大韓土木學會論文集
第14卷 第6號 · 1994年 11月
pp. 1439~1444

지오그리드로 보강된 점성토사의 얕은 기초의 지지력

Bearing Capacity of Shallow Foundation on Geogrid-Reinforced Clay

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Abstract

Laboratory model test results for the ultimate bearing capacity and allowable bearing capacity at various settlement levels conducted on a strip foundation supported by geogrid-reinforced clay soil have been presented. For mobilization of the maximum possible load-carrying capacity, the optimum width and depth of the reinforcement layers, and the location of the first layer of reinforcement with respect to the bottom of the foundation have been determined.

요 지

기존의 문헌을 고찰해 보면, 지오그리드로 보강된 포화점성토에 축조된 얕은 기초의 극한지지력과 허용지지력에 관련된 이론적, 실험적 연구는 실질적으로 존재하지 않는다. 더나아가 얕은 기초는 제한된 침하의 수준에 맞게 설계되어야 하며 그리고 적절한 설계를 고려함에 있어서 허용하중조건에 대한 지지력 비를 평가하는 것이 근본이다. 그래서 지오그리드로 보강된 점성토 상에 축조된 줄기초의 침하수준별 극한지지력과 허용지지력에 대한 모형실험 결과를 제시하였다. 최대 가능한 극한지지력을 유도하게 할 수 있는 부강재들의 최적길이, 심도, 그리고 기초밑 부분으로부터 첫번째 보강재 설치위치를 결정하였다. 모형실험 결과를 근거하여 u/B, N 및 b/B에 따른 BCRu와 BCRs의 변위를 평가하였다.

1. Introduction

Several laboratory model test results are presently available in the literature related to the improvement of the load-bearing capacity of shallow foundations supported by sand reinforced with various materials like metal strips (Binquet and Lee 1975; Fragaszy and Lawton 1984, Huang and Tatsuoka 1988, 1990), metal bars (Huang and Tatsuoka, 1990), rope fibers (Akinmusure and Akinbolade 1981), geotextiles (Guido *et al.*, 1985), and geogrids (Guido *et al.*, 1986; Khing *et al.*, 1993).

These tests were primarily conducted to evaluate the following parameters in a nondimensional form, from which the most beneficial effect

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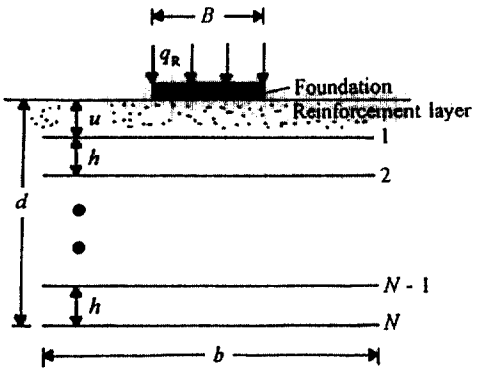

Fig. 1. Geometric Parameters for A Foundation Supported by Reinforced Soil

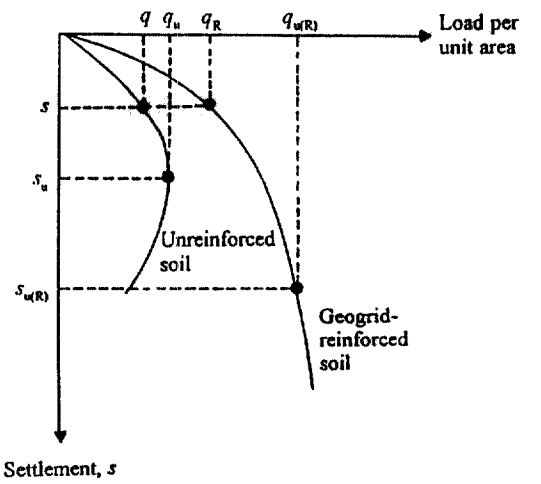

Fig. 2. General Nature of the Load-Displacement Curves

from the soil reinforcement with respect to the ultimate bearing capacity will be derived (Fig. 1): (a) location of the top layer of reinforcement, u; (b) depth of reinforcement, d=u+(N−1)h; and (c) width of each reinforcement layer, b. The improvement in the ultimate bearing capacity due to reinforcement has generally been expressed in a nondimensional form as

$$BCR_u = \frac{q_{u(R)}}{q_u} \quad (1)$$

where $q_{u(R)}$=ultimate bearing capacity with soil reinforcement; and q_u=ultimate bearing capacity without soil reinforcement. $q_{u(R)}$ and q_u have been determined the point where the maximum curvature on the load-displacement curves (Fig. 2).

