Influence of Geosynthetic Reinforcement on Granular Soils

I. ISMAIL AND G. P. RAYMOND

Model test results of strip footings on geosynthetic reinforced granular soil deposits are presented. The deposits consisted of a thin strong layer of granular material placed on a weaker granular material and a uniform single layer of the same weak granular material. The two-layer soil deposit was reinforced with a single layer of geosynthetic reinforcement. The uniform soil deposit was reinforced with one or two layers of geosynthetic reinforcement. The buried depth of the geosynthetic reinforcements was varied to determine the optimum position of placement. The optimum was based on maximizing the ultimate bearing capacity of the footing and reducing its settlement. The effect of repeated loading on the behavior of the geosynthetic reinforced granular deposits is also examined. The best method for improving the performance of weak granular soil deposits is concluded from the results.

Use of geosynthetic reinforcement grids (geogrids) in geotechnical structures, such as paved and unpaved roads, runways, and ballasted tracks, is increasing rapidly. Ballasted tracks, roads, and airfield pavements are examples of shallow foundations constructed using granular soils where the thicknesses of the layers are often relatively small compared with the width of the loaded area.

Ballasted tracks for large gantry cranes, built from granular material, are commonly subjected to very heavy loads. In trafficked areas, a thin top ballast layer is normally placed on top of a subballast layer. The top ballast generally consists of a crushed angular particle made from cobble sizes or quarried rock. The subballast is generally obtained from low-cost aggregates containing uncrushed rounded particles. Though it is potentially more economical to use uncrushed aggregate as the subballast, the subballast may cause a decrease in the stability and the track-holding capacity of the granular cover. Design problems for such construction will vary with their intended purposes. Interest might be with either a foundation failure under a concentrated load (as in the case of a gantry crane) or with trafficking problems due to rutting. One method of improving the load-bearing capacity and reducing the settlement of these tracks is to use a geosynthetic reinforcement. Relatively few studies are available relating to the optimum depth of geosynthetic reinforcement in granular soils. Studies by Dembicki et al. (*1*), Milligan and Love (*2*), Das (*3*), and Kinney (*4*) have evaluated the effects of placing geotextiles and geogrids at the interface of two different soils. However, there have been no investigations on the effects of placing the reinforcement at some other depth. This paper presents the results of such an investigation to determine the influence of the buried depth of the geosynthetic reinforcement in a uniform weak granular soil with a thin upper layer of stronger granular material or the geosynthetic reinforcement of the same uniform weak granular soil on the bearing capacity and settlement of a surface-supported footing. Model testing and the finite element method of analysis were used in the investigation, although only the model testing is reported in detail here.

Civil Engineering, Queen's University, Kingston, Ontario, K7L-3N6, Canada.

OBJECTIVE

The objective was to study, by means of model tests, the influence and comparison of geosynthetic reinforcement in granular soils using three different case deposits. The studies are illustrated schematically in Figure 1. The objectives of the individual case deposit studies were

- *Case 1*: to investigate the effect of a single layer of geosynthetic reinforcement on the bearing capacity and settlement of a footing placed on a thin layer of stronger granular material over a deep layer of weaker granular material,
- *Case 2*: to investigate the effect of two layers of geosynthetic reinforcement on the bearing capacity and settlement of a footing placed on a single layer of the weaker granular material, and
- *Case 3*: to investigate the effect of a single layer of geosynthetic reinforcement on the bearing capacity and settlement of a footing placed on a weaker single layer of the granular material.

