Large Model Spread Footing Load Tests on Geosynthetic Reinforced Soil Foundations

By Michael T. Adams[1] and James G. Collin,[2] Member, ASCE

Abstract: The potential benefits of geosynthetic reinforced soil foundations are investigated using large-scale model footing load tests. A total of 34 load tests were performed to evaluate the effects of single and multiple layers of geosynthetic reinforcement placed below shallow spread footings. Two different geosynthetics are evaluated: a stiff biaxial geogrid and a geocell. Parameters of the testing program include the number of reinforcement layers, spacing between reinforcement layers, the depth to the first reinforcement layer, plan area of the reinforcement, the type of reinforcement, and soil density. Test results indicate that the use of geosynthetic reinforced soil foundations may increase the ultimate bearing capacity of shallow spread footings by a factor of 2.5.

INTRODUCTION

The use of geosynthetics in civil engineering applications is increasing annually. One new application, the construction of a reinforced soil foundation (RSF) to support a shallow spread footing, has considerable potential as a cost–effective alternative to conventional methods of support. In this technique, one or more layers of a geosynthetic reinforcement and controlled fill material are placed beneath the footing to create a composite material with improved performance characteristics (Fig. 1). During the last decade, RSF has received considerable attention from the academic community (Milligan and Love 1984; Guido et al. 1987; Huang and Tatsuoka 1990; Omar et al. 1994; Yetimoglu et al. 1994; Chadbourne 1994; and Espinoza and Bray 1995). Most of these studies have been conducted using small-scale tests to evaluate the potential benefits of a reinforced soil mass below shallow foundations.

These earlier small-scale tests qualitatively demonstrated that a geosynthetic reinforcement placed below a footing can increase both the ultimate bearing capacity and allowable bearing stress at a given settlement; however, because of the various difficulties in accurately modeling full-scale behavior with small-scale laboratory models, practitioners have not been quick to adopt this emerging technology.

In the present investigation, the model spread footings are one to two orders of magnitude larger than those used to evaluate the performance in earlier investigations. This large-scale testing was performed at the Federal Highway Administration's (FHWA) Turner-Fairbank Highway Research Center (T-FHRC), located in McLean, Va. The research included load testing large model spread footings on geosynthetic RSFs. Thirty-four load tests were conducted on RSFs consisting of one to three layers of geosynthetic reinforcement. 0.30, 0.46, 0.61, and 0.91 m square footings were loaded to failure. Thickness of the RSF with respect to the width of the footing varied from 0.25 to 1.5. Primary objectives of these experiments were to evaluate the performance of geosynthetic RSF with respect to bearing capacity and settlement; examine the effects of the spacing, area, and number of reinforcement layers; and observe influences of different sand densities within the reinforced soil mass on the performance of RSFs.

It is believed that the information presented in this paper will offer practitioners a better understanding of the mechanisms of RSF. Results indicate considerable performance improvements in the case of a poorly graded sand fill.

TESTING PROGRAM

All tests were performed in a test pit constructed at T-FHRC. The pit is a reinforced concrete box 5.4 m wide by 6.9 m long by 6 m deep. Precast, steel reinforced, concrete footings were used for all load tests. Square footings were selected to minimize the dimensional effects (strip and rectangular footings). The footing sizes selected for evaluation were 0.3 × 0.3, 0.46 × 0.46, 0.61 × 0.61, and 0.91 × 0.91 m. The footings were loaded with a hydraulic ram jacked against a reaction frame.

To date, five RSF test series have been performed at T-FHRC. A test series is defined as excavating the test pit and then replacing the sand in 0.3 m compacted lifts to a specified sand density. All experiments performed in any one particular test pit fill are included as a test series.

Material Properties

Cohesionless Soil

Fine concrete mortar sand was used in all of the experiments. The sand is classified as a poorly graded sand (SP) by the Uniform Soil Classification System and consists of subangular to angular particles with a D_{50} of 0.25 and a uniformly coefficient (C_U) of 1.7. Fig. 2 shows the grain-size distribution for the sand. The maximum dry unit weight as determined using a vibrating plate (ASTM D4253-91) is 16.7 kN/m^3. The minimum dry density is 13.8 kN/m^3 (ASTM D4254-91).

