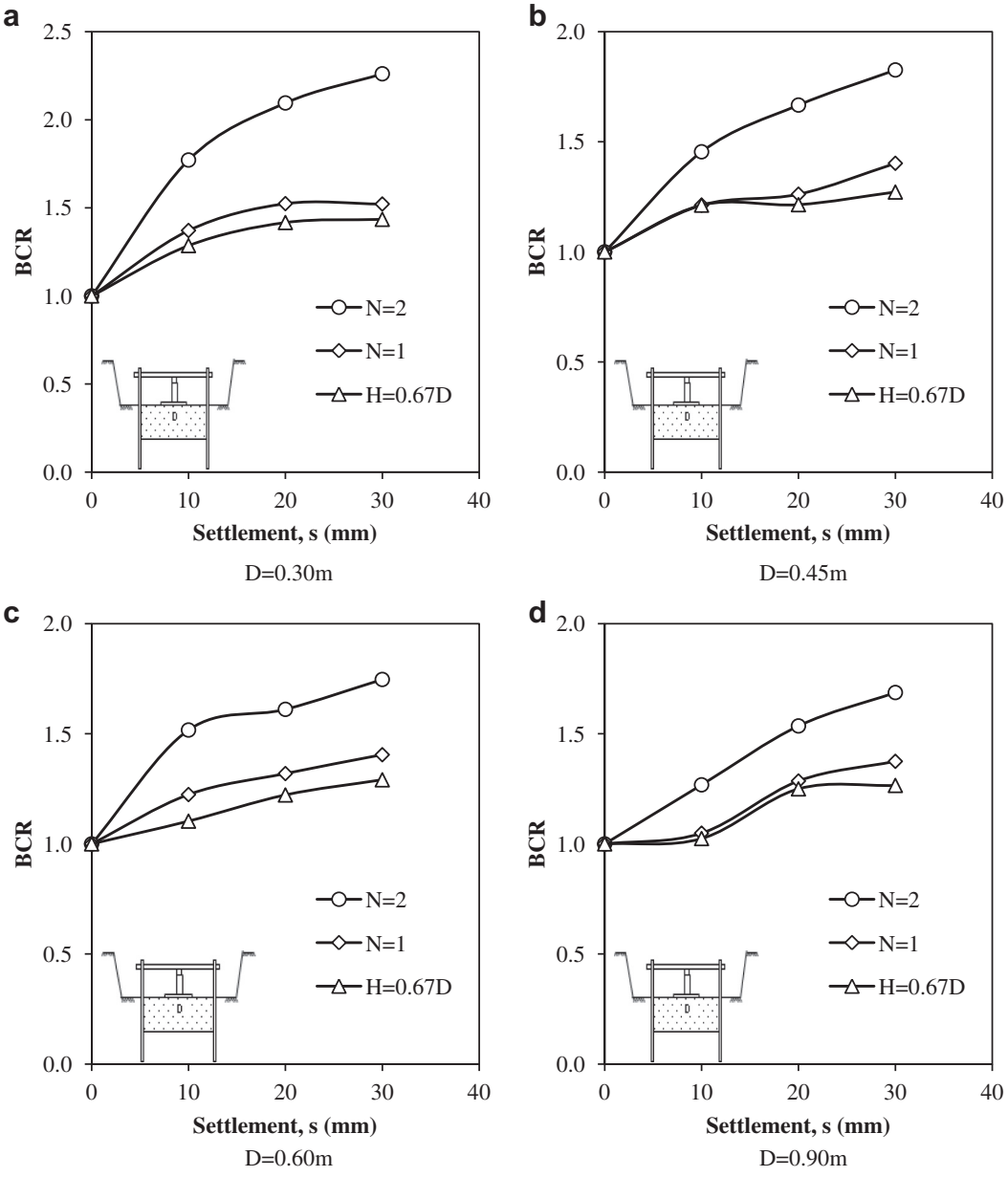


Fig. 13. Comparison of bearing capacity with and without reinforcement.

