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RESEARCH ARTICLE

A Modern Approach to Estimate the Bearing Capacity of Layered Soil

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Abtsract

This study is concerned with the bearing capacity of circular footings on a granular fill layer above a soft clay soil. The results of an extensive series of laboratory and field tests were used to define an empirical equation. This is generally done by estimating the dependent variable (e.g. bearing capacity) based on the independent variables (e.g. granular fill layer thickness, soil and footing parameters and settlement ratio). A logarithmic model has been developed by using regression analysis to estimate the bearing capacity of a circular footing resting on granular fill at any settlement ratio, using all possible regression techniques based on 342 field test data, to select the significant subset of the predictors. The results indicate that the logarithmic model serves a simple and reliable tool to predict the bearing capacity of circular footings placed on a granular fill with different thicknesses above a soft clay soil. And also, the validity of the developed formulation was verified with different plate load test results from literature.

Keywords

bearing capacity, layered soil, regression analysis, field tests, laboratory model tests, circular footing

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1 Introduction

The ultimate bearing capacity of strip footings in homogeneous soils is generally estimated by geotechnical engineers with the bearing capacity equation proposed first by Terzaghi [1] and modified later by Meyerhof [2] and Brinch Hansen [3]. However, natural soil profiles are often layered and/or heterogeneous. Two cases are generally considered for two layer clay soils: a) weak clay overlain by relatively stronger clay, and b) stronger layer overlain by relatively weaker clay. In the former case, it is generally assumed that bearing capacity failure occurs in the upper weak clay. In the second case, the problem is usually analysed as a punching failure through the upper strong clay and as a general shear failure in the lower weak clay.

Several experimental and numerical studies have been described in the literature about the reinforcement of a weak soft soil (Ochiai et al. [4], Otani et al. [5], Yamamoto and Kusuda [6], Ismail, K. M. H. I. [7], Verma, S. K. et al. [8], Thome et al. [9], Mosadegh A. & Nikraz H. [10], Ziaie Moayed, R. et al. [11], Ornek et al [12], and Calik and Sadoglu [13]. Ochiai et al. [4] investigated the theory and the applications of reinforced fill over soft ground in Japan. Otani et al. [5] studied the behavior of strip foundation constructed on reinforced clay. Settlement was found to be reduced with the increase in reinforcement size, stiffness and number of layers. The layout of reinforcement closer to each other caused an increase in carrying capacity. Yamamoto and Kusuda [6] employed approximate solutions for bearing capacity of reinforced soil using the upper-bound theorem of limit analysis. Microscopic observation results of the failure mechanisms of reinforced and unreinforced soils were examined. The method was developed from the failure mechanism results by using upper bound theorem and it was compared with the experimental results. It was found that the upper-bound calculation is an effective way for the evaluation of bearing capacity and the prediction of failure mechanism. Ismail, K. M. H. I. [7] compared the results of the numerical analysis and the field plate loading observations of the circular footing resting on granular soil overlying soft clay. It is demonstrated that the ultimate bearing capacity is directly proportional to the angle of internal friction of granular soil ϕ, the granular layer thickness H, and the foundation depth D, while at the same time it is inversely proportional to the footing diameter B. Plate loading tests have been conducted by Verma, S. K. et al. [8] in a large tank to observe the load settlement behavior of plates of different sizes resting on layered granular soils. Tests were conducted on fine gravel layer overlain sand layer using mild steel plates of square shapes. The effect of the layer placement on the bearing capacity and settlement characteristics of footing has been studied and an equation for predicting the bearing capacity of two layered granular soils is developed based on the plate load test data. Thome et al. [9] proposed a semi-empirical approach from finite element results, for the cemented fill layer above weak soil ground. This method was compared with the field plate loading test results. The results for different base diameters and the deposit thicknesses have shown that this approach can be used. Mosadegh A. & Nikraz H. [10] examined the bearing capacity of a strip footing on one-layer and two-layer soils by using ABAQUS. For a layered, soft- over-strong soil, the effect of layer thickness, soil shear strength and material property on bearing capacity value and failure mechanism have been studied. It was concluded that, the bearing capacity of footing decreases as the height of clayey soil increases whilst the displacement under footing increases. However, the stronger bottom layer have not been effected the ultimate bearing capacity and displacement value of footing after some thickness of clayey soil on top. Ziaie, R. M. et al. [11] performed a finite element analysis to study the bearing capacity of ring footings on a two layered soil. The effects of two factors, the clay layer thickness and the ratio of internal radius (r_i) of the ring footing to external radius (r_0) of the ring, have been analyzed. It was found that, the bearing capacity decreases as the value of r_i/r_0 increases.

In the present study an attempt has been done to develop a regression model on the basis of the actual data obtained from the field tests, for estimating the bearing capacity of circular footing on granular fill layer above a soft clay soil to any *s*/*D* ratio. To the best of the author's knowledge, no attempts so far have been made to estimate the bearing capacity of a circular footing resting on granular fill layer above a soft clay soil, by means of a regression analysis.

2 Materials Used

Soft clay and granular soils were used for the experimental investigations.

2.1 Soft Clay

The soft soil material used in this research was locally available clay soil from the west part of Adana, Turkey. The soil conditions at the experimental test site were determined from a geotechnical site investigation comprising both field and laboratory tests. Two test pit excavations (TP1 and TP2) and four borehole drillings (BH1, BH2, BH3 and BH4) were performed in the water treatment metropolitan municipality test area. The plan view and the locations of the piles in test area are shown in Fig. 1. The main purpose of the piles are to use them as a reaction piles to reach some large load values as in the plate loading test. Standard Penetration Tests (SPT) were carried out during the drilling of each borehole, and the distribution of SPT values with depth is shown in Fig. 2.

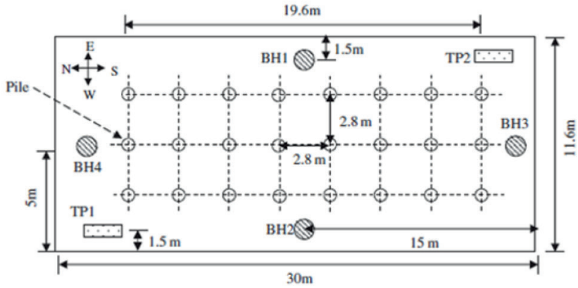

Fig. 1 Plan View Showing Piles, Borings and Test Pits (Ornek et al. [12])

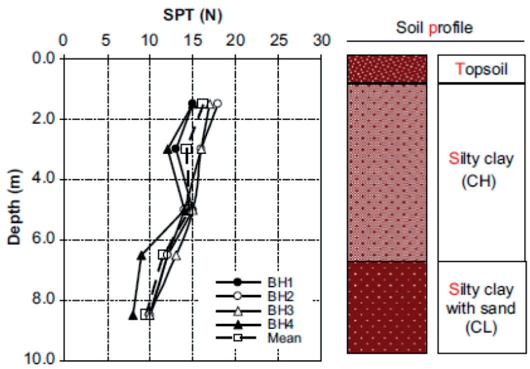

Fig. 2 SPT (N) Values Measured From Boreholes (Ornek et al. [14])