Geosynthetic Reinforcement

Two types of geosynthetic were used as reinforcement for the testing program. Four of the five RSF test series were performed with a punched/drawn polypropylene biaxial geo-

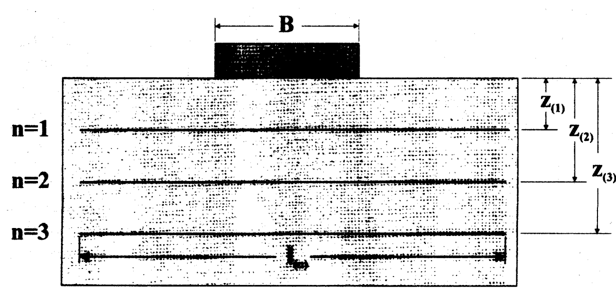

FIG. 1. Spread Footing on Reinforced Soil Foundation

[1]Res. Engr., Fed. Hwy. Admin., Turner-Fairbank Hwy. Res. Ctr., McLean, VA 22101-2296.

[2]Prin., The Collin Group Ltd., 11 Plantation Court, North Bethesda, MD 20852.

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density used in phase I. Five tests were performed in the series, three of which were conducted with two layers of grid centered 150 and 300 mm (or 0.25B and 0.5B) beneath the footing. One test was conducted with only one layer of grid located 150 mm (0.5B) beneath the footing. A control test with no geogrid reinforcement was also conducted. The test pit layout is shown in Fig. 5.

Series 6 was identical to series 5 with the exception that the in-place density of the sand was reduced from 14.5 to 14.2 kN/m^3. Table 2 summarizes all of the variables in the phase II tests.

Test Setup and Procedures

Pit Preparation

At the beginning of each test series, the test pit was excavated to a depth of 4B beneath the largest footing to be tested in the series. This depth was chosen based on Boussinesq stress-distribution theory. Using this theory, the stress below a footing dissipates to effectively zero at a depth of about 3B below the footing. Earlier footing load tests at T-FHRC instrumented with telltales to measure vertical soil strain also showed that there was no measurable vertical strain below a depth of 2B. Therefore, using an excavation depth of 4B should assure that the results are not influenced by previous tests. After excavation, the sand was carefully replaced with a backhoe in 300 mm lifts. During each lift, the sand was raked level and then compacted with a vibrating plate tamper. To maintain a consistent in-place density throughout the test pit, the same compactive effort was used on each lift. In-place density was measured using a nuclear density gauge according to ASTM D2922-91. Direct transmission measurements were taken at a depth of 200 mm in the 300 mm lifts. Five density readings were performed on each lift to ensure a consistent density. After completion of the backfilling operation, the sand was carefully leveled in the areas directly beneath the footings. This was to ensure that the surface footings had full contact with the sand and that the load applied to the footing was normal.

Test Setup and Instrumentation

The precast concrete footings were placed at predetermined locations in the test pit. The reaction frame and reference beams were set into place over the pit. Load was applied with a hydraulic jack and maintained manually with a hand pump. Load was measured with a load cell and strain indicator box. A rocker plate and ball joint were fitted to the load cell. The load cells were calibrated to within 0.03% of their rated capacity.

Four linear variable displacement transducers (LVDTs) measured settlements on the corners of the footings. Settlement data were recorded using a data-acquisition system. The LVDTs were calibrated to a precision of 0.025 mm.

Test Procedure

The load was applied incrementally. Load was increased from increment to increment only when there was no significant change (<0.075 mm) in settlement between any two time intervals. Each load increment was maintained manually. The data-acquisition system recorded settlement at 1, 3, 5, 7, 15, 20, 25, and 30 min intervals from the start of each load increment. Each load increment was held for a minimum of 5 min. After each series was completed, the sand was carefully excavated and the geosynthetic reinforcement visually inspected.