The relations of the subgrade modulus obtained from Series I, II and III are presented in Fig. 14 for footing settlement of δ_{10}, δ_{20} and δ_{30}. The values of subgrade modulus decreases generally with increasing footing settlement for all series. As shown in Fig. 14 that placing of double geogrid layer in the granular fill has the most influence on the subgrade modulus for the same footing settlement. So, the highest soil subgrade modulus, k was obtained in the second set of Series III which the average k was about 45% greater than that of unreinforced case for footing settlement of δ_{10}. The results from elastic theory show that the increase in vertical stress in the soil below a circular footing of diameter D is about 30% of the contact pressure at a depth of D. The depth is commonly referred to as a critical zone within the stress bulb (DeMerchant et al., 2002). Because of the presence of reinforcement in the critical zone, the settlement decreases and corresponding subgrade modulus values increase. Similar results have also been found by DeMerchant et al. (2002) where it was determined that all layers of geogrid should be placed within the critical depth equal to the width of the footing.

The values of k vary according to the size of footing used in field tests as shown in Fig. 15. Thus, k has no unique value and depends on the size of loaded area. As seen that it decreases with increasing the size of footing. As a result, the values of k usually recommended in literature (e.g. Bowles, 1996) should be used carefully.

Fig. 16 also shows the variation of the PRS factors in relation to diameter of footing, D for reinforcement cases at a specified footing pressure ($q = 300$ kPa). For a single layer of reinforcement, the results do not show any additional improvement in footing settlement as mentioned previously. However, the compacted granular fill with two geogrid layers reduced the settlement of the footings by approximately 50% (i.e., $42\% \leq \text{PRS} \leq 49\%$).

5.5. Comparison of the field test results with analytical solutions

The bearing capacity ratio (BCR) is used to compare the results of field tests and the predicted values from the analytical solutions for Series II. The problem has been considered a double-layer