FORMULATION OF EXPERIMENTAL PROGRAM

In this study, an experimental formulation was based on an approximate 10th scale for general rail track engineering practice. Ballast at 40-mm maximum size grading to 20-mm size was modeled by 4.8-mm (#4 sieve) grading to 2.4-mm (#8 sieve) aggregate. Ties at a length of 2 000 mm (typical for a gantry crane) interacting to form a continuous footing of that width were modeled by a plane strain (continuous) 200-mm-wide footing. A soil deposit through a rock cut could be as shallow as one-quarter the tie length (footing width). A deposit of approximately the footing width (200 mm) was selected. This ratio could be greater or less but testing in a work by Raymond et al. (*5*) has shown this to be a reasonable ratio. The microgrid used as reinforcement had a rib size of 0.3 mm or about one-tenth that of typical field geogrids. The minimum microgrid placement depth below the footing used in the study was 12.5 mm. This represents a ballast depth of 125 mm typically required to prevent geogrid damage from the tamper ties that are inserted below the ties to cause below rail-seat (rail-tie crossover) ballast compaction.

GEOSYNTHETIC PROPERTIES

The microgrid used was a biaxial-oriented polypropylene grid with approximately equal tensile strength in both directions. The main

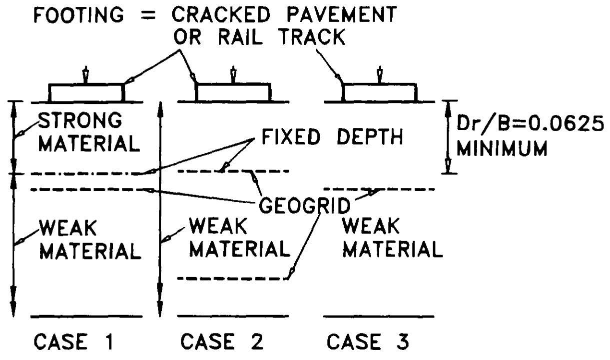

FIGURE 1 Schematic of cases studied.

properties were mass/unit area = 60 g\m³, ultimate tensile strength was approximately 48 kN/m and was independent of strain rate between 1 percent/minute to 0.001 percent/minute; strain at failure was between 10 to 12 percent, depending on testing speed. The faster testing rate resulting in the greater strain at failure. The stress-strain showed an approximately linear (slight curvature) response before failure.

TESTING EQUIPMENT

The layout of the testing equipment is shown in Figure 2. The tank used was 900 mm long, 200 mm wide, and 300 mm deep. The sides of the tank were made of herculite transparent glass with a very small coefficient of friction. The soils used were particles of a uniform 3.25-mm diameter rounded (weak material) and a similarly sized uniform graded crushed (strong material) ceramic Denstone made by Norton Chemical Processing Company. Both soils were repeatedly sized through a No. 4 (4.8-mm) sieve and retained on a No. 8 (2.4-mm) sieve to ensure a uniform grading free of broken smaller sizes. The particles had a specific gravity of 2.4. Their placement density was 1.51 and 1.40 g/m³ for the rounded and crushed particles, respectively. The soil was deposited in the test tank by dropping the particle through a uniform height of 300 mm. Dry drained triaxial tests on the rounded and crushed particles determined the internal angles of friction, ϕ', to be 34 degrees and 44 degrees respectively. A geosynthetic microgrid (geogrid) with an aperture size of 12.5 mm × 12.5 mm and a thickness of 0.3 mm was used in the tests. The geosynthetic reinforcement was cut to a length and width 25.4 mm less than the length and width of the tank to prevent any friction between the geosynthetic reinforcement and the tank walls. The model footings were made from 19-mm-thick aluminum plate. They extended over the whole width of the tank. This simulated a plane-strain loading condition equivalent to track ties where the ballast arches between the ties, approximating a long footing of uniform width. Air pressure–activated loading pistons were used to load the footing. The loads were monitored by a load cell. Dial gauges, having a travel of 25 mm and sensitivity of 0.0025 mm, were placed near each of the four corners of the footing to monitor displacements. Four sets of thrust bearings, located on drilled seats in a rectangular plate, were used to ensure that the load always acted vertically on the footing.