After conducting required conventional laboratory tests (sieve and hydrometer, moisture content, unit weight, liquid and plastic limit, unconfined compressive strength), the soil was prepared for model tests. The soil was identified as high plasticity inorganic clay, CH, according to the unified soil classification system (USCS). The values of liquid limit, plastic limit and plasticity index of soft soil were obtained as 53%, 22% and 31%, respectively. The water content of the stratified soil layers varied between 20% and 25%, depending on depth, which was almost the same as, or greater than, the plastic limit. The value of specific gravity of clay soil was found to be 2.60. The average cohesion values of clay soil were 40 kN/m^2 and 75 kN/m^2 for laboratory model and field tests, respectively (Demir et al. [15]). The soft clay soil properties are shown in Table 1.

Table 1 Clayey Soil Properties

Depth	0-1 m	1-2.2 m	2.2-3.5 m	3.5-5.0 m
Soil Type	Topsoil	CH	CL	CL
w (%)	-	0-21	22-24	22-24
γ_n (kN/m3)	-	19.5-20.5	19.3-22.5	20.8-21.5
γ_s (kN/m3)	-	25.7-26.0	26.0-26.9	25.7-26.6
w_L (%)	-	51-69	28-54	37-44
w_P (%)	-	21-30	19-22	20-25
c_u (%)		60-80	65-75	-

2.2 Granular Soil

The granular fill material used in the experimental studies was obtained from Kabasakal region situated northwest of Adana, Turkey. Some conventional laboratory tests (sieve, moisture content, unit weight, direct shear and proctor) were conducted for this material. All the conventional test results are tabulated in Table 2.

Table 2 Granular Soil Properties

Soil Type	w (%)	γ_{kmax} (kN/m³)	γ_s (kN/m³)	ϕ (°)	c_u (kN/m²)
GW-GM	7.0	21.7	26.4	42.0	15.0

According to the sieve analysis, granular soil was classified as well graded gravel-silty gravel, GW-GM according to the USCS (Fig. 3).

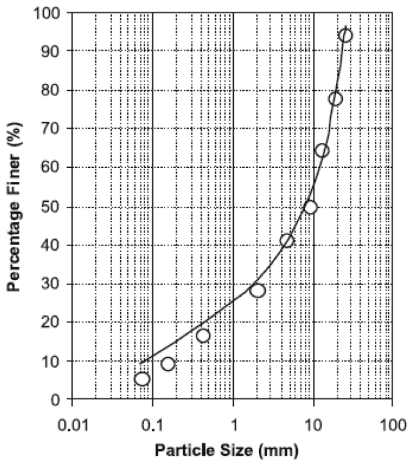

Fig. 3 Grain Size Curve for Granular Fill (Misir [16])

For all test groups, granular soil was prepared at a value of optimum moisture content of 7% and a maximum dry unit weight of 21.7 kN/m³ obtained from the standard proctor test (Fig. 4). To maintain the desired density of the soil in the test area, the same compaction procedure was applied to each granular fill layer.

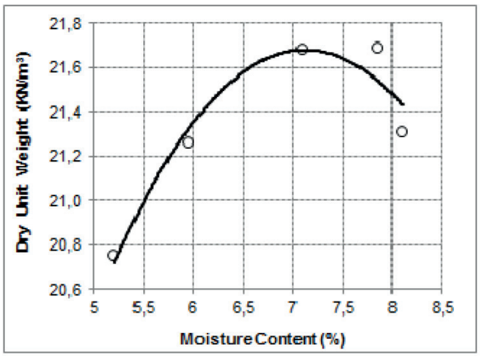

Fig. 4 Standard Proctor Curve for Granular Fill (Misir [16])

The values of internal friction angle and the cohesion of granular fill were obtained as 43° and 15 kN/m², respectively from direct shear test (Fig. 5). Specific gravity of the granular soil was obtained as 2.64 (Demir et al. [15]).

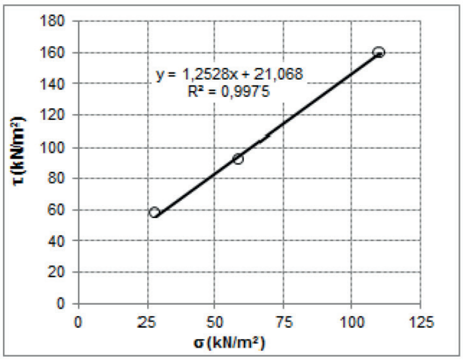

Fig. 5 Direct Shear Test Curve for Granular Fill (Misir [16])

3 Field Tests

3.1 Preparation of Granular Fill Layer

The test programme contains 21 field tests for circular rigid footings, with diameters of 6, 9, 12, 30, 45, 60, 90 cm. In the field experiments, the wooden box was used to form the granular fill layer. Box sizes were selected from the numerical analysis for all the footings which was performed previously to eliminate the boundary effects due to the loading. The total fill layer thicknesses were determined as a multiple of the footing diameter. To obtain the desired granular fill thickness the fill material was placed in layers above the natural clay soil. Each granular fill layer thicknesses was 20 mm for 6, 9, 12 cm footings, and it was 50 mm, for 30, 45, 60, 90 cm footings on natural soft clay. For each granular fill layer, the amount of soil needed was calculated. The fill material was prepared by using tiller to mix the pre-weighted soil and water. For each layer the needed soil poured and was compacted using with an electrical plate compactor, to the predetermined height to achieve the desired densities. Compaction procedure was used throughout the testing program in order to obtain a reasonably homogeneous soil. After preparation of each granular fill layer was completed, the height of each layer was controlled.

3.2 Test Procedure

A total of 21 field tests were conducted on silty clay soil. Reinforced concrete reaction piles were installed on each side of the test footing and connected to each other by means of a 4 m length I–240 steel beam (Laman et al. [17]). A hydraulic jack and two displacement transducers (LVDT) were connected to a data logger and it was connected to a computer, to measure the applied load and settlement of the footing. The circular model footings used in the field tests had diameters of 6 cm, 9 cm and 12 cm with 2 cm thickness and also had diameters of 30 cm, 45 cm 60 cm and 90 cm with 3 cm thickness. The footing was loaded with a hydraulic jack supported against the reaction frame (Laman et al. [17]). Depending on the diameter of the foundations, different capacities of hydraulic jacks were used (Fig. 6). The tests were performed according to the ASTM D 1196-93 (ASTM [18]).