TEST RESULTS

Ultimate bearing capacity (q_{ult}) as used herein is defined as the tangent intersection between the initial, stiff, straighter portion of the load settlement curve and the steeper, straight portion of the curve, as shown in Fig. 6. For some tests the determination of q_{ult} was difficult to evaluate because there was not a sharp change in the shape of the curve (e.g., Fig. 6, test 15294T). The ultimate bearing capacity in this case was conservatively taken as the intersection of the two straight portions of the curve, even though a plunging failure was not reached. To quantify or compare the performance between each test, the bearing-capacity ratio (BCR) was calculated. BCR is defined as the bearing capacity of a footing placed on RSF divided by the bearing capacity of the same footing without the geosynthetic reinforcement. For cohesionless soils allowable settlement and not ultimate bearing capacity often governs design. The bearing capacity of the RSFs at 0.5, 1.0, and 3.0% strain [defined as settlement (s) divided by footing width (B), s/B] are also compared to the bearing capacity of the same footing without the geosynthetic reinforcement (i.e., $BCR_{@\varepsilon\%}$).

Phase I

The results of phase I are graphically shown in Figs. 6–12. Figs. 6–8 combine the load-settlement curves according to individual footing size for test series 1, 2, and 3. Fig. 6 shows that the ultimate bearing capacity of the 0.46 m square control footing, series 1 (without reinforcement), was 245 kPa. The ultimate bearing capacity of the series 2 0.46 m footing with three layers of reinforcement was 664 kPa. This corresponds to a BCR of 2.63. Note that the test for the second 0.46 m footing for series 2 was terminated prior to reaching the ultimate bearing capacity due to a problem in the test setup. The geocell, series 3, experiments were compared to the control results in series 1, as were the three layer geogrid results from series 2. The average ultimate bearing capacity of the two 0.46

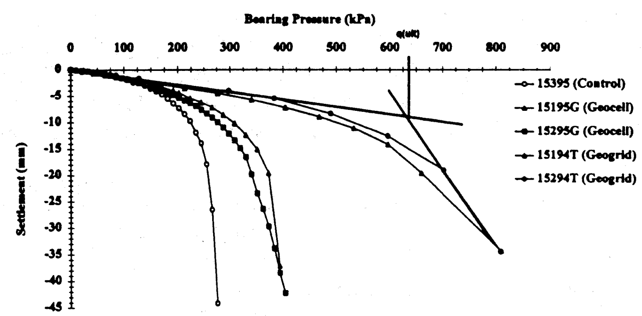

FIG. 6. Bearing Pressure versus Settlement Curves for 0.46 m Footing Series 1–3

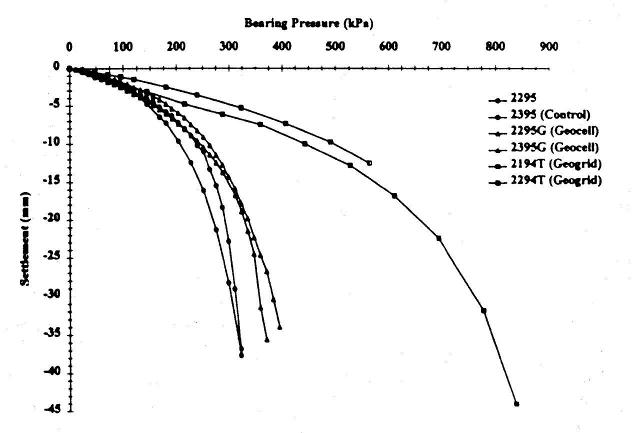

FIG. 7. Bearing Pressure versus Settlement Curves for 0.61 m Footing Series 1–3

m footings for series 3 is 298 kPa, which corresponds to a BCR of 1.27.

