Technical Note

Bearing capacity of embedded strip foundation on geogrid-reinforced sand

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Abstract

Laboratory model test results for the ultimate bearing capacity of a strip foundation supported by multi-layered geogrid-reinforced sand are presented. The depth of embedment of the model foundation, d_f, was varied from zero to B (width of foundation). Only one type of geogrid and one variety of sand at one relative density were used. The ultimate bearing capacity obtained from the model test program has been compared with the theory proposed by Huang and Menq, [1977. Journal of Geotechnical and Geoenvironmental Engineering ASCE 123(1), 30–36]. Based on the present tests, it appears that the theory provides a conservative prediction of the ultimate bearing capacity.

Keywords: Embedment; Geogrid; Sand; Strip; Ultimate bearing capacity

1. Introduction

During the last 20 years or so, results of several studies have been published that relate to the evaluation of the ultimate and allowable bearing capacities of shallow foundations supported by sand reinforced with multi-layered geogrid (e.g., Guido

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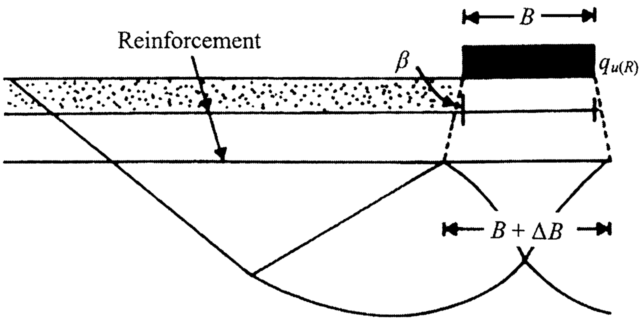

Fig. 1. Wide-slab failure mechanism in reinforced sand supporting a strip foundation. [Note: B = width of foundation; $q_{u(R)}$ = ultimate bearing capacity. For definition of ΔB and β, see Eqs. (9)–(11)].

et al., 1986; Omar et al., 1993; Yetimoglu et al., 1994; Das and Omar, 1994; Khing et al., 1993; Adams and Collin, 1997). All of these studies have been conducted for surface foundation conditions. The effect of the depth of embedment of the foundation, which is the normal situation in all practical cases of construction, has not received proper attention. The purpose of this paper is to report some recent laboratory model test results conducted to evaluate the ultimate bearing capacity of a strip foundation supported by geogrid-reinforced sand. For these tests, the d_f/B value (d_f is the depth of foundation, B is the width of foundation) was varied from zero to one. The experimental results have been compared to the theory developed by Huang and Menq (1997) which is based on the "wide-slab" failure mechanism in soil proposed by Schlosser et al. (1983) as shown in Fig. 1.

2. Geometric parameters

Fig. 2 shows a strip foundation (width B) being supported by sand, which is reinforced with N number of geogrid layers. The vertical spacing between consecutive geogrid layers is h. The top layer of geogrid is located at a depth u measured from the bottom of the foundation. The width of the geogrid reinforcements under the foundation is b. The depth of reinforcement, d, below the bottom of the foundation can be given as

$$d = u + (N - 1)h. \tag{1}$$

The beneficial effect of reinforcement for increasing the ultimate bearing capacity has been generally expressed in the past in terms of a nondimensional quantity called the bearing capacity ratio, BCR or

$$\text{BCR} = \frac{q_{u(R)}}{q_u}, \tag{2}$$

where $q_{u(R)}$ and q_u is the ultimate bearing capacities on reinforced and unreinforced sand, respectively.