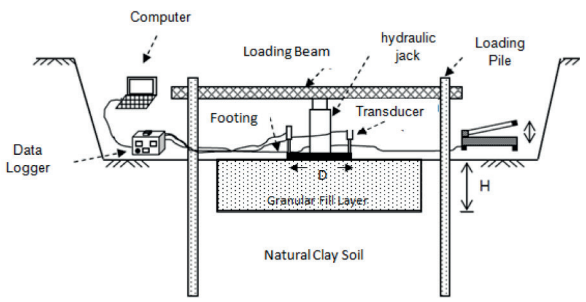

Fig. 6 Schematic View of The Experimental Set-Up (Laman et al. [17])

A load cell was placed between the jack and the footing to measure the applied load. Settlements were measured by using 5 cm capacity two transducers which were placed at the bottom of the foundation plate base. The average value of these two readings, were obtained as the value of settlement (Fig. 7–8).

Fig. 7 Field Test Set-up for D = 30, 45, 60, 90 cm (Laman et al. [17])

Fig. 8 Field Test Set-up for D = 6, 9, 12 cm (Laman et al. [17])

3.3 Test Variables

Field tests were carried out for three different granular fill layer thicknesses. Seven different footing diameters (D) were used for the same granular thickness. The field tests were conducted under three series and symbolized by IM. Parameters considered in these series are footing diameters (D), and granular fill layer thicknesses (H/D) (Table 3).

Table 3 Test Variables for Field Tests

Test Series	Constant Parameters (H/D)	Variable Parameters (D) cm
IM–D–0.33	0.33	6, 9, 12, 30, 45, 60, 90
IM–D–0.67	0.67	6, 9, 12, 30, 45, 60, 90
IM–D–1.00	1.00	6, 9, 12, 30, 45, 60, 90

4 Laboratory Model Tests

As in the field tests, in order to establish the effect of footing diameters and granular fill layer thickness on the bearing capacity of circular footing, laboratory tests were carried out. Experimental setup and preparation procedure of granular fill layer above the natural clay soil were moreless the same as in the field test. Some differencies are reported below.

4.1 Preparation of Granular Fill Layer

A total of 8 laboratory tests were carried out using circular rigid footings with diameters of 6 and 9 cm. The granular material was prepared by mixing the pre-weighted soil and water according to the compaction test results. The granular fill layer was placed on natural soft clay in layers of approximately 25 mm thicknesses. As in the field tests before to compact the granular fill in layers, the amount of soil needed for each lift was calculated first. Then the granular soil, was poured, and compacted using with an electrical plate compactor, to the predetermined height to achieve the desired densities. After preparation of each granular fill layer was completed, the height and the densities of each layer was controlled for a homogeneous fill.

4.2 Test Procedure

A cylindrical test box having 38 cm diameter and 42 cm height, was used in the laboratory tests. This rigid test box is made of steel and has a wall thickness of 5 mm. The inside walls of test box, was polished smooth, to reduce friction with the soil. For the model tests, soft clay soil was kept in an oven for 24 h at a temperature of 105 + 5Co and it was then sieved passing through B. S. sieve No 10 (2.00 mm). The clay soil was thoroughly mixed by hand and placed at the predetermined soil unit weight, into the test box. The remolded clay was placed into the bin in layers of 2.5 cm thickness. The soil in the test box was compacted, by a special hammer to give standard compaction energy. The test box was filled in a similar way to get enough height for each test. The granular material was compressed, as in the field experiments with an electrical plate compactor (Demir et al. [15]). A square steel plate with a width of 50 cm was placed under this heavy model box and firmly clamped using two long pins to prevent any movements during the tests. A model circular footing with a hole at its top center made of mild steel was used to transfer the load to the center of the foundation with a steel ball. Model circular foundations, with diameters of 6 and 9 cm and with thickness of 2 cm were used. The foundation was positioned at the center of the top soil layers, before the tests (Misir [16]). Such an arrangement produced a hinge, which allowed the foundation to rotate freely as it approached failure and eliminated any potential moment transfer from the loading fixture (El Sawwaf [19]).

4.3 Test Variables

Laboratory tests were carried out for two different foundation diameters (*D*). For two constant model footing diameters, the granular fill layer thicknesses (*H*/*D*) were varied between 0.25*D* and 1.50*D* (Table 4). The tests for remolded clay, is symbolized by LM.

Table 4 Test Variables for Laboratory Tests

Test Series	Constant Parameters (D) cm	Variable Parameters (H/D)
LM–6–H/D	6	0.25, 0.50, 1.00, 1.50
LM–9–H/D	9	0.25, 0.50, 1.00, 1.50

5 Statistical Analysis

Empirical estimation methods are often used in many Engineering applications including Geotechnical Engineering (Chen [20], Davarci, B. et al. [21], Adunoye and Agbede [22], Latha et al. [23], Jha et al. [24]). Regression analysis is a statistical methodology used to examine the relationship between a dependent variable and a set of independent variables. Correlation and regression analysis are related in the sense that both deal with relationships among variables. Neither regression nor correlation analyses can be interpreted as establishing cause-and-effect relationships. The correlation coefficient (*R*) measures only the degree of linear association between two variables. In fact, $R^2 = R*R$ (and sometimes known as the coefficient of determination) is used as measure of the quality of the regression.

The method which is used in this study, is preferred as a similarity model and can be adopted for load displacement relationship because the independent variables used are explicit and the dimensionless variables are physically bounded.

As in the regression analysis, it is generally done by estimating the dependent variable (e.g. bearing capacity) based on the independent variables (e.g. settlement ratio (*s*/*D*) and thickness of the granular fill layer (*H*/*D*)). In the experimental studies, granular fill layer with different thicknesses (*H*) was located beneath the foundation (Fig. 3). The aim of carrying out these tests is to analyze the contribution of granular fill on the bearing capacity of soft clay soil. In the tests, granular fill thickness was changed depending on the foundation diameters.

As in any prediction problem, the selection of input variables is very important. For that reason, only the essential variables which have significant effect on the behavior should be selected (Uncuoglu [25]). 6, 9, 60 and 90 cm diameter circular foundations were used in the field experiment to derive the statistical approach. In the analysis, three different granular fill layer thicknesses were used for each of the four different foundation diameters (Fig. 9). As seen from the graphs, the relationship between the bearing capacity (*q*) and settlement ratios (*s*/*D*) for all the curves is fairly linear for small-load ranges, and that the relationship is nonlinear for large-load ranges and does not exhibit any peak values. Also, from a comparison of the curves for different *H*/*D* values, it can be seen that the load–settlement behavior became stiffer as the *H*/*D* ratio increased, due to partially replacing the natural clay soil with a layer of compacted stiffer granular fill, for both $D = 0.06$ m and 0.90 m footing diameters. In these series, the bearing capacity is a function of *H*/*D* (Ornek at al [12]).