Fig. 8 shows the test results for the 0.91 m square footing. Again, the shape of the curves for the control, geocell, and the three layers of geogrid are similar to the tests on the 0.46 and 0.61 m footings. At low strains, however, the geocell has a lower bearing capacity than the control footing. Only after approximately 150 mm of settlement did the bearing capacity of the footing reinforced with the geocell exceed the control. The BCR for the geocell is 1.12. Due to load-capacity limitations of the reaction frame, it was not possible to completely fail the 0.91 m square footing on the three layer geogrid RSF.

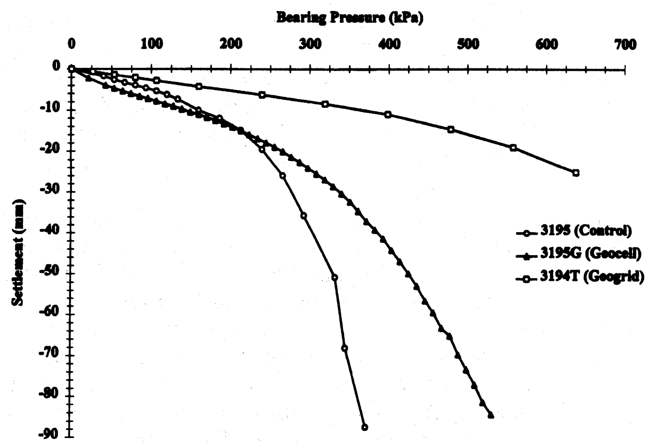

FIG. 8. Bearing Pressure versus Settlement Curves for 0.91 m Footing Series 1–3

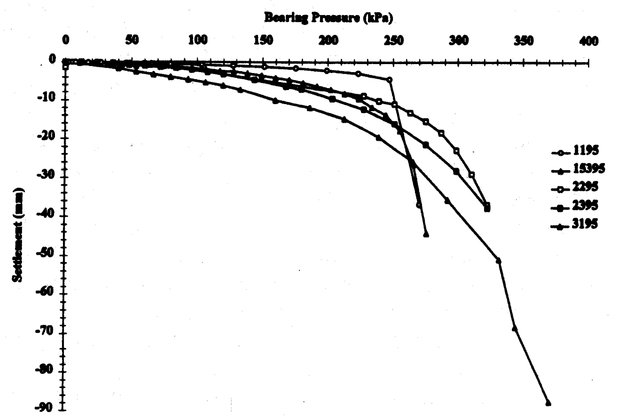

FIG. 9. Bearing Pressure versus Settlement Curves for All Footings for Series 1

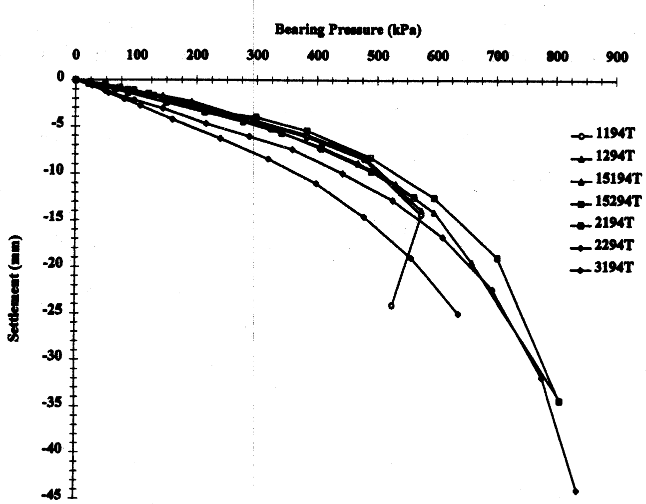

FIG. 10. Bearing Pressure versus Settlement Curves for All Footings for Series 2

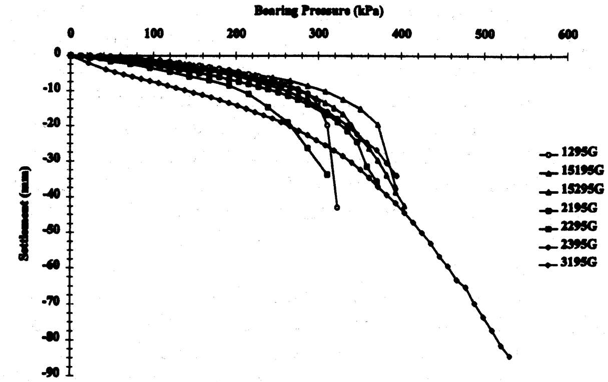

FIG. 11. Bearing Pressure versus Settlement Curves for All Footings for Series 3

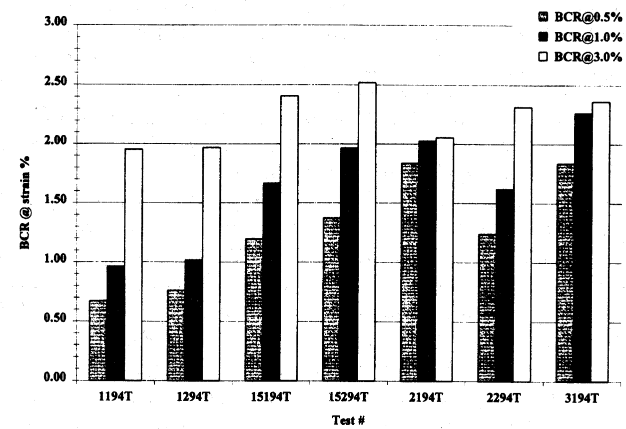

FIG. 12. Test Series 2—Bearing-Capacity Ratio Based on 0.5, 1.0, and 3.0% Settlements

However, the ultimate bearing capacity may be visually estimated from the curve to be approximately 542 kPa. Using this value, BCR for the three-layer case is 1.92.

Figs. 6, 7, and 8 show the beneficial effects of the geogrid RSF by the change in the shape of the load-settlement curve. The mode of failure is different than that for the nonreinforced footings. The slope of the load-settlement curves for the series 1 unreinforced (control) footings changed dramatically at the ultimate bearing capacity and became very steep. Failure occurred suddenly with only a small increase in load; classic soil mechanics describes this as general shear failure. The performance of the series 