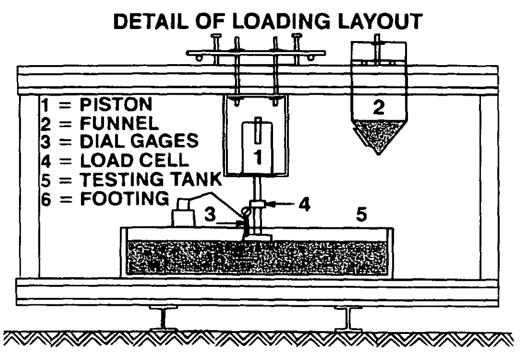

FIGURE 2 Schematic of test equipment.

EXPERIMENTAL STUDIES

Case 1

The first set of Case 1 tests consisted of statically loading a 200-mm-wide footing on a wide soil deposit in which the stronger upper layer was 12.5 mm thick and the lower weaker layer was 200 mm thick for a total depth of the two-layer deposit of 212.5 mm. Hereafter *Ht* and *Hb* will refer to the thickness of the top and bottom layers, respectively, and *B* will refer to the footing width. A single layer of geosynthetic reinforcement was placed at different depths below the surface, *Dr*, of 12.5, 25, 37.5, 50, 62.5, 75, and 125, and 175 mm, along with a test where no geosynthetic reinforcement was used. This gave ratios of geosynthetic reinforcement depth to footing width, *Dr/B*, of 0.0625, 0.125, 0.1875, 0.25, 0.3125, 0.5, 0.625, and 0.875 and the case of no geosynthetic reinforcement. The materials were then loaded statically to catastrophic failure resulting in movement to a purposely placed stop (settlement > 50 mm or *B*/4).

In the second set of tests, a single layer of geosynthetic reinforcement was placed at the same depths *Dr* as for the first set of tests, including the case of no geosynthetic reinforcement. This group of tests was then subjected to repeated loading. The repeated loadings were performed using a square wave at a frequency of 1 Hz, except for pauses at 1, 10, 10^2, 10^3, and 10^4 cycles. The pauses were made to apply a slow incremental applied load cycle of the same magnitude and lasted for about 1 hr. The pauses allowed the change in deformation modulus to be recorded. The moduli values are not presented here. Previous studies by Brown (*6*) showed little change in test observations after 10^4 loading cycles. A maximum average contact cyclic stress of 45 kPa was used. Tests (not presented here) established that for a single unreinforced soil layer, excessive settlement or failure resulted before 10^4 loading cycles when an average contact stress greater than 45 kPa was applied. After completing each repeated load test, the footing foundation was loaded to failure statically.

Case 2

The tests in Case 2 consisted of loading a 200-mm-wide footing on a wide 212.5-mm-deep deposit of the weaker granular soil. Thus, only one layer of granular material was used in this group of tests. Two layers of geosynthetic reinforcement were used. One layer of geosynthetic reinforcement was placed at a constant depth *Dr* of 12.5 mm (*Dr/B* = 0.0625). The second layer of geosynthetic reinforcement was placed at the same depths as for the tests of Case 1.

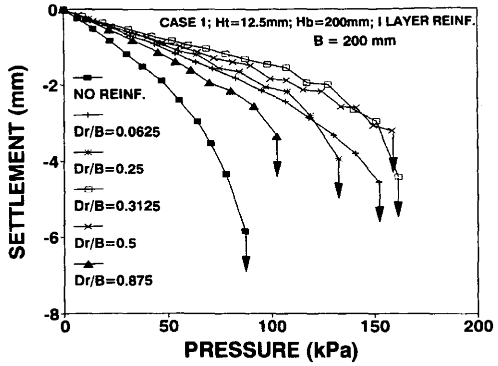

FIGURE 3 Load-settlement results before catastrophic failure for Case 1.

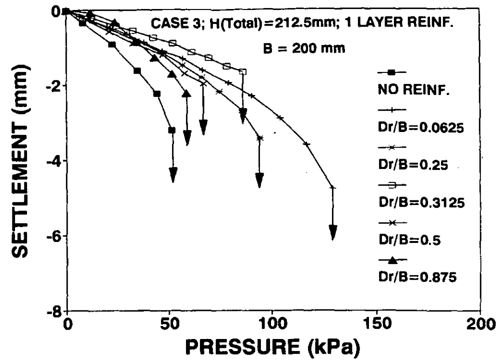

FIGURE 5 Load-settlement results before catastrophic failure for Case 3.