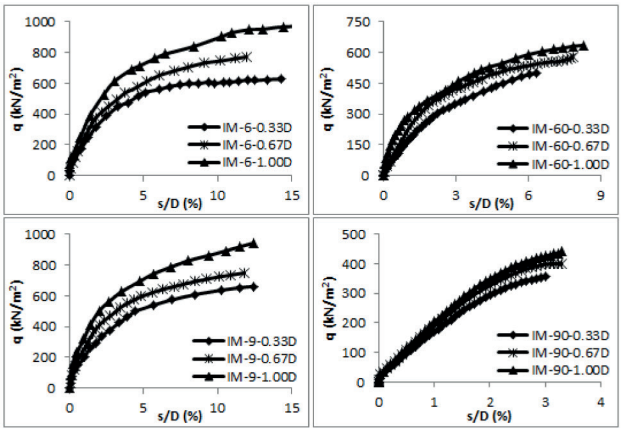

Fig. 9 The Field Test Results Used to Obtain The Formulation

Failure loads are more significant for footings with smaller diameters however, this behavior is not observed clearly for larger diameter. The reason for this is, load handling system was forced and inaccessibility of greater deformation rates. In this study a statistical formulation was developed with these test curves to estimate the bearing capacity of soft clay stabilized with granular fill layer. For this aim, in order to create a formulation in terms of dimensionless parameters for bearing capacity, a theoretical formula is needed. This suggests, a means of extrapolating the results of a circular footings between certain limits, to a design process for model or full-scale footings on the same granular upper material and lower soft foundation material (Thome et al. [9]).

In layered soils, to predict the value of bearing capacity according to the upper stiff layer, is not a realistic way. Therefore, Onalp and Sert [26] has proposed the following formula to reflect this behavior Eq (1).

$$q_{theoric} = \left[1 + 2\frac{H}{D}\tan\alpha_c\right]\left(c_u N_c + \gamma' D_f + \gamma' H_f\right) \tag{1}$$
$$+\left[\frac{K_p \sin(\phi' - \alpha_c)}{\cos\phi' * \cos\alpha_c}\right]\left(\frac{H}{D}\right)\left(\gamma' D_f + \gamma' H_f\right) - \gamma' H\left[1 + 2\frac{H}{D}\tan\alpha_c\right]$$

The expressions in the bearing capacity formula Eq (1) are given in Eqs (2–6).

$$\alpha_c = \tan^{-1}\left[\frac{(\sigma_{mc}/c_u) - (\sigma_{ms}/c_u) * \left(1 + \sin^2\phi'\right)}{\cos\phi' * \sin\phi' * (\sigma_{ms}/c_u) + 1}\right] \tag{2}$$

$$\sigma_{mc}/c_u = N_c c_u \left(1 + \frac{1}{\lambda_c}\frac{H}{D}\frac{\lambda_p}{\lambda_c}\right) \tag{3}$$

$$\sigma_{ms}/c_u = \frac{\sigma_{ms}/c_u - \sqrt{(\sigma_{ms}/c_u)^2 - \cos^2\phi'\left((\sigma_{ms}/c_u)^2 + 1\right)}}{\cos^2\phi'} \tag{4}$$

$$\lambda_p = \frac{\gamma' D_f}{\gamma' D} \tag{5}$$

$$\lambda_c = \frac{c_u N_c}{\gamma' D} \tag{6}$$

This formulation contains foundation, soil and fill layer parameters, to reach ultimate bearing capacity for the desired granular fill layer thickness and equivalent foundation diameter.

However, most of the developed theoretical approaches are independent from settlement ratio. Therefore only the bearing capacity values can be obtained but load-settlement behavior cannot be assessed as it was the case in the experimental studies. The formulation was obtained from 342 data points in Fig. 9 by plotting q values against *s*/*D* values. The settlement ratio *s*/*D* is defined as the ratio of footing settlement s to footing diameter D. Many formulations were tested and the best equation that fits the load deformation behavior, occurred in a non-linear logarithmic equation from as seen in Eq. (8).

$$q/q_{theoric} = a * \ln(x) + b \tag{7}$$

$$q/q_{theoric} = \left[\left(0.074 * \left(\frac{H}{D}\right)^2 - 0.135 * \left(\frac{H}{D}\right) + 0.339\right)\right]\ln\left(\frac{s}{D}\right)$$
$$+\left[0.108 * \left(\frac{H}{D}\right)^2 - 0.217 * \left(\frac{H}{D}\right) + 0.502\right] \tag{8}$$

In generally Eq. (8) can be used both to obtain the value of ultimate bearing capacity at a constant settlement ratio (in general the most common settlement ratios (*s*/*D*) are 3%, 5% or 10% etc. to obtain the ultimate bearing capacities in the literature.) and the behavior of the load displacement relationship at a constant granular fill layer thicknesses.

Therefore, Eq (8) can be used for both the laboratory and field tests for layered soil conditions in a certain limits as in the experiments. The parameters of granular and soft clay soilsmust be used to calculate the value of theoretical bearing capacity (qtheoric). In this study, although the procedure of the remolded clay soil constitution has been generated in a reasonable way, the effect of the bond forces between the clay particles for the natural and remolded clay soils are different from each other and this effect occurs by the value of cohesion. The usage of the theoretical bearing capacity formulation by actual soil parameters, makes the bearing capacity estimation possible for the experimental studies considered in this work by using a single formulation Eq. (8). Although Eq. (8) was derived by using the limited data of the field experiments, by taking into account the parameter of the cohesion, the formulation was used to verify and to validate the laboratory test results.

Regression analysis is a technique used to estimate values that are unknown with the known values. It is important to know the shape and the degree of the functional relationship between variables. The value of correlation coefficient, indicates the degree of reliability, for the estimated values. The results produced high coefficient of determination (R^2) for Eq. (8) to be 0.898 (Fig. 10).

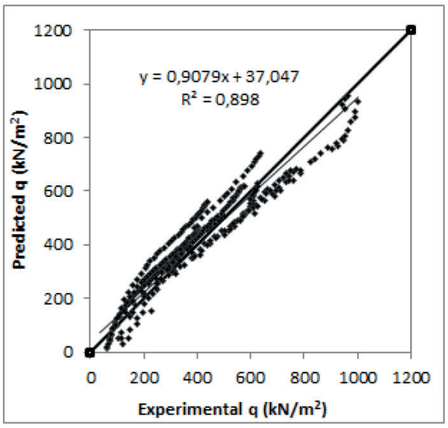

Fig. 10 Correlation of the Experimental and Predicted Results for Formulation Data

6 Results and Discussion

6.1 Comparison of the Field Test Results with Those Obtained from Eq. (8)

The purpose of this study, is to find a relationship between settlement ratio *s*/*D*, and bearing capacity q, depending on the thickness of granular fill layer together with different footing diameters placed on the soft clay soil. Some more field tests were carried out for different footing diameters (D = 12 cm, 30 cm, 45 cm). The results of these tests were not used to obtain Eq. (8). When Eq. (8) was applied to the geometries of these tests, the graphs of q versus *s*/*D* were obtained. Figures 11, 12 and 13 shows clearly that the values by using the proposed analytical solution is in very good agreement with test results for all three different footing diameters and granular fill layer thicknesses.