2, three-layer geogrid RSF results in load-settlement curves that are much flatter than the control footings at failure. The curve is rounded, and the mode of failure can be described as local shear failure. In all cases, the three-layer geogrid RSF performed better than the geocell RSF.

This may be due to the fact that the mechanism governing the interaction between the reinforcement and the soil is different between the geogrid and the geocell. The geocell provides confinement of the sand placed within the open three-dimensional cellular system. This confinement is achieved through hoop stress in the individual cells of the geocell. Geogrids provide confinement and reinforcement by a transfer of stress between the soil and the geogrid through both friction and passive resistance.

Other possible reasons for the difference in performance between geosynthetics are the reinforced soil composite was thicker for the geogrid RSF than for the geocell RSF, and it was difficult to compact the sand within the geocells so the as-placed density may have been less than 14.8 kN/m^3.

Figs. 9, 10, and 11 are the load-settlement curves for all the tests performed in series 1, 2, and 3, respectively. Fig. 9 is the summary plot for the nonreinforced footing load tests (control). The ultimate bearing capacity of the 0.3, 0.46, 0.61, and

0.91 m square footing range from 235 to 283 kPa, with an average of 256 kPa. Fig. 10 is a series 2 summary plot—shown are load-settlement curves for the 0.30, 0.46, 0.61, and 0.91 m square footings on the three-layer RSF. Depth to the reinforcement layers $z_{(1)}$, $z_{(2)}$, and $z_{(3)}$ equal 150, 300, and 460 mm, respectively. Thickness of RSF ranged from 0.5*B* for the 0.91 m footing to 1.5*B* for 0.30 m footing. Figs. 9 and 10 clearly demonstrate the excellent reproducibility of the load tests as the band of all the curves in each series of tests is relatively narrow.