Also included was the case of no geosynthetic reinforcement. Similar to Case 1, two sets of tests—static and repeated load—were conducted, and at the end of the repeated load testing the footing foundation was loaded to failure statically.

Case 3

The tests in Case 3 were identical to the Case 2 tests in all respects except that the upper layer of geosynthetic reinforcement was illuminated. The single layer of geosynthetic reinforcement was thus varied in depth from an increasing ratio *Dr/B* of 0.0625 to 0.875.

STATIC TEST RESULTS

The static load was applied in small increments. Each increment was applied for 60 sec. The settlement was read after 40 sec. The load-settlement observations before catastrophic failure for the static set of tests performed in Cases 1, 2, and 3 are presented in Figures 3, 4, and 5, respectively. At low pressure levels, the settlement for all tests increased at an approximately constant rate. As failure was approached, the incremental rate of settlement increased until catastrophic failure occurred. Note that the subsequent load increment after failure, for every test, caused the maximum permitted movement of the loading piston. This was set to allow a footing settlement of at least 50 mm (i.e., > $B/4$). Herein, the ultimate bearing capacity (UBC calculated as the average intensity of loading, q_u) used is the last stable load placed on the footing. It may be seen from the figures that the settlement patterns for tests in all cases were similar, and that the geosynthetic reinforcement had the effect of increasing the UBC (q_u) of the footing and decreasing the settlement at any given load. The general trend was for the higher UBC (q_u) to be associated with the stiffer settlement responses, although this was not true for every test result. Although the effect of reinforcement locations on both the UBC (q_u) and settlement is variable, significant improvement may be seen when $Dr/B < 0.5$ for Cases 1 and 2 and < 0.3 for Case 3. When the geosynthetic reinforcement is placed to give the most benefit (optimum depth), the UBC (q_u) was approximately doubled (or greater) and the settlement at the same load was reduced by approximately 50 percent or more.

The results for the UBC (q_u) versus *Dr/B* for Cases 1, 2, and 3 are plotted in Figure 6. All were tested on the same overall depth of granular material (i.e., 212.5 mm). The results of the tests on the uniform soil deposit with a single layer of reinforcement (Case 3) follow the same trend as previously reported (*7–11,5*). In this case, the closer the geosynthetic reinforcement is located to the footing base, the more effective the soil geosynthetic reinforcement. This does not occur for the two layered soil deposit of Case 1 or the uniform soil with two layers of geosynthetic reinforcement of Case 2. It may be seen from Figure 6 that the UBC (q_u) of the reinforced two-layer deposit (Case 1) and the uniform soil with two layers of geosynthetic reinforcement (Case 2) are very much governed by the depth of the geosynthetic reinforcement.

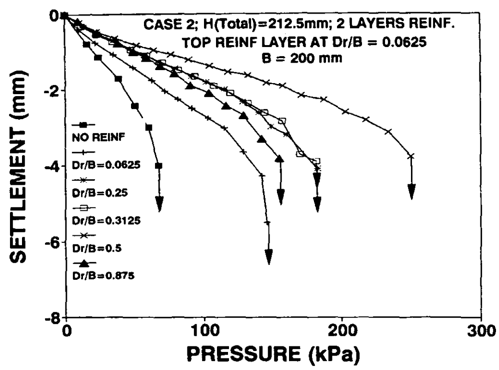

FIGURE 4 Load-settlement results before catastrophic failure for Case 2.

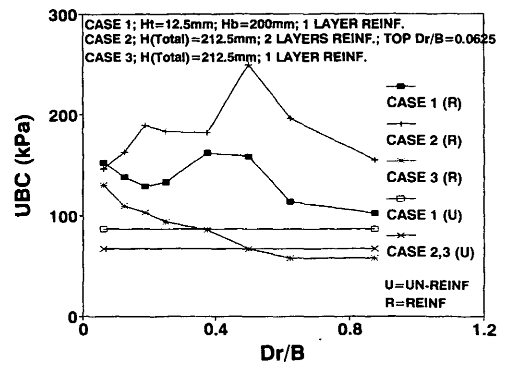

FIGURE 6 Variation of UBC (q_u) with *Dr/B* for Cases 1, 2, and 3.