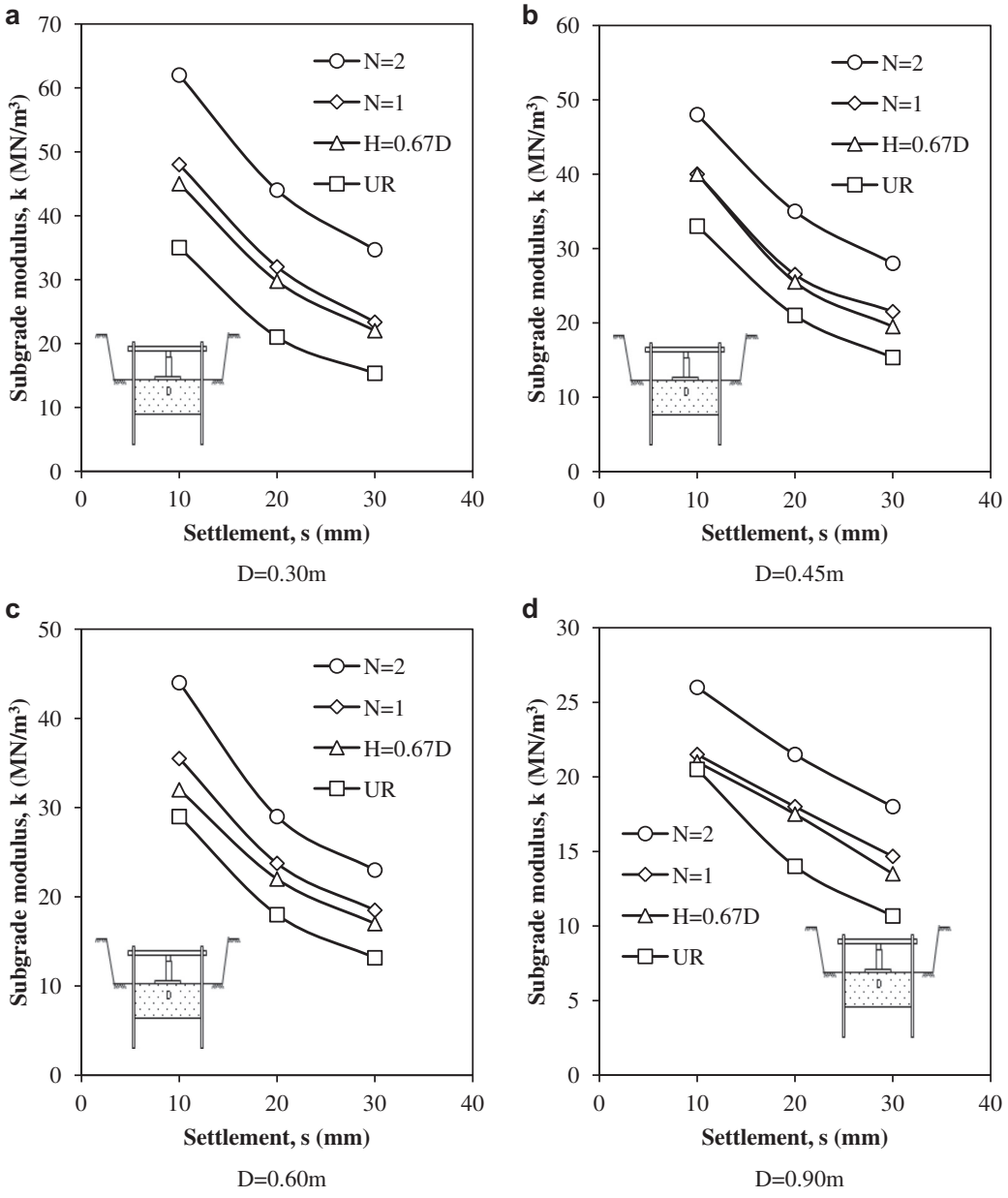


Fig. 14. Comparison of subgrade modulus with and without reinforcement.

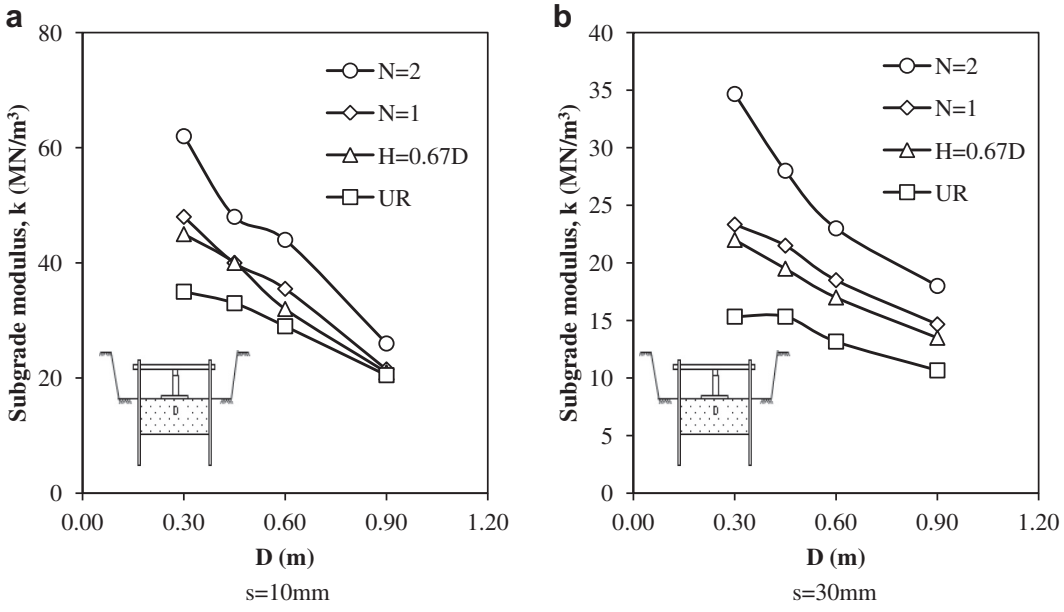


Fig. 15. Subgrade modulus versus footing size for Series I, II and III.

footing system in which the upper layer has been artificially compacted by using granular fill. The bearing capacity ratios for the double-layer footing system were calculated based on aforementioned analytical solutions (Meyerhof and Hanna, 1978; Michalowski and Shi, 1995; Love et al., 1987). Fig. 17 presents the comparison of the results of field test (typically selected $D = 0.45$ m) and the analytical solutions. As seen, the experimental results are consistent with analytical results proposed by different researchers.

