Paper presented in International
Symposium on Tsunami Reconstruction
with Geosynthetics
8 to 9 December 2005, Bangkok

ULTIMATE BEARING CAPACITY OF ECCENTRICALLY LOADED STRIP FOUNDATION ON SAND REINFORCED WITH GEOGRIDS

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ABSTRACT

This paper reports the results of model loading tests performed on an eccentrically loaded strip foundation supported by multi-layered geogrid-reinforced sand. Only one type of geogrid and sand at one relative density of compaction were considered. Based on the present laboratory test results, an empirical relationship for the reduction factor has been developed. This relationship can be used to estimate the ultimate bearing capacity under eccentric loading if the corresponding value under centric loading is known.

INTRODUCTION

Mechanically stabilized earth technology is now well established in the heavy construction industry as a reliable and useful method in the construction of structures such as retaining walls, embankments over soft soil, steep slopes, and various other structures. Owing to the fact that the soil reinforcement techniques by geosynthetics have become useful and rather cost-effective in solving many problems in geotechnical engineering practice, several examples referring to the behaviour of soil with inclusions and to the feasibility of its use in practical application can be found in the literature of the last three decades.

The use of geogrid layers could be particularly convenient when the mechanical characteristics of the soil beneath a foundation would suggest the designer in adopting an alternative solution, e.g. a deep foundation. Over the last decade, the use of geogrids for soil reinforcement has increased greatly, primarily because geogrids are dimensionally stable and combine features such as high tensile modulous (low strain at high load), open grid structure, positive shear connection characteristics, light weight, and long service life. The open grid structure provides enhanced soil-reinforcement interaction.

During the last twenty 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 multiple layers of geogrid (Yetimoglu et al. 1994, Das and Omar 1994, Khing et al. 1998, Adam and Collin 1997). Most of the experimental studies cited above were conducted for surface foundation condition (that is, depth of foundation, $D_f = 0$). Shin and Das (2000) and Shin et al. (2002), Patra et al. (2005)

provided the results of a limited number of laboratory model studies for the ultimate bearing capacity of strip foundations with D_f/B (B = width of foundation) greater than zero.

None of the published studies, however, address the effect of load eccentricity on the ultimate bearing capacity. The purpose of this paper is to report some recent laboratory bearing capacity test results on eccentrically loaded strip foundations with D_f/B varying from zero to one.

BEARING CAPACITY THEORY ON UNREINFORCED SAND UNDER ECCENTRIC LOADING

Various theories for the determination of ultimate bearing capacity of shallow foundation on unreinforced sand under eccentric loading are available in literature. These are summarized as follows.

Equivalent area method: Meyorhof (1953) proposed a semi-empirical procedure to estimate the ultimate bearing capacity of a shallow foundation subjected to eccentric loading (Figure 1) that is generally referred to as the "equivalent area method".

$$q_{u(e)} = \frac{Q_u}{B} = \left[qN_q F_{qd} + \frac{1}{2}\gamma B' N_\gamma F_{\gamma d} \right] \frac{B}{B'} \tag{1}$$

where:

$q_{u(e)}$ = ultimate bearing capacity with load eccentricity e
Q_u = ultimate load per unit length of foundation;
$q = \gamma d_f$, γ = unit weight of soil, d_f = depth of foundation
B = width of foundation, $B^{/}$ = B-2e
e = eccentricity of sand
N_q, N_γ = bearing capacity factors
$F_{qd} = F_{\gamma d}$ = depth factors (Meyorhof, 1963) = $1 + 0.1\left(\frac{d_f}{B}\right)\tan\left(45 + \frac{\phi'}{2}\right)$
$\varphi^{/}$ = friction angle of sand