In the case of the two-layer soil deposit (Case 1) when the geosynthetic reinforcement is placed very near the footing base, a high bearing capacity is first observed. As the depth of the geosynthetic reinforcement increased, the UBC (q_u) first decreased until the geosynthetic reinforcement depth to footing width ratio, Dr/B, equaled 0.1875. As the ratios of Dr/B then increased, the UBC (q_u) also increased until a maximum was observed at a ratio of Dr/B between 0.3 to 0.5, after which the UBC (q_u) decreased as Dr/B continued to increase. It is to be expected that had the geosynthetic reinforcement depth continued to be increased, a depth at which there would be a negligible effect from the introduction of the geosynthetic reinforcement could be identified. Indeed, it has been shown (*9*) that at a depth between $Dr/B = 1$ to 2, in a uniform soil deposit of depth to footing width ratio $H/B = 3$, the geosynthetic reinforcement had a negative effect [i.e., the UBC (q_u) decreased below that of the UBC (q_u) of an unreinforced deposit].

When two layers of geosynthetic reinforcement (Case 2) were introduced into the uniform soil deposit, the plot of the UBC (q_u) with Dr/B obtained gave trends similar to the Case 1 where a thin strong layer is placed on top of a deeper weaker layer. The UBC (q_u) increased as the depth to footing width ratio, Dr/B, of the second layer of the geosynthetic reinforcement increased reaching a maximum at a $Dr/B = 0.5$. The UBC (q_u) then decreased for values of Dr/B of the second deeper reinforcement greater than 0.5. During the experimental testing of Case 2, the geosynthetic reinforcement that was initially placed at values of $Dr/B \geq 0.5$ failed by breaking into two pieces below the center line of the footing. In view of this, the test was repeated several times using a number of stronger geogrids. Within experimental accuracy, all the tests using the same configuration gave the same test results. Herein, only the averages of the tests in which the geosynthetic reinforcement remained intact are reported.

The values of UBC (q_u) recorded at all Dr/B values for Case 2 are the highest values recorded in all the three cases investigated. This shows the advantages of using two layers of geosynthetic reinforcement. It must also be remembered that for Case 2, where two layers of geosynthetic reinforcement was used in a weaker soil, the UBC (q_u) was higher than the two-layer deposit with a thin stronger soil layer in the upper zone (Case 1). In an extension of the research (not presented here), it was observed that so long as the lower geosynthetic reinforcement is $\geq B$ and the upper geosynthetic reinforcement is $\geq 1.5\ B$, the same high UBC (q_u) are obtained (both geosynthetic reinforcements being centered below the footing). In fact, both these lengths may be reduced by a length of 0.5 B each, and the geosynthetic reinforcement was observed to have some beneficial value. This means that beneficial effects may be achieved from small widths of geosynthetic reinforcement. Thus the cost of using two layers of geosynthetic reinforcement should not be a factor preventing adoption of this procedure.

REPEATED LOADING TEST RESULTS

Plots of the settlement versus logarithm of number of loading cycles for the footing on a two-layer granular deposit with geosynthetic reinforcement at various depths (Case 1) are presented in Figure 7. The curves characterizing the settlements all trend in the same nonlinear pattern. These plots show that the cumulative plastic settlements observed, at the same number of load cycles, decreased when the geosynthetic reinforcement was added. The plastic settlement is defined herein as the remaining settlement after the removal of the load.