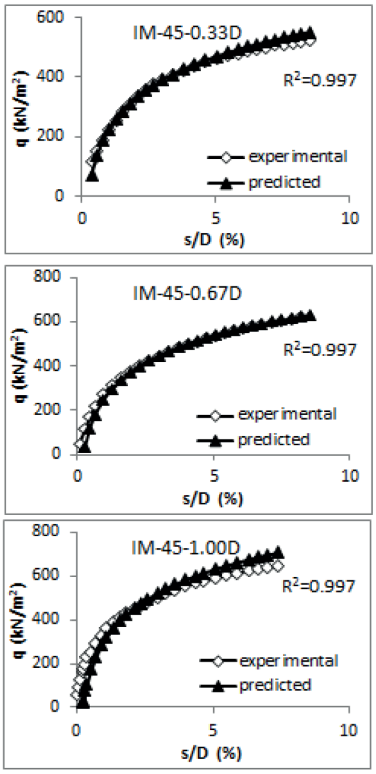

Fig. 11 Comparison of Experimental and Eq. (8) Results for D = 45 cm New Field Tests

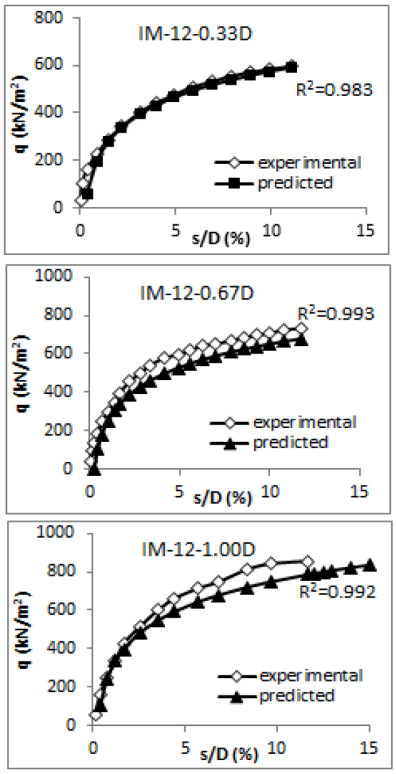

Fig. 13 Comparison of Experimental and Eq. (8) Results for D = 12 cm New Field Tests

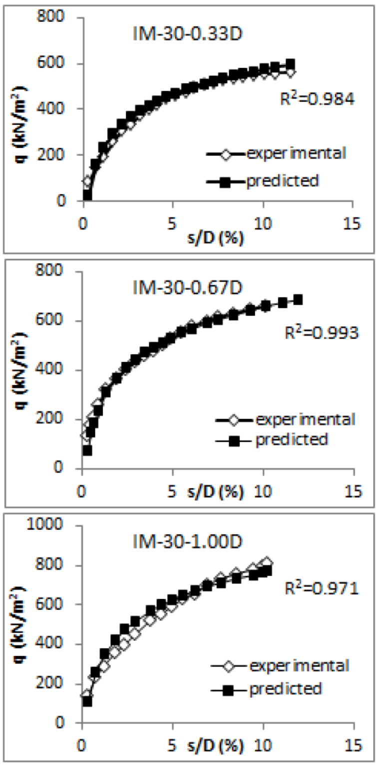

Fig. 12 Comparison of Experimental and Eq (8) Results for D = 30 cm New Field Tests

As seen from the results that, the q-s/D relationship predicted by using Eq. (8) has shown a similar non-linear behavior with experimental results. Also, the ultimate bearing capacity obtained by using Eq. (8), gives similar results obtained from field tests as shown in Figs. 11–13.

Fig. 14 presents the measured bearing capacities against the predicted bearing capacities by the network model with R^2 coefficients for the input and the output values, respectively. The linear 1:1 line was also plotted in these figures to discuss the performance of the statistical models. It is seen from the figure that by using Eq. (8), the location points of the experimental and the predicted bearing capacity values are scattered around the 1:1 line for both input and output phases (Fig. 14).

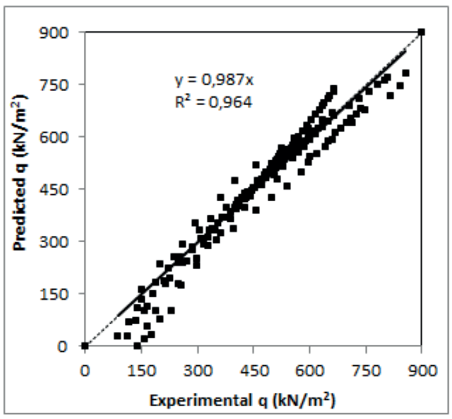

Fig. 14 Correlation of the Experimental and Predicted Results for New Field Tests

In engineering practice foundations are always designed at a limited settlement level because of serviceability requirements. The field tests and the formulation results are summarized in Table 5. In this table, the bearing capacities (q_u) obtained at a settlement ratio of s/D = 3% are presented. The error between the experimental results and the estimated bearing capacity values by using Eq. (8) was calculated, by the mean absolute percentage error method Eq. (9). The mean absolute percentage error (MAPE), is a measure of accuracy of a method for constructing fitted time series values in statistics, specifically in trend estimation. It usually expresses accuracy as a percentage, and is defined by the following formula:

$$MAPE = 100 * \frac{\sum_{i=1}^{n} \frac{|A_i - F_i|}{A_i}}{n} \qquad (9)$$

where A_i is the actual value and F_i is the forecast value. In the proposed model, the value of mean absolute percentage error varies within the range of 0.02% and 13.05% and the formulation estimates 184 field experiment data, with an error rate of 5.47% on average.

Table 5 Comparison of Bearing Capacities Obtained from Eq. (8) and Field Test Results for s/D = 3%

Test series	H/D	D (cm)	Experimental q_u (kN/m^2)	Predicted q_u (kN/m^2)	Mape (%)
IM-12-0.33D	0.33		403.3	396.6	1.7
IM-12-0.67D	0.67	12	511.7	444.9	13.0
IM-12-1.00D	1.00		557.3	513.5	7.8
IM-30-0.33D	0.33		377.6	397.4	5.3
IM-30-0.67D	0.67	30	435.7	444.5	2.0
IM-30-1.00D	1.00		456.1	519.1	13.8
IM-45-0.33D	0.33		389.8	388.7	0.3
IM-45-0.67D	0.67	45	446.0	446.1	0.02
IM-45-1.00D	1.00		499.3	525.7	5.3
The Mean Absolute Error:					**5.4**

In this section, regression analysis of the results of field tests which were not used to obtain the formula were shown satisfactory results with the results of the formula. In the field tests due to the limitations in the experimental assembly, it was not possible to load more than a certain value. The large-scale experiments were terminated at settlement rate of 3% due to the limitation of loading mechanism. Considering the curve obtained from the formula showed that the bearing capacity values for the values of 3%, is also acceptable.