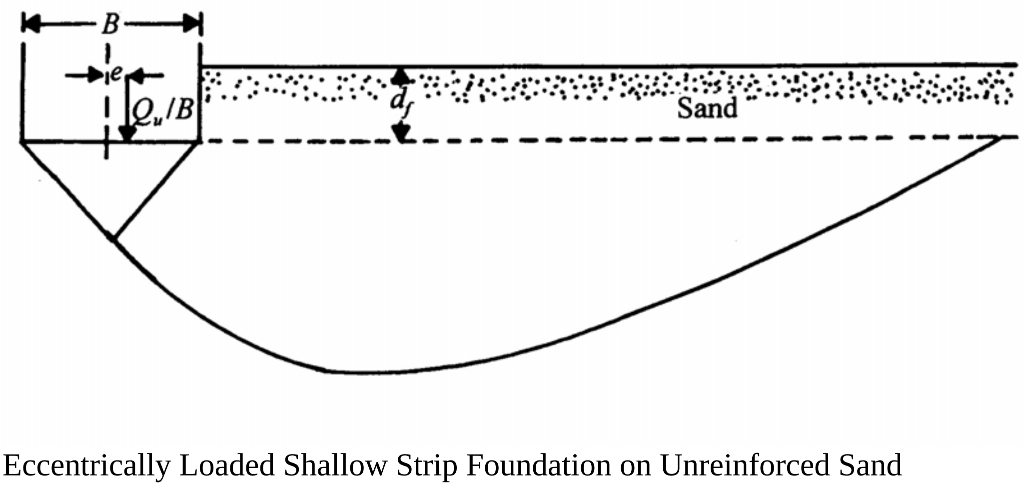

Fig. 1 Eccentrically Loaded Shallow Strip Foundation on Unreinforced Sand

Reduction factor method: Purkayastha and Char (1977) carried out the stability analysis of an eccentrically loaded strip foundation on sand using the method of slices proposed by Janbu (1957). Based on this study, they proposed that,

$$\frac{q_{u(e)}}{q_{u(e=0)}} = 1 - R_K \tag{2}$$

where R_K = the reduction factor = $\alpha\left(\frac{e}{B}\right)^K$

Based on a statistical analysis, it was also shown that B and $\varphi^{/}$ have no influence on R_K. The average values of α and K are respectively, 1.81 and 0.8. Figure 2 shows a comparison between the equivalent area method of Meyorhof (1953) and reduction factor method along with some past published laboratory experimental results (for $d_f/B = 0$). From Figure 2 it is obvious that, for $e/B \leq 0.2$, both methods provide reasonable good prediction of the ultimate bearing capacity $q_{u(e)}$.