Another widely used approach to estimate the bearing capacity of granular fill layers overlying clay is load spread models where is assumed that the granular fill acts to spread the load beneath the footing and that the foundation fails when bearing capacity failures occur within the clay. Love et al. (1987) suggested that the pressure load on the footing may be calculated as the bearing pressure $N_c \times c_u$ over the with B' and so, the pressure on the footing may then be calculated as;

$$q = N_c \times c_u \frac{B'}{B} \tag{4}$$

This procedure is clearly appropriate for cases where the strength of fill layer is substantially greater than that of the clay. The load spread mechanism within the fill layer may be modeled relatively simply by assuming that the vertical stress associated with the footing load is confined to a zone defined by lines inclined at an angle β to vertical, as shown in Fig. 18. Load from the footing is assumed to be distributed uniformly over a width B' at the base of

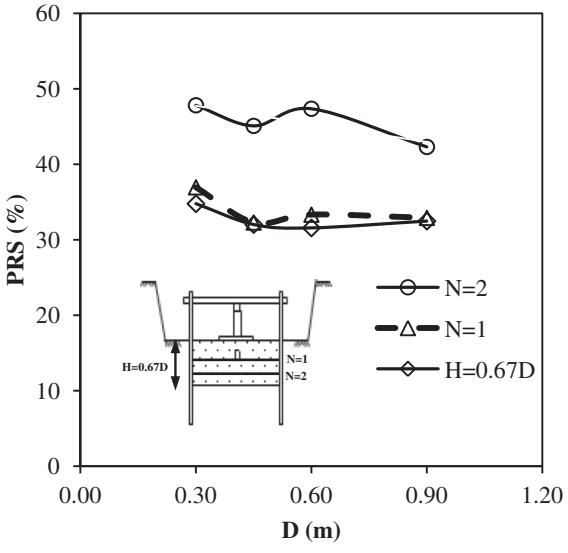


Fig. 16. Percentage reduction in footing settlement RSF for all sizes in tests Series II and III.

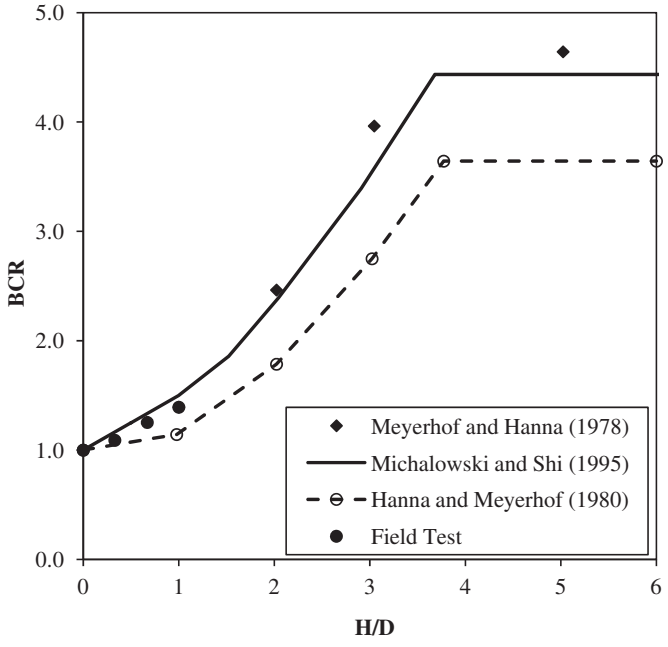


Fig. 17. Comparison of results of field test and different studies.

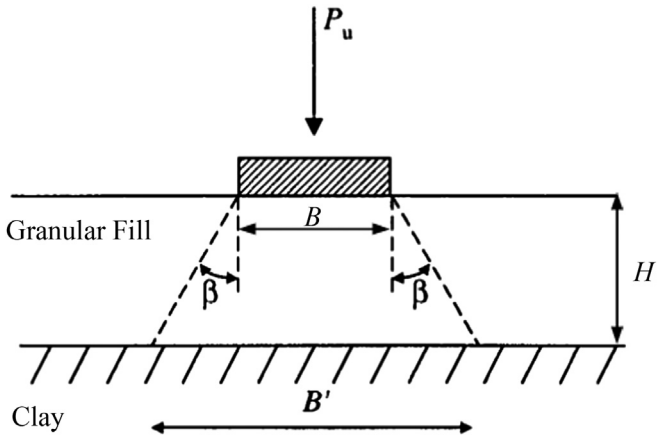


Fig. 18. Load spread mechanism.