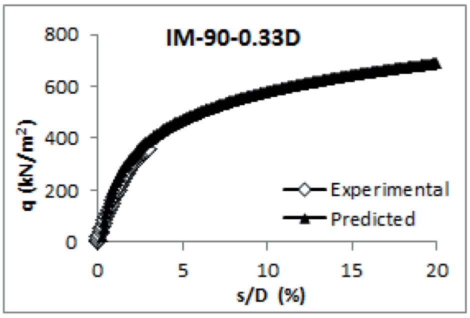

Fig. 15 The Estimation of the Curve of q versus s/D for the Remaining Part of the Test

So, the experimental curve of non-linear behavior of load-deformation relationship q–s/D couldn't be fully achieved and the ultimate bearing capacity values could not be obtained clearly (Fig. 15) for these diameters. At this point the formulation was used to produce the full form of the curve which was not reached in the experiments. The load deformation curve obtained from Eq. (8) continued after the termination point of the experimental curve. As a result, the curve obtained from the formula is highly significant and represents the actual behavior of the soil until the point of changing the inclination of the curve and the formulation is in compliance with the experimental results up to this point. It is shown that, the soil behavior and the failure load can be obtained easily by using the formula derived in this study without any extra experimental work.

6.2 Comparison of the Laboratory Test Results with Those Obtained from Eq. (8)

As in the field tests, in order to test Eq. (8) for laboratory tests, for different footing diameters and granular fill material thicknesses which were not considered while obtaining Eq. (8), the results of these tests were compared with the results obtained by using Eq. (8) with the parameters of the tests (Fig.16).

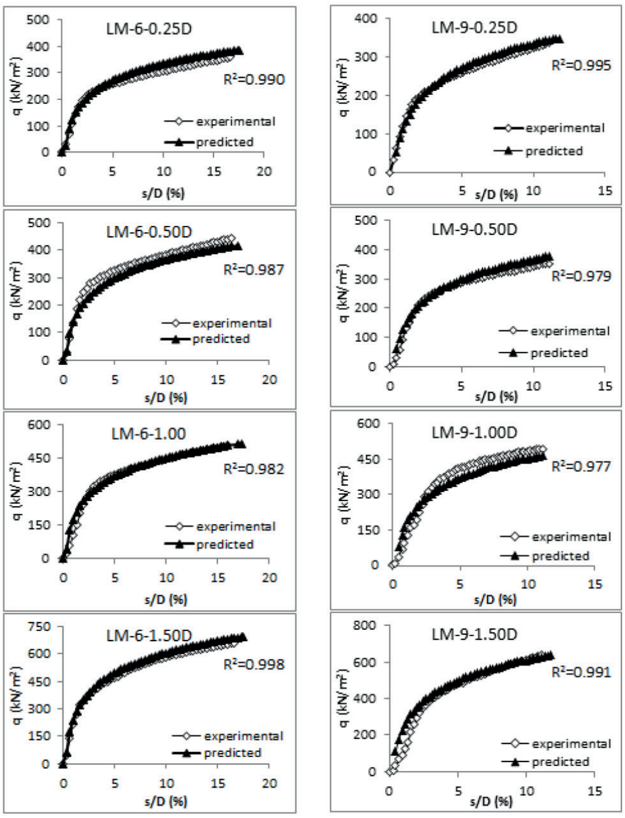

Fig. 16 Comparison of Experimental and Eq. (8) Results for Laboratory Model Tests

Fig. 16 shows clearly that the values obtained by using the proposed analytical solution are again in very good agreement with the laboratory test results. For each laboratory test, the determination coefficient between the experimental and the predicted ultimate bearing capacities are given in Fig. 17. All the laboratory test results were evaluated together; the average coefficient of determination, as shown in Fig. 17 was obtained as 0.971.

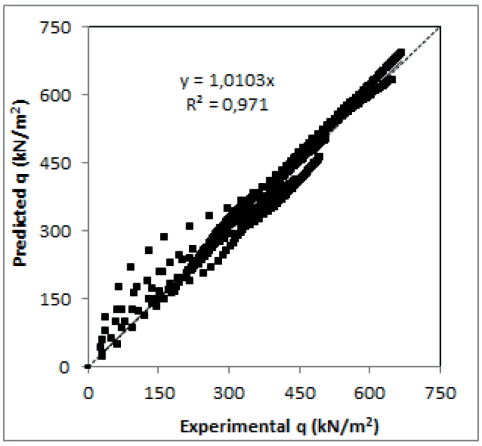

Fig. 17 Correlation of the Experimental and Predicted Results for Laboratory Model Tests

The bearing capacity of laboratory test results and those obtained from Eq. (8) are summarized in Table 6. In this table, the bearing capacities obtained at settlement ratio of s/D = 3% are presented. Bearing capacity values increase with an increase in thickness of granular fill. In the proposed model, the value of mean absolute percentage error varies within the range of 1.19 and 13.57. The formulation was estimated 403 laboratory experiment data, with an error rate of 5.15% on average. Also as seen from Table 6, when the thickness of the granular fill layer for the laboratory tests were increased from 0.25D to 1.50D, a significant increment of the bearing capacity have been observed up to 76%.

Table 6 Comparison of Bearing Capacities Obtained from Eq. (8) and Laboratory Test Results for s/D=3%

Test series	H/D	D (cm)	Experimental q_u (kN/m²)	Predicted q_u (kN/m²)	Mape (%)
LM-6-0.25D	0.25	6	227.5	224.8	1.2
LM-6-0.50D	0.50		285.8	247.0	13.5
LM-6-1.00D	1.00		324.9	307.3	5.4
LM-6-1.50D	1.50		399.0	414.2	3.8
LM-9-0.25D	0.25	9	222.2	224.8	1.2
LM-9-0.50D	0.50		251.2	247.2	1.6
LM-9-1.00D	1.00		336.6	308.3	8.4
LM-9-1.50D	1.50		393.3	416.7	5.9
The Mean Absolute Error:					**5.1**

6.3 Verification of Eq. (8) by Comparing with Literature

In this section, the developed formulation (Eq. 8) was applied to different model test results to investigate the validity. For this purpose, three different experimental studies from the literature were used to compare. One of the aforementioned studies belongs to Biswas et al. [27]. In the study, the researchers investigated the behaviour of geogrid reinforced sand-clay foundation systems, with clay subgrades of different strengths.

Model tests were carried out on a circular footing of 150 mm diameter (D) resting on layered soil. The layered systems were comprised dense sand of varying layer thicknesses (H = 0.63D–2.19D) overlying the clay subgrades of different undrained shear strengths (c_u), ranging from 7 to 60 kPa.