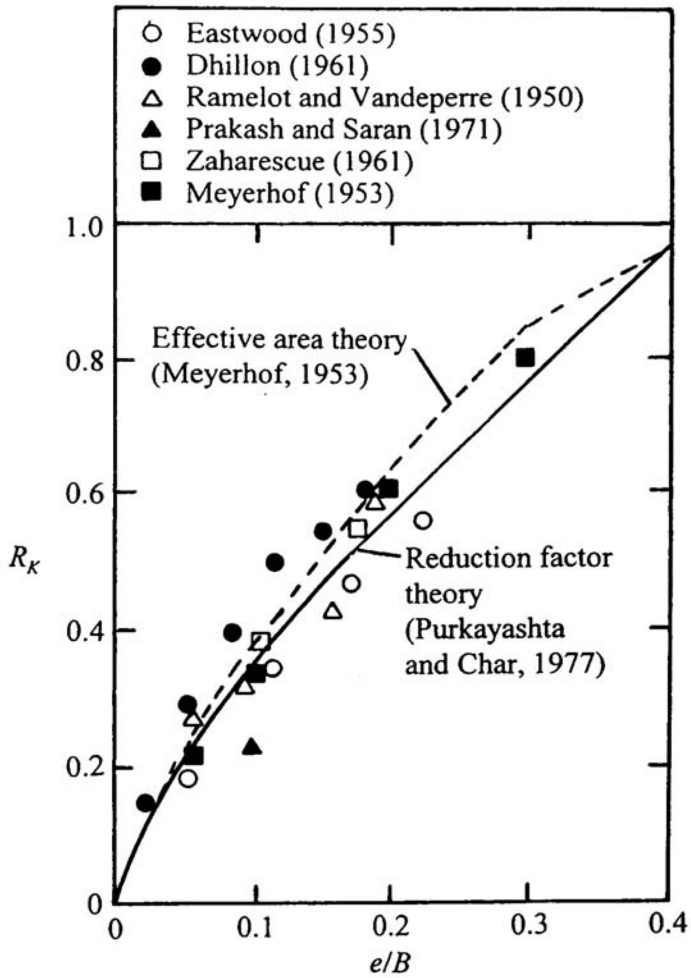

Fig. 2 Comparison of Effective Area Theory and the Reduction Factor Theory with Experimental Results for $d_f/B = 0$ (redrawn from Purkayastha and Char, 1977)

GEOMETRIC PARAMETERS AND ULTIMATE BEARING CAPACITY OF REINFORCED SAND

At the present time, a theory for the estimation of the ultimate bearing capacity of a strip foundation subjected to centric load is still under development. The most promising one is that given by Schlosser, et al. (1983), and it has been discussed in detail by Huang and Menq (1997). This is the so-called “wide slab mechanism” as shown in Fig. 3. In this figure, the shallow strip foundation has a depth of embedment of d_f. The sand is reinforced with N layers of geogrids, each having a width b. The depth of reinforcement, d, is given as follows:

$$d = u + (N-1)h \tag{3}$$

where u = vertical distance between the bottom of the foundation and the first layer of geogrid and h = distance between consecutive layers of reinforcement. According to the wide slab mechanism, the load on the foundation spreads out at an angle β with respect to the vertical from the edge of the foundation. At the bottom of the reinforced layer, the width of the loaded area is,

$$B + \Delta B = B + 2\, d \tan\beta \tag{4}$$

The failure in the soil below the reinforcement will be as shown in Figure 3. The angle β is a function of several factors and was discussed by Huang and Menq (1997). Conservatively, without using the depth factors, the ultimate bearing capacity q_{uR} can be given as:

$$q_{uR} = q'N_q + \frac{1}{2}\gamma\left(B+\Delta B\right)N_\gamma \tag{5}$$

where $q' = \gamma D_f = \gamma(d_f + d)$.

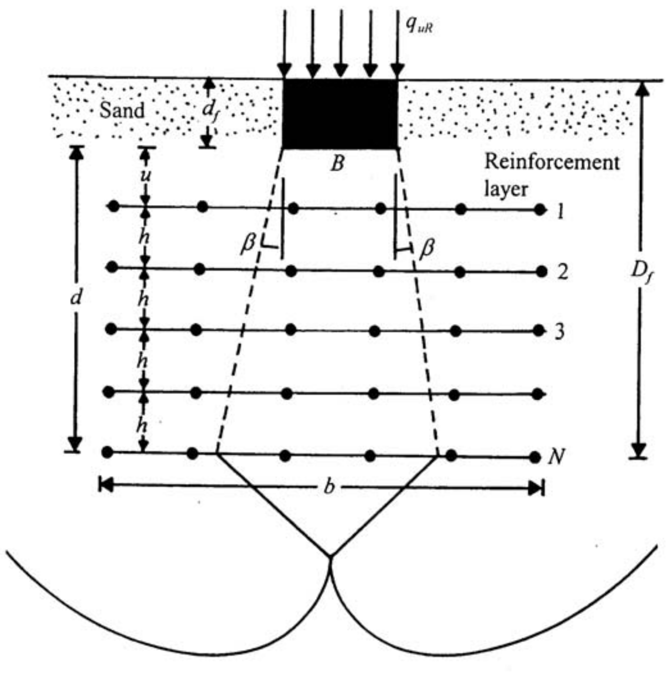

Fig. 3 Wide Slab Failure Mechanism in Soil for Shallow Foundation Supported by Multiple Layers of Geogrid-Reinforced Sand

Patra et al. (2005) by conducting model tests on the centrally loaded embedded foundations have shown that the theoretical relationship for ultimate bearing capacity developed by Huang and Menq (1997) provides conservative predictions.