A review of the existing literature shows that, unlike the bearing capacity studies on reinforced sand, theoretical and/or experimental studies related to the ultimate and allowable bearing capacities of shallow foundations supported by geogrid-reinforced saturated clayey soil are practically nonexistent. Limited information on the topic of geosynthetic-reinforced clay can be found in the works of Ingold and Miller (1982), Milligan and Love (1984), and Dawson and Lee (1988).

Furthermore, most shallow foundations are designed for limited levels of settlement.

Hense, for proper design consideration, it is essential to evaluate the bearing capacity ratio at allowable loading condition. Referring to Fig. 2, the bearing capacity ratio at an allowable settlement level (BCRs) can be defined as ($s \leq s_u$; s=settlement, and s_u=settlement at ultimate load on unreinforced soil)

$$BCR_s = \frac{q_R}{q} \quad (2)$$

where q_R and q=load per unit area of foundation reinforced and unreinforecd soil, respectively, at a settlement level s.

The purpose of this paper is to present some recent laboratory model test results for the bearing capacity of a surface strip foundation supported by a nearsaturated geogrid-reinforced clayey soil. Based on the model test results, the variations of BCR_u and BCR_s with u/B, N, and b/B have been evaluated.

2. Laboratory Model Tests

For the present model tests, a natural clayey soil was used. The soil had 98% finer than No. 200 sieve (0.075 mm opening) and 23% finer than 0.002 mm. Other physical properties of the soil are: liquid limit=44%, plasticity index=20%, and specific gravity of soil solids=2.74. The clayey soil obtained from the field was pulverized in the laboratory and mixed with predetermined amounts

Table 1. Laboratory Model Tests

Series	u/B	N	h/B	b/B	Comments
A	–	–	–	–	Test on clay only
B	0.25	4	0.333	2, 3, 4, 6, 8, 10	Tests with geogrid reinforcement
C	0.4	4	0.333	2, 3, 4, 6, 8, 10	
D	0.6	4	0.333	2, 3, 4, 6, 8, 10	
E	0.8	4	0.333	2, 3, 4, 6, 8, 10	
F	0.4	1, 2, 3, 4, 5, 6	0.333	4	Tests with geogrid reinforcement

Note: (1) For definition of u, h, b and B, refer to Fig 1.
(2) N=number of geogrid layers
(3) c_u=3.14 KN/m^2 for all tests; moisture content≈42.5%, average moist unit weigh≈17.4 KN/m^2, average degree of saturation≈97%

of water so that in the compacted condition the degree of saturated will be greater than 95%. For uniform moisture distribution, the moist soil was then placed in plastic bags and cured for about a week before use.

A biaxial geogrid was used as reinforcing material. The physical properties of the geogrid are: structure-puncture sheet drawn, polymer-PP/HDPE copolymer, junction method-unitized, aperture size(MD/XMD)-25.4 mm/33.02 mm, rib thickness-0.76 mm, and junction thickness-2.29 mm.

The model foundation measured 76.2 mm(B) × 304.8 mm and was made out of an aluminum plate. The model test box measured 1.09 m (length)× 304.8 m (width)×0.91 m (height). The sides of the box were braced with angle irons to avoid yielding during soil compaction and the actual model tests. The ends of the model foundation and the sides of the model test box were made as smooth as possible to reduce friction during the tests.