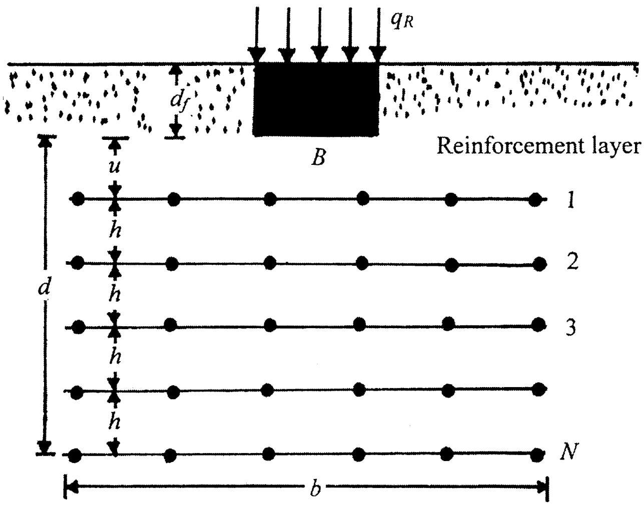

Fig. 2. Shallow strip foundation on geogrid-reinforced sand.

Table 1
Physical properties of the geogrid

Peak tensile strength	60 kN/m
Tensile strength at 2.0% strain	14 kN/m
Tensile strength at 5.0% strain	30 kN/m
Strain at break	8%
Aperture size	94 mm × 42 mm

3. Laboratory model tests

The model foundation used for this study had a width of 80 mm and a length of 360 mm. It was made out of a mild steel plate with a thickness of 25 mm. The bottom of the model foundation was made rough by coating it with glue and then rolling it over sand. Bearing capacity tests were conducted in a box measuring 0.8 m (length) × 0.365 m (width) × 0.7 m (depth). The inside walls of the box and the edges of the model were polished to reduce friction as much as possible. The sides of the box were heavily braced to avoid lateral yielding. Locally available sand dried in an oven was used for the present model tests. The sand used for the tests had 100% passing 1.18 mm size sieve and 0% passing 0.075 mm size sieve. For all tests, the average unit weight and the relative density of compaction were kept at 14.81 kN/m^3 and 71%, respectively. The average peak friction angle φ' of the sand at the test conditions as determined from direct shear tests was 41°. A uniaxial geogrid was used for the present tests. The physical properties of the geogrid are given in Table 1.

In conducting a model test, sand was placed in lifts of 25 mm in the test box. For each lift, the amount of soil required to produce the desired unit weight was weighed

Table 2
Details of model tests

Test series	N	u/B	h/B	b/B	d_f/B
A	0	—	—	—	0–1.0 (unreinforced sand)
B	2, 3, 4	0.35	0.25	5	0
C	4	0.35	0.25	5	0, 0.25, 0.5, 0.75, 1.0

and compacted using a flat bottomed wooden block. Geogrid layers were placed in the sand at desired values of u/B and h/B. The model foundation was placed on the surface as well as at desired depths below the surface of the sand bed. Load to the model foundation was applied through an electrically operated hydraulic jack. Two dial gauges having 0.01 mm accuracy placed on either side of the footing recorded the settlement of the foundation. Load was applied in small increments and the resulting deformations recorded so that the entire load-settlement curve could be obtained until failure. Since the length of the model foundation was approximately the same as the width of the test box, it can be assumed that an approximate plane strain condition did exist during the tests.

For the present test program, the following parameters were adopted for the geogrid reinforcement layers:

$$u/B = 0.35,$$
$$h/B = 0.25,$$
$$b/B = 5,$$
$$N = 2, 3, \text{and } 4 \text{ (that is, } d/B \text{ varying from 0.6 to 1.1).}$$

The sequence of the model tests is given in Table 2. Several tests were repeated in the laboratory. The variation of the ultimate bearing capacity between these tests was less than $\pm 4\%$. No attempt was made to observe and measure the deformation of the geogrid layers and the development of the failure surface in sand below the bottom of the reinforcement.