Figure 8 shows, for all three cases, the variation of the cumulative plastic settlement under the maximum number of load applications applied prior to the static load testing to catastrophic failure. Generally the optimum geosynthetic reinforcement position is observed to be between values of $Dr/B = 0.3$ and 0.5. If the geosynthetic reinforcement is placed outside this range, the repeated load settlements are higher. Again, the uniform material with two layers of geosynthetic reinforcement (Case 2) is superior to the other two cases. Generally, the position of geosynthetic reinforcement was less significant in the two-layer granular deposit (Case 1) than it was in the uniform granular deposits (Cases 2 and 3).

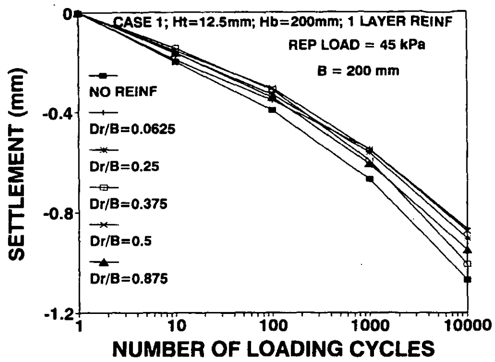

FIGURE 7 Typical plot of settlement with number of loading cycles.

Figure 9 shows for all three cases the UBC (q_u) obtained from the static load testing at the end of the 10^4 cycles of repeated loading and the values without repeated loading against the Dr/B ratio. For the repeated loading tests the UBC (q_u) was obtained by loading the soil deposits to failure statically after completing 10^4 cycles. The results show that repeated loading increased the UBC (q_u) above that obtained when no cyclic loading was applied for all deposits tested having the same dimensions and arrangement of geosynthetic reinforcement.

Note that, as state earlier, during the Case 2 static testing when the geosynthetic reinforcement was at a depth ratio of $Dr/B \geq 0.5$ the initial geosynthetic reinforcement used failed by breaking into two pieces directly below the center line of the footing. These tests were repeated using a stronger reinforcement, and only the static failure tests where the reinforcement remained intact are reported in Figure 9. The tests that were duplicated with a stronger geosynthetic reinforcement gave, within experimental accuracy, identical repeated loading results [i.e., the strength of the reinforcements (unfailed) substituted had no measurable effect on the repeated loading portion of the results].

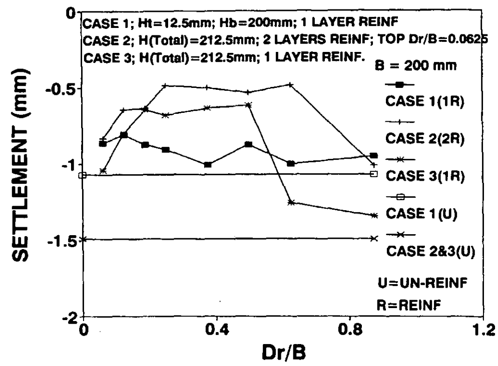

FIGURE 8 Variation of cumulative plastic settlement with Dr/B for Cases 1, 2, and 3.

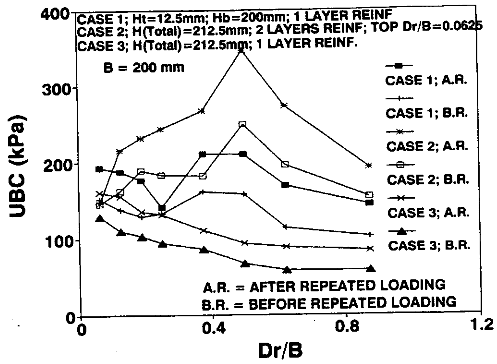

FIGURE 9 Comparison of UBC (q_u) with *Dr/B* for pre-cyclic (B.R.) and post-cyclic (A.R.) tests.