The geogrid RSFs tested in phase 1 of this research program improved the ultimate bearing capacity of the foundation system by approximately 230% (i.e., $1.92 \le \text{BCR} \le 2.63$). This improvement in ultimate bearing capacity occurred at settlements between 10 and 20 mm. Depending on the application, this amount of settlement may be unacceptable. The allowable bearing capacity was, therefore, considered to determine if the RSF provides improved performance at small settlements (s/B = 0.5, 1.0, and 3.0%). Fig. 12 shows the results for the phase 1, series 2 tests. At small normalized settlements (s/B) of 0.5%, the BCR increases with footing size. For the 0.3 m footing, the average $\text{BCR}_{@0.5\%}$ is 0.71. The average $\text{BCR}_{@0.5\%}$ for the 0.46, 0.61, and 0.91 m square footings is 1.28, 1.54, and 1.84, respectively. An explanation for this phenomena may be that the z/B ratios for the reinforcement decrease as the footing size increases. In the case of the 0.91 m footing, the three layers of reinforcement were all within 0.5*B* of the bottom of the footing. As the footing load was applied, the geogrid reinforcement was immediately mobilized to reduce lateral soil strain. BCRs at 1.0 and 3.0% strain for the larger footing show an improvement greater than 2.0 in many cases. The 0.3 m footing, however, does not show any substantial improvement until a strain of 3.0%. This may be due to the fact that the depth to the first layer of reinforcement is too deep (0.5*B*). Lateral soil shear is believed to have occurred above the first layer of reinforcement, and not until substantial settlement occurred was the reinforcement mobilized.

In the series 2 tests, it was observed that when the geogrid was centered underneath the spread footing, rotation of the footing during loading was reduced; the four corners of the footing settled uniformly. The footings located in the corners of the test pit when loaded to failure, however, rotated. These spread footings were not centered on RSF, and pullout of the reinforcement may have occurred.

Phase II

Fig. 13 shows the results of series 4. Test series 4 evaluated the performance of a 0.61 m footing on one layer of geogrid at a depth (z) below the bottom of the footing of 150 or 225 mm, which corresponds to a z/B ratio of 0.25–0.375. Discrete sizes of reinforcement were also used in this series to establish the effects of reinforcement area. A control test without reinforcement was not performed in series 4. However, for comparison, the results of an unreinforced 0.61 m footing test, performed during a different test series, in the mortar sand at a density of 14.7 kN/m^3 was used in the calculation of series 4 BCRs, and is included in Fig. 13.

The results clearly show that performance was improved with one layer of reinforcement. The shape of the individual load-settlement curves for this test series are fairly similar to the curves for the three layers of geogrid. However, the initial part of test series 4 curves are not quite as flat as the curves for the three layers of reinforcement. It would appear that the bearing capacity of the footings tested using one layer of geogrid is not sensitive to depth of reinforcement, for a z/B ranging between 0.25 and 0.375. Furthermore, the bearing capacity is not sensitive to the variation in plan area of the reinforcement. TL169, the 1.8 × 1.8 m sheet at 0.375*B*, performed best in terms of ultimate capacity even though one would expect TL189, the 2.4 × 2.4 m sheet, to have the greatest ultimate capacity because of its greater area.

The BCR for the six tests in series 4 ranged from 1.56 to 1.89. This close band indicates that the bearing capacity is relatively insensitive to the parameters evaluated. The differences in performance between individual load tests in test series 4 are probably within the reproducibility of the test.

Fig. 14 is a plot of the load-settlement curves for series 5. This was the first test series to evaluate the effect of soil density on the bearing-capacity relationships observed in test series 1–4. The parameters for test series 5 were the soil density, 14.5 kN/m^3, the number of layers of reinforcement (0, 1, and 2), and the plan area of the reinforcement (1.2 × 1.2, 1.8 × 1.8, and 2.4 × 2.4 m). It is interesting to note that the test with one layer of reinforcement placed at a z/B of 0.25 performed as well as two of the tests with two layers of reinforcement. The load-settlement curves for tests TL286, TL2661, and TL2861 are very similar, as are the BCRs for these tests (1.16, 1.13, and 1.19). A higher BCR (1.47) for TL2461 is possibly due to a higher in-place density of the soil within the RSF under this footing.