4. Model test results

4.1. Ultimate bearing capacity for unreinforced sand (test series A)

These tests were conducted on unreinforced sand. The ultimate load at $d_f/B = 0$, 0.25, 0.5, 0.75, and 1.0 occurred at s/B (s is the settlement of foundation) of 17%, 18.8%, 21.8%, 26%, and 24.6%, respectively. Fig. 3 shows the plot of ultimate bearing capacity versus embedment ratio obtained from these tests. For vertical loading condition, the ultimate bearing capacity, q_u, of a strip foundation on

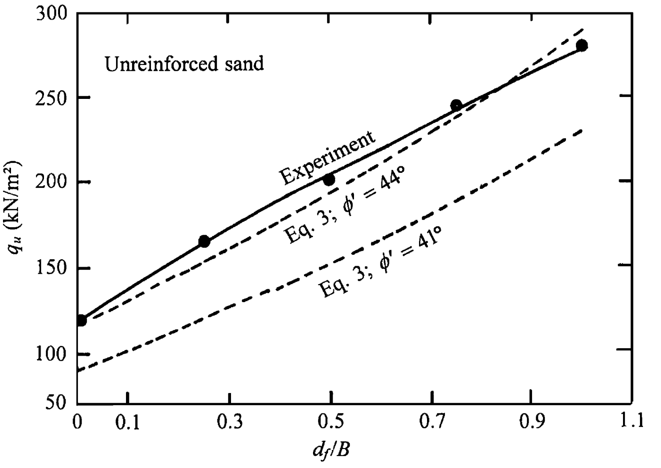

Fig. 3. Variation of q_u with d_f/B (Series A).

unreinforced sand can be expressed as,

$$q_u = \tfrac{1}{2}\gamma B N_\gamma F_{\gamma d} + qN_q F_{qd}, \tag{3}$$

where q is the γD_f, γ the unit weight of sand, N_q and $N\gamma$ the bearing capacity factors, $F\gamma_d$ and F_{qd} the depth factors.

The bearing capacity factors can be given by the following relationships (Vesic, 1973),

$$N_q = e^{\pi \tan \phi'} \tan^2\left(\frac{\pi}{4} + \frac{\phi'}{2}\right). \tag{4}$$

$$N_\gamma = 2(N_q + 1)\tan\phi', \tag{5}$$

where φ' is the effective friction angle of sand.

The depth factors can be expressed as (Hanson, 1970),

$$F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2 \frac{d_f}{B}. \tag{6}$$

$$F_{\gamma d} = 1. \tag{7}$$

Using the above relationships, the theoretical ultimate bearing capacities for the present test conditions have been calculated and are plotted in Fig. 3 along with the experimental values. Generally, the experimental values are higher than those obtained using Eq. (3). As has been pointed out by several investigators in the past, this is not very unusual primarily due to the inherent difficulty in establishing the proper magnitude of φ' for bearing capacity calculations. From Eq. (3), for surface foundation (that is, $d_f/B = 0$),

$$q_u = \tfrac{1}{2}\gamma B N_\gamma,$$

or

$$N_\gamma = \frac{2q_u}{\gamma B}. \quad (8)$$

Using the experimental values of q, γ, and B, the experimental value of $N\gamma$ was back calculated. This value of $N\gamma$ corresponds to about 44° (Eq. (4)). Using this deduced value of $\varphi' = 44°$, the variation of q_u with d_f/B was calculated, and this is also shown in Fig. 3. The general agreement of this theoretical variation with experimental results appears to be excellent.

4.2. Tests for surface foundation on reinforced sand (test series B)

Test series B was conducted on a surface foundation supported by multi-layered geogrid reinforcement (that is, $d/B = 0.6$, 0.85, and 1.1). The ultimate loads at $d/B = 0.6$, 0.85, and 1.1 were realized at $s/B = 18.8\%$, 20%, and 22.5%, respectively. The ultimate loads, $q_{u(R)}$, obtained from these tests are shown in Fig. 4.

Huang and Menq (1997) have provided a tentative relationship to determine the ultimate bearing capacity of a strip surface foundation on reinforced sand based on "wide-slab" mechanism as described in Fig. 1. The relationships can be expressed as

$$q_{u(R)} = 0.5(B + \Delta B)\gamma N_\gamma + \gamma\, d N_q, \quad (9)$$

where

$$\Delta B = 2d \tan\beta, \quad (10)$$

$$\tan\beta = 0.68 - 2.071\left(\frac{h}{B}\right) + 0.743(\mathrm{CR}) + 0.03\left(\frac{b}{B}\right). \quad (11)$$

CR is the cover ratio $= w/W$; w the width of longitudinal ribs; W the center-to-center spacing of the longitudinal ribs.

