An experimental evaluation of the behavior of footings on geosynthetic-reinforced sand

Murad Abu-Farsakh\textsuperscript{a,*}, Qiming Chen\textsuperscript{a}, Radhey Sharma\textsuperscript{b}

\textsuperscript{a}Louisiana Transportation Research Center, Louisiana State University, 4101 Gourrier Avenue, Baton Rouge, LA 70808, USA
\textsuperscript{b}Department of Civil and Environmental Engineering, West Virginia University, 625 Engineering Science Building, Morgantown, WV 26506, USA

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Abstract

This research was performed to investigate the behavior of geosynthetic-reinforced sandy soil foundations and to study the effect of different parameters contributing to their performance using laboratory model tests. The parameters investigated in this study included top layer spacing, number of reinforcement layers, vertical spacing between layers, tensile modulus and type of geosynthetic reinforcement, embedment depth, and shape of footing. The effect of geosynthetic reinforcement on the vertical stress distribution in the sand and the strain distribution along the reinforcement were also investigated. The test results demonstrated the potential benefit of using geosynthetic-reinforced sand foundations. The test results also showed that the reinforcement configuration/layout has a very significant effect on the behavior of reinforced sand foundation. With two or more layers of reinforcement, the settlement can be reduced by 20% at all footing pressure levels. Sand reinforced by the composite of geogrid and geotextile performed better than those reinforced by geogrid or geotextile alone. The inclusion of reinforcement can redistribute the applied footing load to a more uniform pattern, hence reducing the stress concentration, which will result reduced settlement. Finally, the results of model tests were compared with the analytical solution developed by the authors in previous studies; and the analytical solution gave a good predication of the experimental results of footing on geosynthetic reinforced sand.

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Keywords: Geosynthetics; Reinforced soil foundation; Laboratory model test; Sand; Bearing capacity ratio; Settlement reduction factor; Stress influence factor.

1. Introduction

Reinforced soil foundation (RSF) has been employed in engineering practice to increase soil’s bearing capacity and reduce footing settlement. Binquet and Lee (1975a,b) conducted a study to evaluate the bearing capacity of strip footings on reinforced sandy soil. Since then, substantial research efforts have been focused on investigating the behavior of reinforced soil foundations (RSF) as well as the effects of the different parameters on its bearing capacity. Among them, the bearing capacity of footings on reinforced sandy soil have been experimentally studied by many researchers (e.g., Akinmusuru and Akinbolade, 1981; Guido et al., 1985, 1986; Huang and Tatsuoka, 1990; Omar et al., 1993a,b; Das and Omar, 1994; Yetimoglu et al., 1994; Adams and Collin, 1997; Gabr et al., 1998; Shin et al., 2002; Basudhar et al., 2007; Ghazavi and Lavasan, 2008; Latha and Somwanshi, 2009a,b; Vinod et al., 2009; Moghadass Tafreshi and Dawson, in press; Lavasan and Ghazavi, 2012). The sand-reinforcement
interaction mechanisms are generally evaluated using direct shear and pull-out tests (e.g., Horpibulsuk and Niramitkornburee, 2010). Binquet and Lee (1975b) identified three possible failure modes depending on the configuration and tensile strength of reinforcement. They also developed a design method for a strip footing on reinforced sand based on the concept of tension membrane effect. Huang and Tatsuoka (1990) presented two mechanisms that can describe the increase in bearing capacity of RSF: deep footing mechanism and wide-slab mechanism. They substantiated the strain restraining effect (confinedment effect) by successfully using short reinforcement with a length \(L\) equal to the footing width \(B\) to reinforce sand. Adams and Collin (1997) performed a series of large scale field tests of footing on reinforced sand. Their results showed that the bearing capacity ratio (BCR) can be as high as 2.5, but the amount of settlement required for this improvement is significant and unacceptable for foundation applications. The BCR here is defined as the ratio of the bearing capacity of reinforced soil to that of unreinforced soil. Gabr et al. (1998) used plate load tests with pressure cells to study the stress distribution in geogrid reinforced sand. The results showed a better attenuation of the stress due to the inclusion of the reinforcement.