The BCRs for series 5 are generally less than the BCRs in series 4. This reduction in the improvement (benefit) of the reinforced footing is believed to be a function of the in-place density of the foundation soil. For less dense soils, more soil strain is required before the beneficial effects of the geogrid can be mobilized. This trend is also evident in series 6.

Fig. 15 shows the results of test series 6. This test series was identical to series 5, except the in-place density of the foundation soil was reduced to 14.1 kN/m^3. Because of this reduction in density, the control for series 6 was half the capacity of the control in series 5; yet the BCRs for series 6 are slightly higher than for series 5. In this case, the method of

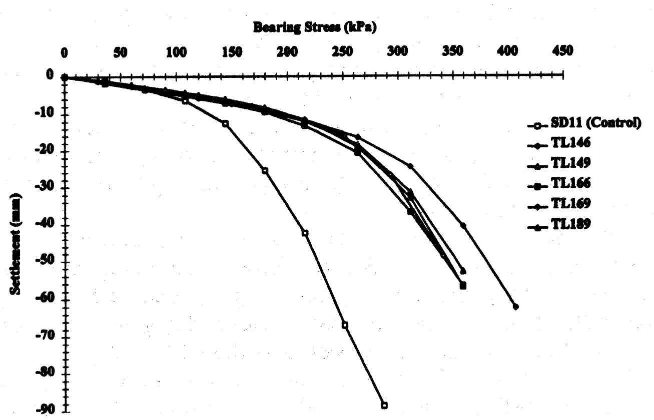

FIG. 13. Bearing Pressure versus Settlement Curves for 0.61 m Footings Series 4

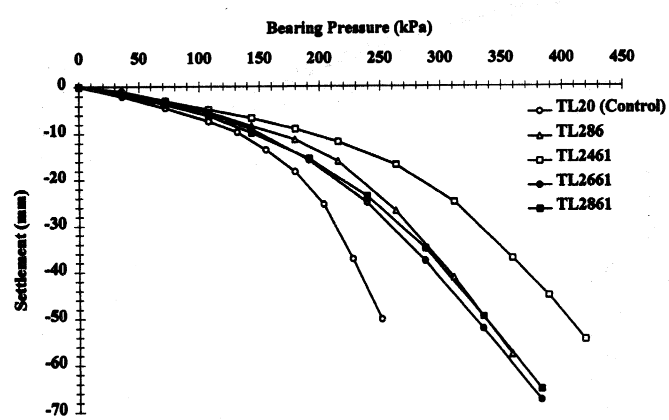

FIG. 14. Bearing Pressure versus Settlement Curves for 0.61 m Footings Series 5

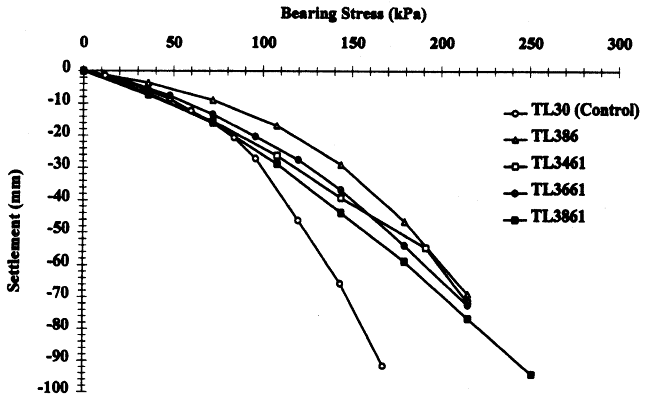

FIG. 15. Bearing Pressure versus Settlement Curves for 0.61 m Footings Series 6

using the tangent intersect to evaluate the performance is probably misleading. Perhaps another method of determining the ultimate bearing capacity would have been more appropriate for the case where allowable settlement controls performance.