NUMERICAL ANALYSIS

To analyze the results theoretically the finite element method (FEM) of analysis was used. Details of the program have been presented elsewhere (*11*). The FEM mesh was given the same dimensions as the experimental equipment. The tank ends restricted the horizontal displacements, and the tank bottom restricted the vertical displacements. The end walls and base were modeled as smooth (i.e., frictionless and nonadhesive). Eight node quadrilateral elements and an extended hyperbolic elastoplastic model with Mohr-Coulomb's failure criterion were used to model the soil. Beam elements with a high moment of inertia and a high lateral stiffness were used to model the rigid body motion of the footing. Three node bar elements were used to model the geosynthetic reinforcement. Six-node interface elements were used to model the friction between the soil and the footing, and the soil and the geosynthetic reinforcement. The angle of friction between the soil and the footing was taken as two-thirds of the angle of friction of the soil.

The FEM load-settlement curves failed to give catastrophic failures as recorded in the static tests; the results are therefore not given here. The results, however, showed the same trends as the static tests insofar as they predicted the best depth for placement of the reinforcement. For the uniform soil with two layers of geosynthetic reinforcement (Case 2), this occurred at a ratio *Dr/B* of approximately 0.3. The results also showed that the uniform soil deposit with two layers of reinforcement gave the best reinforcement benefit. This suggests promise for further work in refinement of finite element modeling for estimating the optimum placement of geosynthetic reinforcement configurations.

CONCLUSIONS

A number of laboratory model tests and FEM analyses for the UBC (q_u) of a strip footing were performed on two layers of geosynthetic reinforced granular material. Conclusions from the observations are as follows.

1. The results for Case 1 showed that, where a two-layer granular soil deposit with a single layer of geosynthetic reinforcement was tested, the UBC (q_u) was highest and the settlements for the same load lowest when the ratio of *Dr/B* was in the range of 0.3 to 0.5 (Figure 3).
2. Similar to Conclusion 1 for Case 1, the results for Case 2 showed that, where a uniform granular soil deposit with two layers of geosynthetic reinforcement was tested, the UBC (q_u) was highest and the settlements for the same load lowest when the ratio of *Dr/B* was in the range of 0.3 to 0.5 (Figure 4).
3. In contrast to Conclusions 1 and 2, the results for Case 3 showed that, where a uniform granular soil deposit with a single layer of reinforcement was tested, the UBC (q_u) decreased and the settlements for the same load increased as the geosynthetic reinforcement depth increased (Figure 5).
4. When the UBC (q_u) for the double geosynthetic reinforced uniform granular deposit (Case 2) was compared with the UBC (q_u) for either the singly reinforced two-layer granular soil deposit (Case 1) or the UBC (q_u) of the singly geosynthetic reinforced uniform granular deposit (Case 3), the UBC (q_u) for Case 2 is always higher for the same (lower) geosynthetic reinforcement positions (Figure 6).
5. When the UBC (q_u) for the single geosynthetic reinforced two-layer granular deposits (Case 1) is compared with the UBC (q_u) for the single geosynthetic reinforced uniform granular deposit (Case 3), the UBC (q_u) for Case 1 is always higher for the same geosynthetic reinforcement positions (Figure 6).
6. For any given reinforcement configuration, the static UBC (q_u) observed at the end of 10^4 cycles of repeated loading was greater than the static UBC (q_u) values observed when the load was not cycled before static loading (Figure 9).
7. A single layer of geosynthetic reinforcement positioned close to the footing base (at a ratio of $Dr/B = 0.0625$) in a uniform granular soil (Case 3) increased the UBC (q_u) and decreased the settlements at the same loads over the case of unreinforced soil (Figure 5). The reinforcement at shallow depths gave benefit trends similar to the effect of placing a thin stronger unreinforced granular layer used in this study on the weaker unreinforced granular deposit (shallow reinforced results for Case 3 in Figure 5 compared with unreinforced results for Case 1 in Figure 3).
8. A geosynthetic reinforcement at a depth ratio of $Dr/B = 0.3$ to 0.5 increased considerably the UBC (q_u) of (a) a two-layer soil having a thin upper stronger soil layer (Case 1) or (b) a uniform soil with an upper reinforcement layer (Case 2). Similarly the settlements were reduced.
9. Optimumly placed reinforcement reduced the cumulative plastic settlements caused by repeated loadings (Figure 8).

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