For model tests in clay, the moist soil was placed in the test box and compacted in 25.4 mm thick layers by a flat bottomed hammer. The geogrid layers were placed in the clay at desired values of u/B and h/B. The model foundation was placed on the surface of the compacted clay bed. Load to the model foundation was applied by a hydraulic jack. The load and corresponding settlement were measured by a proving ring and two dial gauges placed on each side of the center line

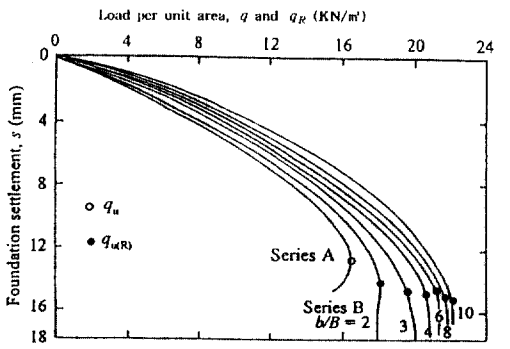

Fig. 3. Plot of Load Per Unit Area Versus Settlement (Series A and B)(Note: For Series B, N=4, h/B=1/3, and u/B=0.25)

of the foundation. The undrained shear strength, c_u, of the compacted clay was determined at the end of each bering capacity test using a hand vane shear device. Six series of tests were conducted, details of which are given in Table 1.

3. Model Test Results

3.1 Test Series A

This test was conducted on unreinforced clay. The load per unit area of the foundation, q, versus settlement, s, obtained from the test is shown in Fig. 3. From this plot, the magnitude of the ultimate bearing capacity can be determined to be 16.34 KN/m^2. For strip surface foundation

$$q_u = c_u N_c \quad (3)$$

Using the average value of $c_u = 3.14\ KN/m^2$, the magnitude of the experimental bearing capacity factor, N_c, can be calculated to be 5.21 which is in good agreement with the theoretical value of $N_c = 5.14$.

3.2 Test Series B, C, D, and E

The tests in these four series were conducted to evaluate the variation of BCR_u and BCR_s (at various s/s_u ratios) with u/B and b/B. For all tests the number of geogrid layers was kept at 4 and the spacing between the layers was equal to B/3. Fig. 3 shows the plots of q_R (load per unit area of the foundation) versus s for Test Series B, for which the magnitude of u/B was 0.25. The plots of q_R versus s for Series C, D, and E were similar in nature. The magnitudes of the settlement at ultimate load were practically the same, that is, varying between 15% to 18% of the width of the foundation, B.

Using the experimental value of q_u obtained from Series A and those of $q_{u(R)}$ from tests in Series B, C, D, and E, the variation of the bearing capacity ratios with respect to the ultimate bearing capacity, BCR_u, for various u/B ratios was calculated by Eq. (1) and is shown in Fig. 4a. In a similar manner, using Eq. (2) and the experimental load-settlement plots obtained from Series A, B, C, D, and E, the variations of BCR_s with u/B for $s/s_u = 0.75$, 0.5 and 0.25 are shown in Figs. 4b, 4c, and 4d. Based on Fig. 4, the following general conclusions can be drawn:

1. For a given b/B ratio, the bearing capacity ratio, BCR_s and BCR_u, increases with u/B and reaches a maximum and $u/B \approx 0.4$ to 0.45. For $u/B > 0.4$ to 0.45, the magnitude of BCR_u and BCR_s gradually decreases. Thus, for the present tests, the optimum value of u/B [that is, $(u/B)_{cr}$] for ultimate bearing capacity consideration and/or for consideration of the bearing capacity at limited settlement levels can be taken as 0.4.

2. At $u/B = (u/B)_{cr} \approx 0.4$, the bearing capacity ratio decreases to some extent with the increase of the settlement level [that is, BCR_s (at $s/s_u = 0.25$) > BCR_s (at $s/s_u = 0.5$) and similarly BCR_s (at