As seen in Figure 18, four different test results with different undrained shear strength and fill layer thicknesses were selected to compare and validate Eq. (8) with the experimental results of Biswas et al. [27].

Three of these tests were performed in different granular fill layer thicknesses (H/D = 0.63, 1.15, 1.67) in cases where the undrained cohesion was 7 kPa. The last graph shows the case of c = 15 kPa and H/D = 0.63.

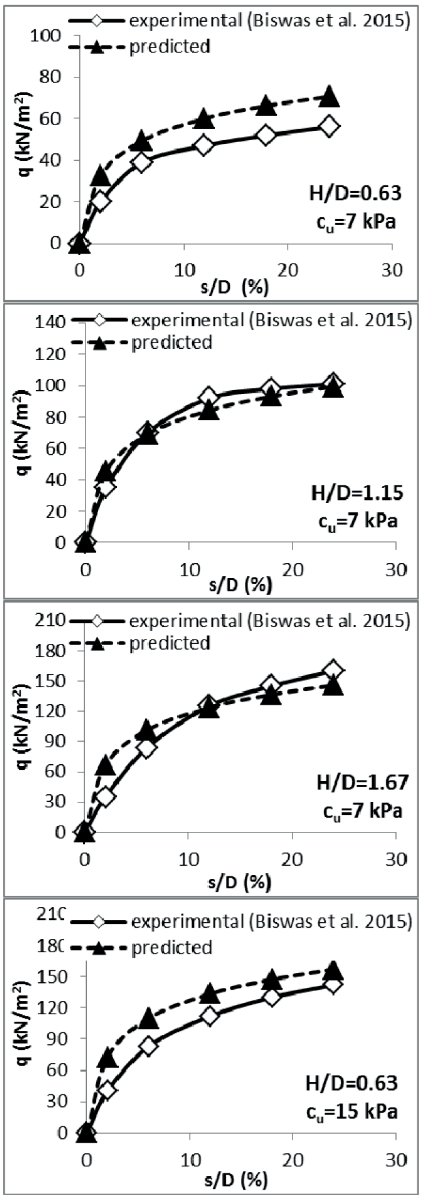

Fig. 18 Comparison of *q–s/D* Relationship Between Eq. (8) and Biswas et al. [27]

The developed formulation was applied to these experimental results. Upon analyzing the *q–s/D* behaviors, both of the graphs, reveal nonlinear behavior as shown in Figure 18. Generally, the predicted pressure values were higher than test results at the same settlement ratios. In case of the lower granular fill layer thicknesses (0.63), the differences between the results of Eq. (8) and Biswas et al. [27] were relatively more. According to minimum curvature point approach, the calculated mean absolute percentage error (MAPE) values at that point were obtained maximum 25%, in case of $H/D = 0.63$ for different cohesion values As a result, increasing the thicknesses of granular fill layer for 1.15 and 1.67, caused a decrease in MAPE values for 1% and 13%, respectively.

The determination coefficient obtained from the Biswas et al. [27] and the predicted ultimate bearing capacities are given in Fig. 19. All the results in Fig 18 were evaluated together; the average coefficient of determination, as shown in Fig. 19 was obtained as 0.916.

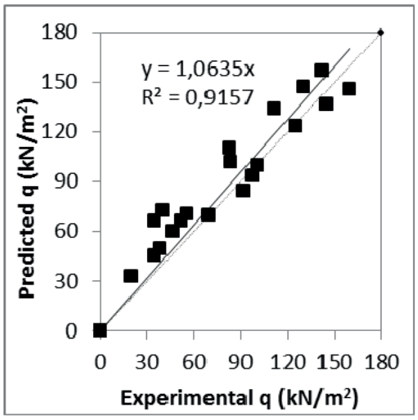

Fig. 19 Correlations of the Eq. (8) and Biswas et al. [27]

The second comparison was done with the results of Dash et al. [28]. In that article, they have studied effectiveness of geocell reinforcement placed in the granular fill overlying soft clay beds by small-scale model tests in the laboratory. Five different series of tests (i.e. A–E) were carried out by varying different parameters such as, unreinforcement, width of geocell layer (*b*) and height of geocell layer (*h*) etc. Under series A, tests were conducted on unreinforced soil beds with different thickness (*H*) of the overlying sand layer.

The model footing used was rigid steel plate and had 150mm diameter (*D*) and 30mm thickness. The sand used in this investigation was a dry sand and the friction angle of the sand at 70% relative density as determined from standard triaxial compression tests is found to be 41^o. The tests were conducted for clay bed in the 3.13 *kPa* undrained shear strength condition.

The layered systems were comprised varying layer of sand thicknesses ($H = 0.52D – 2.62D$) overlying the clay subgrade.

In comparison between the results of Eq [8] and Dash et al. [28] for 0.94 and 1.36 *H/D* values according to limit ranges of this study, the behavior of the load-settlement ratios are in very good agreement as seen in Fig. 20. And also, the average MAPE values were obtained less than 10% at the point of 5% settlement ratio both of the *H/D* ratios.

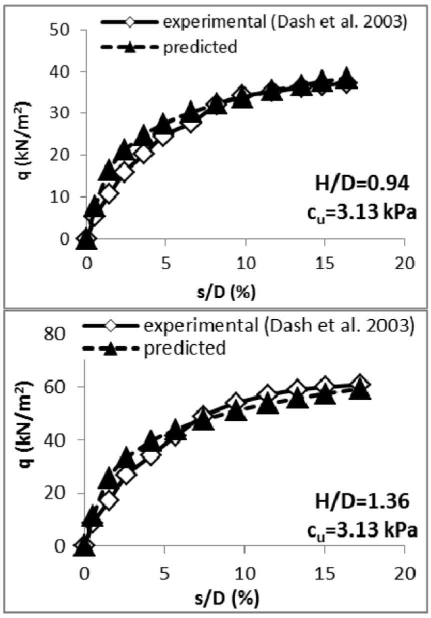

Fig. 20 Comparison of *q–s/D* Relationship Between Eq. (8) and Dash et al. [28]

The determination coefficient between the Dash et al. [28] and the predicted ultimate bearing capacities are given in Fig. 21. All the results in Fig 20 were evaluated together; the $y = x$ line was coincided with the q points. The average coefficient of determination, as shown in Fig. 19 was obtained as 0.956.