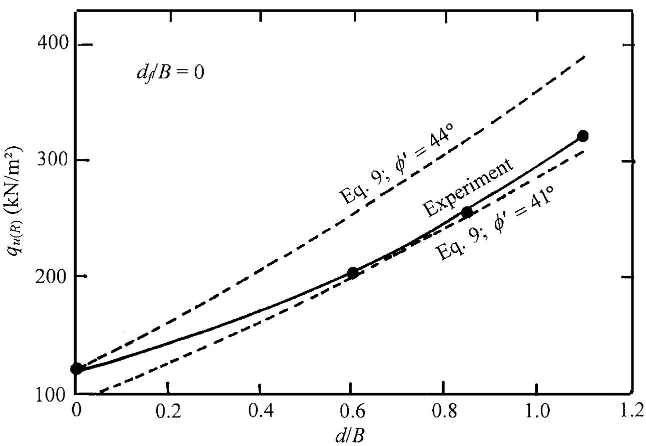

Fig. 4. Variation of $q_{u(R)}$ with d/B for surface foundation (Series B).

Eq. (9) is valid for the following ranges:

$$0 \leqslant \tan\beta \leqslant 1, \quad 1 \leqslant \frac{b}{B} \leqslant 10,$$
$$0.25 \leqslant \frac{h}{B} \leqslant 0.5, \quad 1 \leqslant N \leqslant 5,$$
$$0.02 \leqslant \mathrm{CR} \leqslant 1.0, \quad 0.3 \leqslant \frac{d}{B} \leqslant 2.5.$$

Comparing Eqs. (3) and (9), it is apparent that the depth factors ($F\gamma_d$ and F_{qd}) have not been incorporated in Eq. (9). This is primarily to be conservative due to several uncertainties involved. Using the experimental values of d, B, h, b, w, W, and $\varphi' = 41^\circ$ in Eq. (9), the theoretical variation of $q_{u(R)}$ can be obtained. This is shown in Fig. 4. There appears to be an excellent agreement between the theory and experimental values. Similar calculations for $q_{u(R)}$ with $\varphi' = 44^\circ$ were done (as in Fig. 3). This variation of $q_{u(R)}$ with d/B is also shown in Fig. 4. It may be seen that, for this case, the theoretically obtained $q_{u(R)}$ is somewhat higher than that obtained experimentally. At $d/B = 1.1$, the theoretical value of $q_{u(R)}$ is about 20% higher than that obtained from the experiment.

4.3. Bearing capacity for $d/B > 0$ and $d_f/B > 0$ (test series C)

In test series C, all tests were conducted with $d/B = 1.1$ and d_f/B varying from zero to 1.0. The ultimate bearing capacities at $d_f/B = 0$, 0.25, 0.5, 0.75 and 1.0 were obtained at $s/B = 19.9\%$, 22.3%, 26.3%, 30.1% and 33.4%, respectively. Fig. 5 shows the variation of $q_{u(R)}$ with d_f/B obtained from these tests. The bearing capacity relationship given in Eq. (9) can be modified slightly for $d_f/B > 0$ to the form