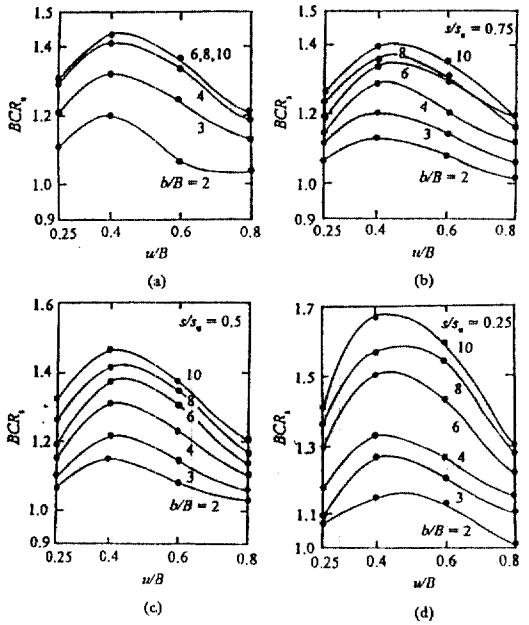

Fig. 4. Plot of BCR_u and BCR_s Versus u/B for Various b/B Values from Test Series B, C, D, and E (Note: N=4; h/B=1/3)

$s/s_u = 0.25) > BCR_u$]

3. If the magnitude of $(u/B) < (u/B)_{cr}$, then the failure surface can be approximated by two straight lines with slopes 2.7 vertical to 1 horizontal. However, for $(u/B) \geq (u/B)_{cr}$, the failure surface in soil is fully contained above the first layer of geogrid. Hence, it is obvious that higher efficiency as related to the bearing capacity can be derived by keeping $(u/B) < (u/B)_{cr}$.

Figs. 5a, 5b, 5c, and 5d show the plots of BCR_u and BCR_s at $s/s_u = 0.75$, 0.5, and 0.25 for u/B = 0.25, 0.4, 0.6, and 0.8, respectively. For any given u/B ratio, the bearing capacity ratio, BCR_u and BCR_s, increases with b/B in practically two linear segments, the initial one being steeper than the latter one. The magnitude of b/B at the point of intersection of these two segments may be referred to as the critical width ratio, $(b/B)_{cr}$. The significance of the critical width ratio is that the effectiveness of the geogrid layers in increasing the load-bearing capacity of foundation decreases for $b/B > (b/B)_{cr}$. Based on Fig. 5, the following general conclusions can be drawn:

1. The magnitude of $(b/B)_{cr}$ is about 6 for $s/s_u =$

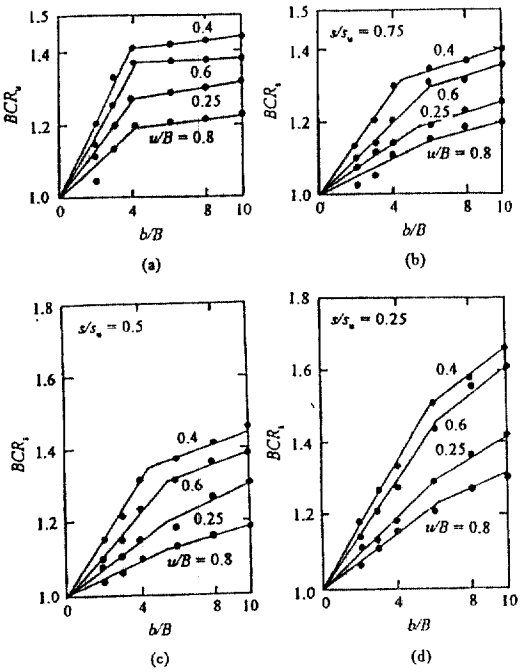

Fig. 5. Plot of BCR_u and BCR_s Versus b/B for Various u/B Values from Test Series B, C, D, and E (Note: N=4; h/B=1/3)

0.25 and decreases to about 4 at ultimate bearing capacity.

2. For $b/B \leq (b/B)_{cr}$ and a given u/B, the slope of the BCR versus b/B lines generally decreases with the increase of s/s_u. The same is true for the slope of BCR versus b/B for $b/B \leq (b/B)_{cr}$ with $\Delta(BCR_u)/\Delta(b/B)$ being practically zero.

3. For any given s/s_u, the magnitude of $\Delta(BCR)/\Delta(b/B)$ decreases with the increase of u/B. This is true for both regions, that is, b/B being less or greater than $(b/B)_{cr}$.