For a given soil, the magnitude of the ultimate bearing capacity will depend on u/B, h/B, d/B, and b/B. However, each of these parameters will have a critical value beyond which further increase will not have any significant influence on the enhancement of bearing capacity ratio. Based on these studies published thus far, the approximate critical values of the above-stated nondimensional parameters for strip foundations can be summarized as follows: $(u/B)_{cr} \cong 0.25$ to 0.4, $(b/B)_{cr} \cong 5$ to 8, and $(d/B)_{cr} \cong 2$.

At the present time, published studies relating to the ultimate bearing capacity of shallow foundations on geogrid-reinforced sand subjected to eccentric loading [$q_{uR(e)}$] are practically nonexistent. This paper provides such results for a strip foundation based on limited laboratory model tests.

EXPERIMENTAL PROGRAM AND SETUP

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 0.8m (length) x 0.365m (width) x 0.7m (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 0.7-mm size sieve and 0% passing 0.3-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 72%, respectively. The average peak friction angle ϕ' of the sand at the test conditions as determined from direct shear tests was 42.4^0. Tensar biaxial geogrid (BX1100) was used for the present tests. In conducting a model tests, 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 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. Centric or eccentric 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 model foundation 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. Since the length of the model foundation was approximately the same as the width of the test box, it can be assumed that an approximately plane strain condition did exist during the tests. The load set-up is shown in Fig. 4.

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. The sequence of the model tests is given in Table 1.

Fig. 4. Global View of the Experimental Model

Table 1 Sequence of Model Tests

Test No.	d_f/ B	N	D_f/B	e/B
1 - 5	0	4	1.1	0, 0.05, 0.075, 0.1, 0.15
6 - 10	0.5	4	1.6	0, 0.05, 0.075, 0.1, 0.15
11, 12	0.25	4	1.35	0, 0.1
13, 14	0.75	4	1.85	0, 0.1
15, 16	1.0	4	2.1	0, 0.1
17, 18	0	2	0.6	0, 0.1
19, 20	0	3	0.85	0, 0.1

MODEL TEST RESULTS AND DISCUSSIONS

Typical plots of load per unit area, q_R, versus settlement along the center line of the model foundation (test nos. 6 through 10) obtained from the laboratory tests have been shown in Fig. 5. With the increase in the e/B values, the ultimate bearing capacity decreased accompanied by a decrease in the settlement level at which the ultimate load occurred. The variation of $q_{uR(e)}$ with e/B and d_f/B obtained from tests nos. 1 through 10 is shown in Fig. 6.

It appears that the ultimate bearing capacities shown in Figure 6 can be expressed in a form similar to that of reduction method (viz. eq. 2). Or,

$$R_K = 1 - \frac{q_{uR(e)}}{q_{uR(e=0)}} \tag{6}$$

where R_K is the reduction factor. The reduction factor for the present study will be a function of $[D_f/B = (d_f + d)/B]$ and e/B. Thus:

$$R_K = \beta_1 \left(\frac{D_f}{B}\right)^{\beta_2} \left(\frac{e}{B}\right)^{\beta_3} \tag{7}$$

The reduction factors, R_K, obtained from the experimental values given in Figure 6 for D_f/B = 1.1 and 1.6 (test nos. 1 to 5 and 6 to 10) are shown in Fig. 7. It is obvious that, for a given D_f/B, β_3 is approximately equal to 1.21. Or:

$$R_K \propto \left(\frac{e}{B}\right)^{1.21} \tag{8}$$

In order to obtain the magnitude of β_1 and β_2, the experimental values of the ultimate bearing capacity from test nos. 1 and 4, 6 and 9, 11 and 12, 13 and 14, 15 and 16, 17 and 18, and 19 and 20 can be compiled, and the variation of R_K (for e/B = 0.1) with D_f/B can be evaluated. Figure 8 shows the plots of $q_{uR(e)}$ (for e/B = 0 and e/B = 0.1) against D_f/B. The reduction factors thus obtained from these $q_{uR(e)}$ values are plotted in Fig. 9 .