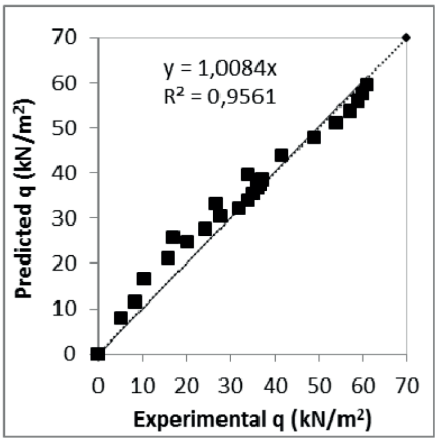

Fig. 21 Correlations of the Eq. (8) and Dash et al. [28]

The last comparison was done with the results of Ibrahim [7]. This paper focuses on variable factors which affect the global bearing capacity such as: granular soil thickness, relative density, foundation depth, footing size, and the extension of granular soil with respect to footing edge. The granular fill layer thickness (H) was tested for 0, B/2, B, 2B and 4B values.

Figure 22 shows the comparison between field observations and predicted results of load settlement curves for a rigid loaded circular plate with a diameter of $B = 0.2$ m and H/D ratios of 0.5 and 1.0. In the comparisons soil medium was in two different density conditions being medium to loose sand with $\phi = 35^o$ and very dense sand with $\phi = 45^o$ overlain by soft clay $c = 21$ kPa.

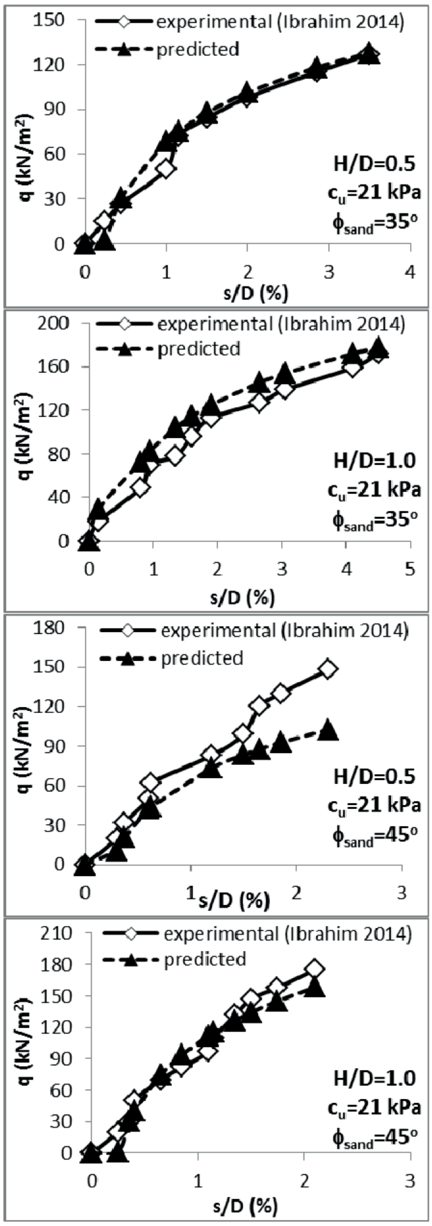

Fig. 22 Comparison of *q–s/D* Relationship Between Eq. (8) and Ibrahim [7]

The behavior of the graphs shows that, both, the results of Eq[8] and Ibrahim [7], were in a non-linear form. In comparison between the results of Eq. (8) and Ibrahim [7] for 0.5 and 1.0 H/D values according to limit ranges in this study, the behavior of the load- settlement ratios are quite close each other.

The average MAPE values in the point of minimum curvature are in the range of 4–10%. The determination coefficient of the data between predicted and Ibrahim [7] was obtained as 0.910.

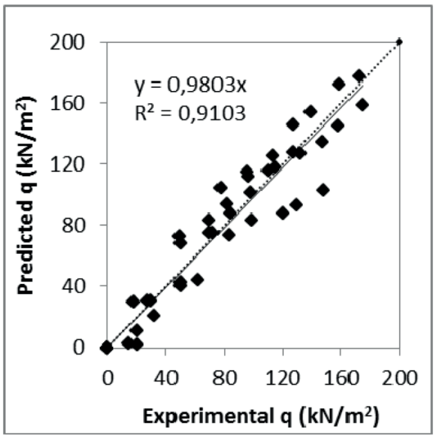

Fig. 23 Correlations of the Eq. (8) and Ibrahim [7]

As mentioned earlier, the developed formulation was verificated successfully, for the results of field tests which were not used to obtain the formula.

Also, from the comparisons of these formulation and the laboratory and field plate load test results from the literature, it can be concluded that the results are in a very good agreement to predict the behavior of convergence by different works in the literature.

7 Limitations

The results reported in the present study are valid only for the subsurface condition at the test location and for the thickness of the granular fill layer used in these tests. The size and scale effects of model foundations have not been investigated. Therefore these findings need additional verification before they can be applied to full-size foundations.

8 Conclusions

The bearing capacity of circular footings on granular fill layer over a soft clay soil was investigated using an empirical estimation method based on physical modelling in the laboratory and at site. In general, the statistical analysis is performed when the experimental cases are difficult and the cost of constructing and monitoring full-scale test embankments is quite high. To be able to this, the statistical model should be required verification and validation using experimental data.

On the basis of analysis of the results obtained from the present investigation, the following conclusions can be drawn:

- The bearing capacity equations of circular footings with partial replacement of granular fill layer as a soil improvement technique were never encountered during the literature review studies.
- From the laboratory and field test results, it is concluded that, the bearing capacity of the circular footing on granular fill layer over soft clay soil was increased up to 78% depending on the granular fill layer thicknesses for the different footing diameters.
- For layered soil conditions investigated in this study, Equation 8 reflects successfully the non-linear behavior of the observed load-deformation relationship, which is based on logarithmic approach.
- Eq. (8) takes into account of granular fill layer thickness, foundation depth, the rate of settlement, the foundation diameter, and the index and engineering parameters for weak ground and backfill material have taken into account by using Equation 1. Comparisons between the results of experimental studies and those of predictions by using Equation 8 are in very good agreement.
- The formulation was derived from 342 field data taken from 12 field tests. In addition to these, 184 field test data from 9 field tests and 403 laboratory model test data from 8 laboratory tests were used to verify Eq. (8). In these comparisons, both of the experimental test groups were obtained with the average error rate of 5%. Thus, the field and laboratory test results were estimated with correlation coefficients better from than 0.97 by using Eq. (8).
- The most important parameter that affects the bearing capacity of remolded soils, is the value of cohesion. Although field tests have been developed using this approach, the approach is quite successful on the results of laboratory tests.
- The statistical method in determining the bearing capacity of layered soil, for a desired settlement ratio in which the stiff soil is above the weak layer, provides realistic results for the parameters considered in this study.
- The distribution of the bearing capacity of layered soil against the settlement ratio s/D, can be estimated reasonably and easily by using Eq. (8).
- Another advantage of the formulation, for the load steps that could not be reached as with IM-90-0.33D for any reason, the form of the curve can be successfully estimated by Eq. (8).
- From the comparisons of these formulation and the laboratory and field plate load test results from the literature, it can be concluded that the results are in a very good agreement to predict the behavior of convergence by different works in the literature.

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