3.3 Test Series F

The tests in this series were conducted to determine the optimum depth of reinforcement required to obtain the maximum bearing capacity ratio. For all tests, the width of reinforcement was kept equal to 4B since this was the critical requirement, b_{cr}, for ultimate load consideration. The h/B ratio for all tests was 1/3; however, the number of reinforcement layers was varied. Using the experimental load-settlement diagrams, the variations of BCR_u and BCR_s, have been calculated and plotted in Fig. 6 against their corresponding d/B values. It can be seen from this figure that, in spite of some scatter, the variation of BCR_s with d/B for $s/s_u=0.5$ and 0.75 can be represented by a single curve which plots below the curve of BCR_u and BCR_s for $s/s_u=0.25$. However, for all curves, the bearing capacity ratio increases with d/B up to a maximum at $d/B=(d/B)_{cr} \approx 1.8$. The reason the BCR_s curves plot below the BCR_u curve is that the width of the reinforcements used for these tests is 4B, which is the b_{cr} for ultimate load consideration. As shown in Figs. 5b, 5c, and 5d, the magnitude of b_{cr} needs to be about 6B for BCR_s to exceed BCR_u for any given d/B ratio. In any case, either for ultimate bearing capacity consideration or for consideration of the allowable bearing capacity the magnitude of the critical reinforcement depth ratio, $(d/B)_{cr}$, is about 1.8. It can also be speculated that, if b/B is kept about 5, the magnitude of BCR_u and BCR_s, for any d/B ratio will be practically the same.

4. General Observations

There are several deficiencies in the laboratory bearing capacity studies of the type described in this paper. They are:1. Most small-scale laboratory bearing capacity studies and subjected to scale

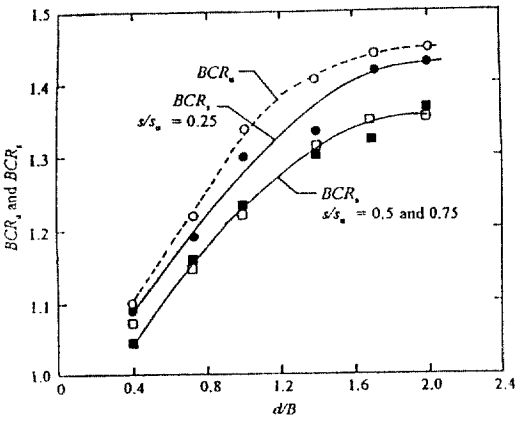

Fig. 6. Plot of BCR_u and BCR_s Versus d/B from Test Series F (Note: u/B=0.4; h/B=1/3; b/B=4)

effect. The scale effect needs further study by conducting large-scale laboratory test and also field tests.

2. Questions may arise as to the desirability of using full-size geogrids for testing model-size foundations. The flexibility of geogrids in relation to the flexibility of the model foundation needs further investigation. However, it needs to be pointed out that the geogrid used for the present tests is the weakest geogrid commercially available in the United States.

3. The maximum value of BCR_u and BCR_s derived from the present tests is about 1.5. Similar tests conducted in sand (Khing *et al.*, 1993) showed BCR_u and BCR_s to be 3 to 3.5 or more. This may be, in the case of sand, partially due to the higher passive pressure resistance developed at the soil-geogrid rib interface ($\phi>0$).

5. Conclusions

The results of a number of laboratory model tests for the bearing capacity of shallow strip foundations on a geogrid-reinforced, near-saturated clay has been reported. Based on the model test results, for ultimate bearing capacity and bearing capacity at limited settlement levels, the following conclusions can be drawn:

1. The optimum depth of reinforcement is about 1.8 times the width of the foundation.

2. The most economical width of the reinforcement layers for mobilization of maximum benefit as related to allowable load-carrying capacity is about 6B. If the width of the reinforcement layers is reduced to about 4B, the magnitude of BCR_s for all values of d/B will be about 15% to 20% less as compared to BCR_u.

3. In general, for full depth of reinforcement, the increase in the allowable bearing capacity is about 30% to 40% compared to that in unreinforced clay.

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(接受 : 1994. 7. 23)