Extensive experimental, including small-scale laboratory model tests (Chen et al., 2007) and large-scale field tests (Abu-Farsakh et al., 2008), and numerical (Abu-Farsakh et al., 2007) studies were conducted by the authors to study the behavior of reinforced soil foundation; and all those studies have been focused on clayey soil. Meanwhile, the authors also made great efforts to develop analytical solutions to estimate the bearing capacity of reinforced soil (Sharma et al., 2009). This study will, therefore, focus on footings on reinforced sandy soil foundation and provide experimental data to verify the analytical model proposed by the authors.

2. Objectives and scope

A review of existing literature revealed that most of the experimental studies on geosynthetic-reinforced sand foundation were for unconfined conditions (no embedment). Limited information is available for confined conditions (embedded footing), especially in evaluating the performance of reinforced sand in terms of stress distribution within the sand and strain distribution along the reinforcement. The main objectives of this research were to investigate the behavior of geosynthetic-reinforced sand foundations and to study the effect of different parameters on their improved performance. For these purposes, extensive laboratory model tests were conducted on geosynthetic-reinforced sand foundations. Because footings are usually built at a certain embedment depth, most of tests in this research study were conducted on footings with embedment. The parameters investigated in the model tests included the top layer spacing \(a\), which is defined as the distance of the topmost layer of the reinforcement from the bottom of the embedded footing, the number of reinforcement layers \(N\), the vertical spacing between reinforcement layers \(h\), the tensile modulus of geosynthetic reinforcement and type of reinforcement, embedment depth \(D\), and shape of footing. The experimental study also investigated the stress distribution in sand and the strain distribution along the reinforcement. Fig. 1 depicts a typical geosynthetic RSF describing the geometric parameters and a typical layout of instrumentation (pressure cells and strain gauges) used in the present study. Finally, the results of model tests were compared with the analytical solution developed by the authors in previous studies (Sharma et al., 2009).

3. Material properties and test program

3.1. Material properties and model foundation

A series of laboratory model footing tests were conducted on geosynthetic reinforced sand foundation at the Geotechnical Engineering Research Laboratory (GERL) of the Louisiana Transportation Research Center (LTRC). The foundation soil consisted of uniform sand having a mean particle size \(D_{50}\) of 0.45 mm, an effective size \(D_{10}\) of 0.226 mm, and uniformity coefficient \(U_c\) and coefficient of curvature \(U_u\) equal to 2.07 and 1.25, respectively. This sand was classified as SP according to the Unified Soil Classification System (USCS), and A-1-1-b(0) according to the AASHTO soil classification system. The maximum dry density of the soil is 1620 kg/m\(^3\) with an optimum moisture content of 4.8% as determined by Standard Proctor test. Large scale \((304.8 \text{ mm} \times 304.8 \text{ mm} \times 130.9 \text{ mm})\) direct shear tests on this sand at densities of 1.686 to 1.764 kg/m\(^3\) and a moisture content of 5% revealed internal friction angles of 44° to 48°.

The model tests were conducted in a 1.5 m long, 0.91 m wide, and 0.91 m deep steel test box. The model footings used in the tests were 25.4 mm thick steel plates with dimensions of 152 mm × 152 mm (B × L) for square footings and

![Fig. 1. Geosynthetic reinforced soil foundation.](image)
152 mm × 254 mm (B × L) for rectangular footings, which were chosen, based on the dimension of box, to minimize the boundary effect. The testing procedure was performed according to the ASTM D 1196–93 (ASTM, 1993), where the load increments were applied and maintained until the rate of settlement was less than 0.03 mm/min for three minutes consecutively. The load and the corresponding footing settlement were measured by a ring load cell and two dial gauges (Fig. 1), respectively.

Three types of geogrids, GG1, GG2, and GG3, and one type of geotextile, GT1, were used as reinforcement in the tests. A composite, GGT1, which is a combination of GG2 geogrid and GT1 geotextile (i.e., GT1 geotextile is placed directly on the top of GG2 geogrid to form a new reinforcement) was also used in the present study. As a composite layered material, the equivalent mechanical properties of GGT1 were obtained by assuming that both layers sustain the same deformation during tension. The size of reinforcement was 1.47 m × 0.86 m in all tests. The physical and mechanical properties of these geosynthetics as provided by the manufacturers are listed in Table 1.