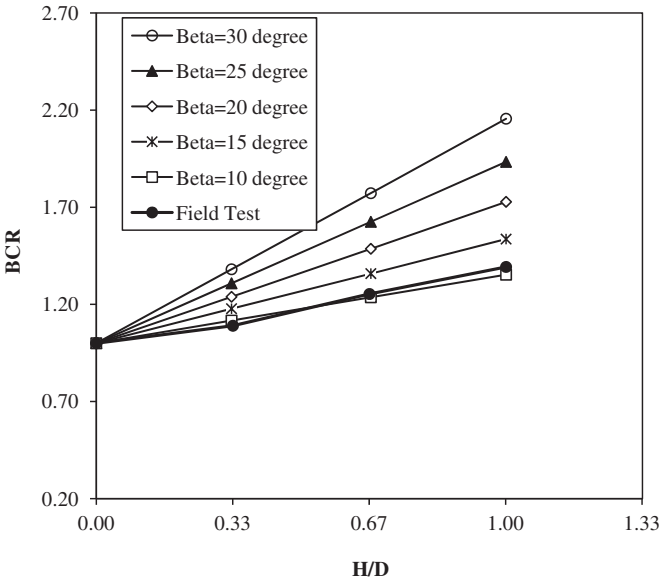


Fig. 19. BCR-H/D relations for $D = 0.45$ m.

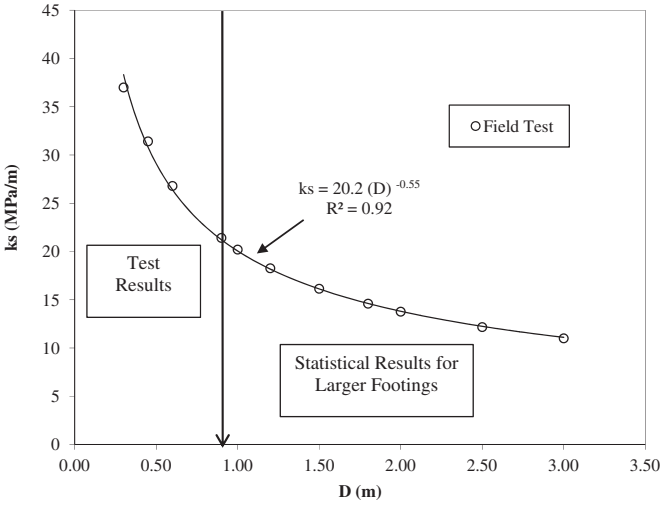


Fig. 20. The experimental results and statistical equation for Test Series I.

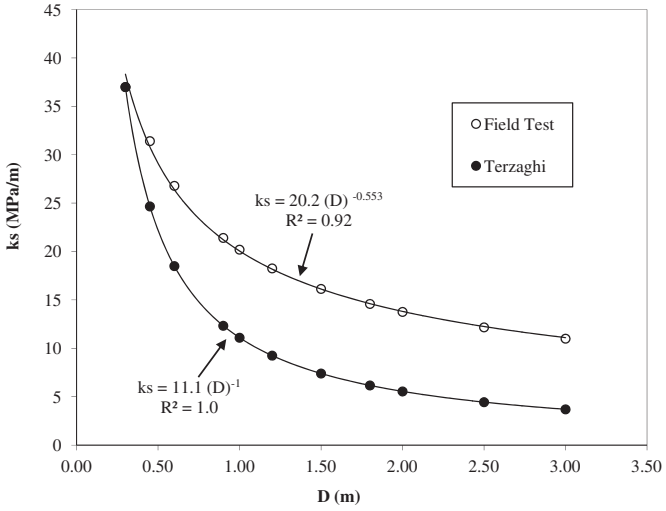


Fig. 21. Comparisons of the results of field test and Terzaghi's Equation for Test Series I.

the fill layer where, $B' = B + 2D\tan\beta$. Although the chosen value of β can have an important influence on the calculated bearing capacity, it is often not clear how its value should be selected (Burd and Frydman, 1997). Brocklehurst (1993) showed that the value of β is strongly influenced by strength of the clay. Lyons and Fannin (2006) pointed out that load spread was observed to vary significantly with granular fill thickness ($20° \leq \beta \leq 40°$); the higher values occurring at greater thickness.