For loose soils within the reinforced soil mass, the effects of the reinforcement are minimized because relatively large strains are required to mobilize the reinforcement. Also, large settlements can occur above the reinforcement. This phenomenon is shown in Fig. 15 at settlements less than 20–25 mm; the load-settlement curves for the reinforced footings have the same slope as the control test. The reinforcement is mobilized only after approximately 25 mm of footing settlement. The curves for the four load tests with reinforcement are again very similar in shape and appear to be relatively insensitive to the parameters evaluated. The BCR for the four different reinforcement cases evaluated ranged from 1.11 to 1.68. Table 2 lists the BCR for all the load tests in the phase II test program.

Table 2 lists the BCR at s/B of 0.5, 1.0, and 3.0%. The RSF at these low strain levels did not improve the performance of the shallow foundation. This confirms the previously stated observation that if the RSF is composed of soil in a loose state, large deformations of the footing are required to mobilize the beneficial effects of the reinforcement.

LIMITATIONS OF RESEARCH PROGRAM

While this is certainly the biggest large-scale load test program on geosynthetic reinforced spread footings to date, there are several limitations that should be mentioned. First, the tests were conducted on only one soil type at different densities. The results observed from this test program may be different for other soils. Only two types of geosynthetic reinforcement were evaluated; the results are therefore specific to the reinforcements tested. Other geosynthetics may perform very differently. Footing sizes of up to 0.91 m were tested with depth of reinforcement to footing width ratios varying from 0.25 to 1.5. The trends observed for these conditions should be verified for larger footings. Additionally, only surface footings were tested. The effect of footing embedment should also be included.

CONCLUSIONS

Thirty-four large model load tests were conducted to evaluate the potential benefits of geosynthetic-reinforced spread footings. One to three layers of geogrid reinforcement or one layer of geocell was placed beneath the 0.30, 0.46, 0.61, and 0.91 m square footings. The reinforcement z/B ratios were between 0.25 and 1.5. The results clearly demonstrate that geosynthetic reinforcement can substantially increase the ultimate bearing capacity of shallow spread footings on sand. The improvement, quantified as the BCR, can be significant for three layers of grid (BCR > 2.6). The BCR at 0.5, 1.0, and 3.0% settlement (s/B) was also increased when the depth to the top layer of reinforcement was less than $0.5B$. The maximum improvement in bearing capacity at low strains ($s/B = 0.5\%$) occurs when the depth to the top layer of reinforcement is within a depth of $0.25B$ from the bottom of the footing.

This improved performance can be attributed to an increase in shear strength in the reinforced soil mass from the inclusion of the geosynthetic reinforcement. The soil-geosynthetic system forms a composite material that inhibits development of the soil-failure wedge beneath shallow spread footings.

For one layer of reinforcement there appears to be an improvement in performance if the sand within the RSF is compacted to a high relative density so that stress transfer to the reinforcement occurs before large soil strains occur. Additionally, the spread footings tested on a RSF were less likely to experience a general shear, plunging failure, provided the first layer of reinforcement was placed within $0.4B$ beneath the base of the footing.

Three layers of geogrid reinforcement substantially outperforms one or two layers of reinforcement with BCRs of approximately 2.5 versus 1.6. This improvement is probably a function of the thickness of the RSF below the footing, z/B ratio. The three-layer grid case had a z/B between 0.5, and 1.5; the one- and two-layer grid cases had a z/B between 0.25 and 0.5.

Future research should further investigate the effects of the thickness of the reinforced soil mass below the footing as a function of footing width. Different soils should be evaluated to see if the trends observed in the mortar sand tests in this program apply to other soils.

The effects of a two-layer system (i.e., loose soil beneath a RSF constructed with a dense fill material) may show even larger BCRs than observed here. More large model testing on RSF in different ground scenarios will improve the understanding of the mechanism of reinforcement, which will hopefully lead to the development of a rational design methodology for this very promising new technology.

APPENDIX. REFERENCES

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