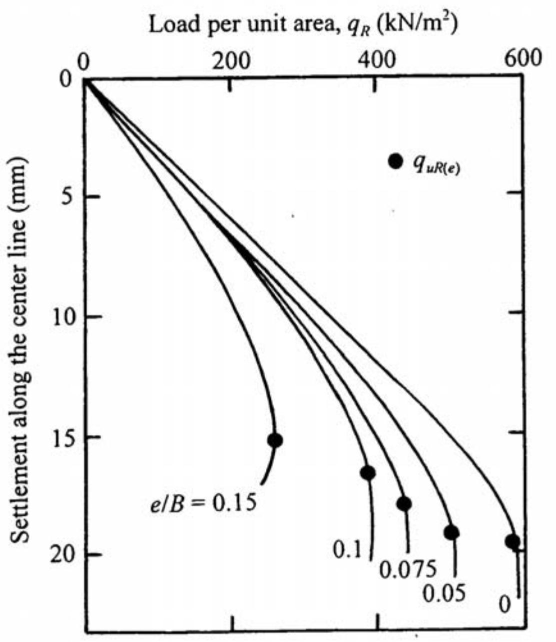

Fig. 5 Plot of Load per Unit Area, q_R, vs. Settlement (Test Nos. 6 through 10)

From this figure:

$$R_K \approx 0.315\left(\frac{D_f}{B}\right)^{-0.14} \tag{7}$$

Thus, comparing Eqs. 7, 8, and 9, β_2 = -0.14 and $\beta_1\left(\frac{e}{B}\right)^{\beta_3} = \beta_1\left(0.1\right)^{1.21} = 0.315$. Hence, $\beta_1 \cong$ 5.11.

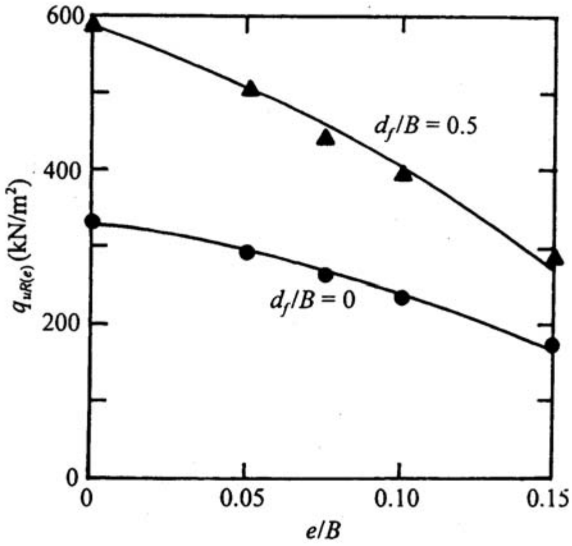

Fig. 6 Plot of $q_{uR(e)}$ vs. e/B (Tests 1 through 10)

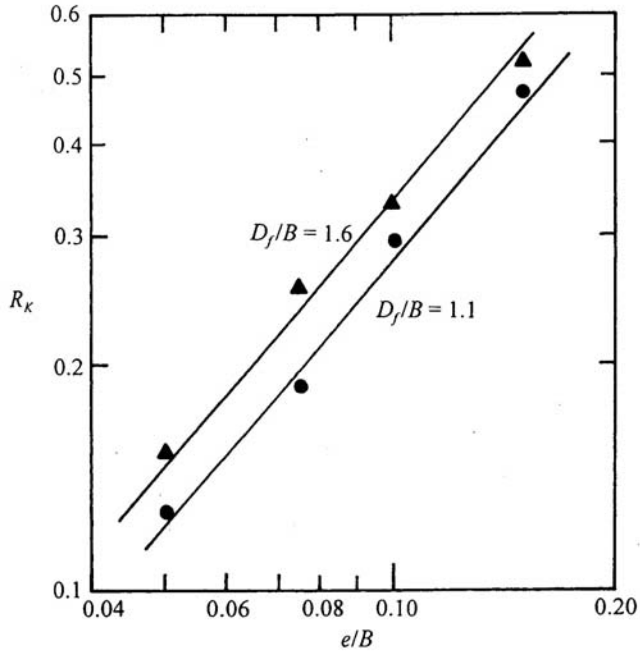

Fig. 7 Plot of R_K vs. e/B [for D_f/B = 1.1 (Tests 1,2,3,4 qnd 5) and D_f/B = 1.6 (for Tests 6, 7, 8, 9, and 10)]