The results obtained from the model footings ($D = 0.45$ m) on the granular fill layer over natural clay were compared with the results predicted by Eq. (4) in Fig. 19. As seen from the figure that the best fits with the experimental results have been obtained at $\beta = 10$ degree.

5.6. Statistical analysis of the subgrade modulus

The plate load test (PLT) which is done with 0.30–1.00 m diameter circular plate is generally known as a direct method to estimate the subgrade modulus. Terzaghi (1955) proposed that *ks*, for full sized footings, could be obtained from PLT using the following equations:

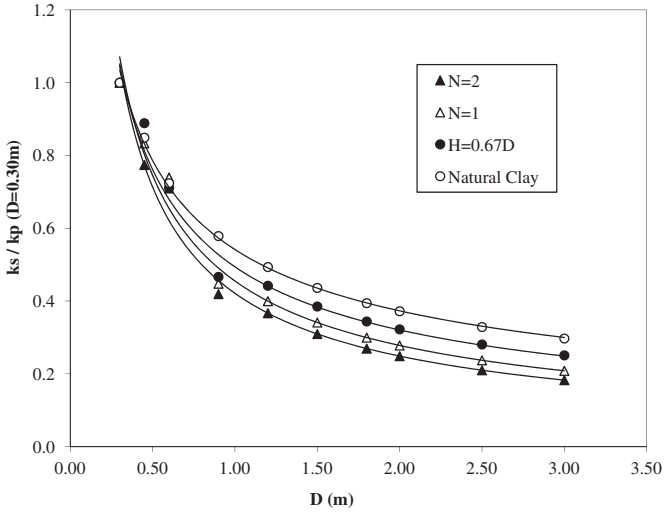


Fig. 22. Relationship between diameter of footing (D) and normalized subgrade modulus for Test Series I, II and III.

Table 5
Expressions of subgrade modulus for different cases.

	Natural clay (Series I)	Stabilized with granular fill ($H = 0.67D$, Series II)	Reinforced with geogrid (for $N = 1$, Series III)	Reinforced with geogrid (for $N = 2$, Series III)
kp (Mpa/m)	37.0	45.0	48.0	62.0
Approximate expression	$ks = kp\left[\frac{0.3}{D}\right]^{0.52}$	$ks = kp\left[\frac{0.3}{D}\right]^{0.57}$	$ks = kp\left[\frac{0.3}{D}\right]^{0.60}$	$ks = kp\left[\frac{0.3}{D}\right]^{0.70}$

For clayey soils;

$$ks = kp\frac{Bp}{B} \tag{5}$$

For sandy soils;

$$ks = kp\left(\frac{B + Bp}{2B}\right) \tag{6}$$

where Bp is the plate diameter (or side dimension of the square plate) used in the PLT to produce kp (the value of ks for bearing plate) and B – side dimension of full sized footing. In this study, the values of subgrade modulus for double-layer footing system have been obtained by using statistical analysis and verified with field test results. Firstly, the values of subgrade modulus were extended with the statistical analysis using the data of field test to large size footings (after $D = 0.90$ m of diameter) for Test Series I. The results of statistical analysis on relationship between the diameter of footing (D) and ks showed that the following power function provides the best fit for correlation between plate load test data (ks) and diameter of footing (D) (Fig. 20).

$$ks = 20.20 \times (D)^{-0.55} \tag{7}$$

The values of subgrade modulus were also extended with existing empirical equations (for clay soil proposed by Terzaghi) using the test data from 0.305 m diameter plate to large size footings. The results of statistical analysis and empirical equations were compared to determine the correlation for large size footings in Fig. 21.