$$q_{u(R)} = 0.5(B + \Delta B)\gamma N_\gamma + \gamma(d_f + d)N_q. \tag{12}$$

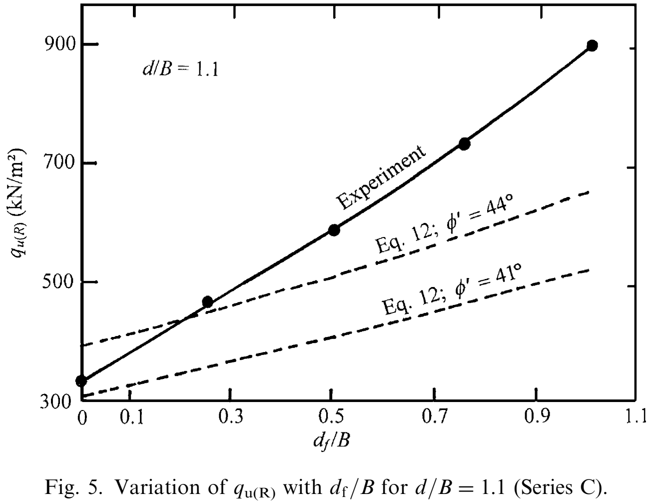

Fig. 5. Variation of $q_{u(R)}$ with d_f/B for $d/B = 1.1$ (Series C).

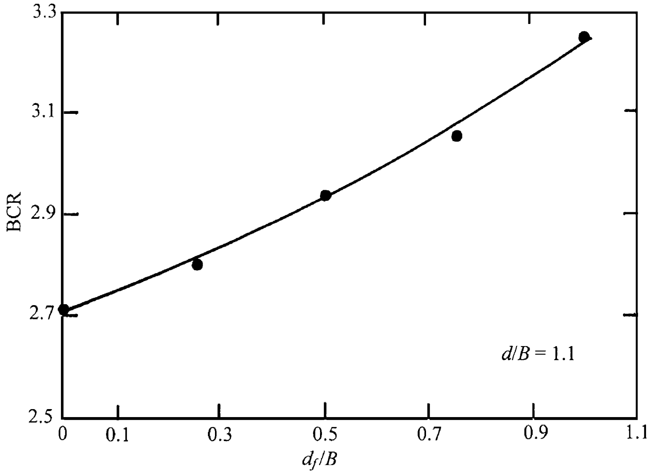

Fig. 6. Variation of BCR with d_f/B (Series C).

Using proper parameters for the present tests and $\varphi' = 41^\circ$, the variations of $q_{u(R)}$ with d_f/B have been calculated from Eq. (12) and shown in Fig. 5. The experimental values are about 15 – 70% higher than those obtained from Eq. (12). The difference increases with the increase in d_f/B; thus, Eq. (12) provides a conservative estimate of $q_{u(R)}$.

As in Figs. 3 and 4, using $\varphi' = 44^\circ$ and other proper values in Eq. (12), the variations of $q_{u(R)}$ with d_f/B were obtained and plotted in Fig. 5. It can be seen that these values, in general, are lower than the experimental values. At $d_f/B \approx 0.25$ and 1, the differences were about 0% and 35%, respectively.

The bearing capacity ratio with respect to ultimate bearing capacity was defined in Eq. (2). Fig. 6 shows the variation of BCR with d_f/B for the tests in this series. The magnitude of BCR increases with the increase in d_f/B. The experimental value of BCR at $d_f/B = 1$ is about 20% higher than that obtained at $d_f/B = 0$ for similar reinforcement-depth ratio (d/B).

5. Conclusions and recommendations

Laboratory model results for an embedded strip foundation supported by geogrid-reinforced sand have been presented. The ultimate bearing capacities obtained from these tests have been compared with the theory developed by Huang and Menq (1997). Based on the present tests, the following conclusions can be drawn:

(1) For the same soil, geogrid and its configuration, the ultimate bearing capacity and BCR increases with the increase in embedment ratio d_f/B.
(2) The theoretical relationship for ultimate bearing capacity developed by Huang and Menq (1997) provides somewhat conservative predictions.

It is recommended that tests of this type be carried out for kartstic soils and weak cohesive soils to evaluate the improvement in bearing capacity , which may be helpful in field conditions. In many cases foundations are designed for limited settlement conditions. For that reason, it may be useful to evaluate the efficiency of reinforcements at small settlements of the foundation in future studies.

As with all small-scale model tests relating to bearing capacity studies, scale effects may influence the quantitative results. It is recommended that future studies include large-scale field studies to validate the laboratory-based results.

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