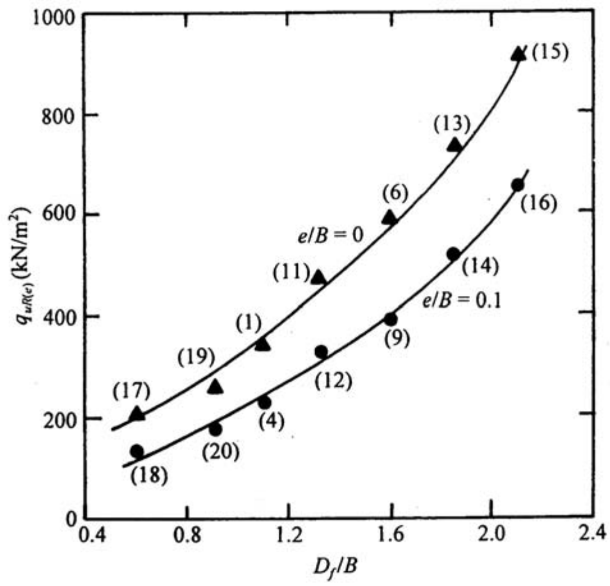

Fig. 8 Plot of $q_{uR(e)}$ versus D_f/B. (Note: The number in parenthesis is the number of the test shown in Table 3

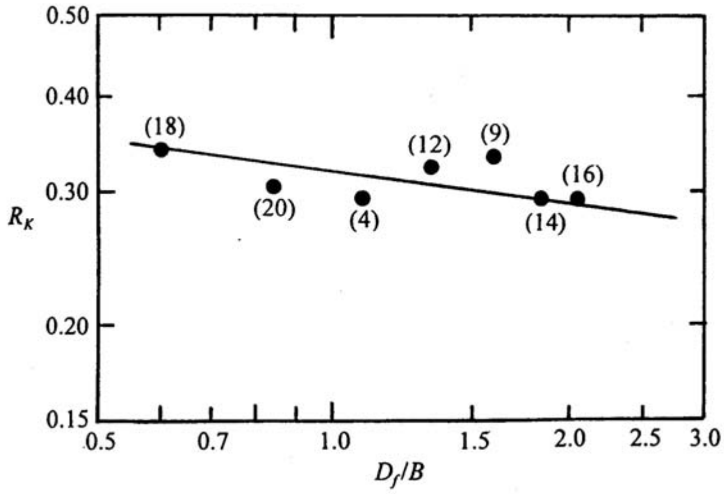

Fig. 9 Plot of R_K versus D_f/B for e/B = 0.1. (Note: The number in parenthesis is the number of the test; Table 3)

Substituting the values of β_1, β_2, and β_3 in Eq. 5:

$$R_K = 5.11\left(\frac{D_f}{B}\right)^{-0.14}\left(\frac{e}{B}\right)^{1.21} \tag{10}$$

The predicted values obtained by the preceding empirical relationship for the reduction factor is within plus or minus 8% of the present experimental results. The relation can be further improved when additional field and laboratory experimental results are available.

CONCLUSIONS

Based on the limited number of present model test results on the ultimate bearing capacity of an eccentrically loaded strip foundation supported by sand with multiple layers of geogrid reinforcement, the following conclusions can be drawn:

1. The ultimate bearing capacity of the foundation is reduced by the load eccentricity
2. The ultimate bearing capacity of an eccentrically loaded foundation resting on geogrid-reinforced sand can be estimated by the load-reduction factor.
3. The load reduction factor is found to be a function of e/B and D_f/B.

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