Based on the results of the field tests, the modulus of subgrade (ks) is decreased as the diameter of plate increased. However the values of both methods (Terzaghi equation and statistical analysis) showed that the results of Terzaghi method are lower than statistical results. The results of statistical analysis on relationship between diameter of footing (D) and the results of ks showed that the following power function provides the best fit for correlation between field test data of ks and diameter of footing (D). The statistical correlation between modulus of subgrade reaction (ks) and diameter of footing (D) is obtained as follows for Test Series I;

$$ks = 1.82 \times kp \times \left(\frac{Bp}{B}\right)^{0.55} \tag{8}$$

However, there is no direct method to calculate subgrade modulus of double-layer foundation system. Some statistical relations are suggested based on the experimental results for these systems (with geogrid reinforced upper layer). The values of subgrade modulus have been determined based on the test results conducted on natural clay stabilized with and without granular fill and reinforced granular fill layer. Fig. 22 illustrates the relationship between the subgrade modulus and footing diameter based on a reference diameter of 0.30 m for different cases (Series I, II and III). Table 5 shows approximate expressions obtained for different conditions ($H = 0.67D$, $N = 2$) by field Test Series. It should be noted that these equations cannot be used for all ground conditions and must be adjusted depending on the ground conditions and geogrid configurations.

6. Limitations of research program

While this study is one of the large-scale load test program on geogrid reinforced circular footings to date, 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. Footing sizes of up to 0.90 m were tested with depth of reinforcement to footing width ratios between 0.17 and 0.67. 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 findings obtained from this study will be of use for designing and constructing, for storage tanks founded on soft soils, large stabilized areas for parking of vehicles etc. The investigation is considered to have provided a useful basis for further research leading to an increased understanding of the bearing capacity and subgrade modulus variation of circular footings resting on geogrid reinforced granular fill over clay soils.

7. Conclusions

A series of large scale field tests was undertaken to investigate the load-settlement behavior of circular footings over natural clay soil and to compare this behavior with that of comparable reinforced granular fill (without geogrid) and reinforcement with a single or two layers of geogrid in the granular fill, including analytical and statistical approaches. Based on the results from this investigation, the following main conclusions can be drawn:

- The results show that the compacted granular fill layer increases the load-bearing capacity of the footing and decreases the settlement.
- When natural clay deposits are replaced with stiffer granular fill ($H = 0.67D$), the bearing capacity increases about 40%. The results do not show any additional improvement on bearing capacity by placing a layer of geogrid at the interface of the natural clay and the compacted granular fill. A further improvement on bearing capacity can be obtained by placing second geogrid layer within the compacted granular fill. The compacted granular fill with two geogrid layers improved the bearing capacity of the footings by approximately 130% at large settlements while the bearing capacity at low settlements is improved by approximately 70%. The results show that the geogrid layers are mobilized at large strains.
- The values of subgrade modulus also increase significantly using two geogrid layers of reinforcement within the compacted granular fill layer. The improvement on the subgrade modulus varies according to the footing settlement. The results show that subgrade modulus decreases with increasing footing settlement. It has also no unique value and depends on the size of loaded area. The results show that subgrade modulus decreases as the footing diameter increases.
- Footing settlements can be reduced by placing a compacted granular fill layer of limited thickness over the weak clay

subsoil. The reduction in footing settlement is approximately 35% comparing with unimproved case. A further reduction can be obtained by placing two layers of geogrid within the granular fill. Reduction in footing settlement is approximately 50%. The reinforced granular fill layer results in redistribution of the applied load to a wider area and thus minimizing stress concentration and achieving improved distribution of induced stress.

- Geogrid reinforcement provides an economic advantage which reduces the thickness of the fill layer and limits the amount of natural clay to be removed.
- Based on field test results of 305 mm diameter plate to large size footings, statistical equations were obtained for subgrade modulus of double-layer system. There is no method to find out subgrade modulus of reinforced layered soils. An approximate method has been suggested to compute the subgrade modulus of circular footing on reinforced layered soil. The results of this study can be used for preliminary estimates of subgrade modulus of reinforced layered soils when geogrids are used as reinforcement within the granular fill.

Acknowledgements

The work presented in this paper was carried out with funding from TUBITAK (The Scientific and Technological Research Council of Turkey) grant number 106M496.

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