3.2. Section preparation and compaction control

Due to time constraints, only one test was conducted for each case in this study. However, two cases for the model footing test on sand were selected to check the repeatability of test sections. The corresponding pressure–settlement curves can be found in Chen (2007). The results gave the author greater confidence in the reproducibility of test sections because all test sections preparation followed the same strict quality control procedure: (1) the sand was placed and compacted in lifts inside the steel test box; the thickness of each lift varies from 25 mm to 76 mm, depending on reinforcement spacing; the amount of sand needed for each lift was calculated; (2) the test samples were prepared by hand mixing 10 kg or less sand with water every time; (3) the sand was poured into the box until the amount of sand determined in the first step reached; (4) the sand was raked level and compacted using a 203 mm × 203 mm plate adapted to a vibratory jackhammer to the predetermined height; the compaction-quality control processes to achieve the required soil densities were accomplished by conducting three passes of vibrating compaction: the compaction effort was applied through the plate for approximately eight seconds in the first pass, three seconds in the second pass, and one second in the third pass at each location; (5) the nuclear density gauge and the soil stiffness gauge device were used to measure the density and stiffness modulus for each lift.

The dry densities for sand with/without reinforcement varied from 1690 to 1763 kg/m³ with moisture contents ranging from 4.5% to 5%. The corresponding stiffness moduli were in the range of 50–60 MPa. The moist sand instead of dry sand was used in this study because the moist sand is easier to compact with vibratory jackhammer; and it is also a common practice in Louisiana to put in water when compacting sand.

3.3. Experimental testing program

Two series of tests were conducted: unconfined (surface footing) tests, and confined (embedded footing, embedment depth ratio, \( D/\ell = 1.0 \)) tests. The stress distribution in the sand was measured by Model 4800 VW earth pressure cells (102 mm diameter) from Geokon Inc. that were installed at different locations within the sand. The strain distribution along the reinforcement was measured using electrical resistance strain gauges (EP-08-250BG) from Vishay Micro—Measurements that were instrumented at different locations along the reinforcement. Table 2 summarizes the test program and test variables.

4. Test results and analysis

Typical load–settlement curves of the model footing tests are graphically shown in Fig. 2. In the following sections, the analytical discussions of test results are presented. It should be noted that the experimental test results indicated that the magnitude of settlement ratio \( s/B \) at ultimate bearing capacity is about 7–10% for embedded footings (e.g., Fig. 2b and c) and 4–7% for surface footings (e.g., Fig. 2a) on both unreinforced and reinforced sands.

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Polymer type</th>
<th>( T^a ), (kN/m)</th>
<th>( E^b ), (kN/m)</th>
<th>Aperture size, (mm)</th>
<th>( d_{min}/D_{90} )</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>MD(^c)</td>
<td>CD(^d)</td>
<td>MD(^c)</td>
<td>CD(^d)</td>
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<tr>
<td>GG1</td>
<td>PET</td>
<td>7.3</td>
<td>7.3</td>
<td>365</td>
<td>365</td>
</tr>
<tr>
<td>GG2</td>
<td>PP</td>
<td>3.6</td>
<td>5.1</td>
<td>182</td>
<td>255</td>
</tr>
<tr>
<td>GG3</td>
<td>PET</td>
<td>16</td>
<td>16</td>
<td>800</td>
<td>800</td>
</tr>
<tr>
<td>GT1</td>
<td>High-tenacity PP yarn, woven</td>
<td>14</td>
<td>19.3</td>
<td>700</td>
<td>965</td>
</tr>
<tr>
<td>GGT1</td>
<td>PP</td>
<td>17.6</td>
<td>24.4</td>
<td>882</td>
<td>1220</td>
</tr>
</tbody>
</table>

\(^a\) Tensile strength (at 2% strain),

\(^b\) Tensile modulus (at 2% strain),

\(^c\) Machine direction,

\(^d\) Cross machine direction, \( d_{min} \): Minimum geogrid opening, \( D_{90} \): Mean particle size of sand.
It seemed that although the inclusion of reinforcement can increase the ultimate bearing capacity of sand, it has minimal effect on footing settlement at the ultimate load (Fig. 2). Binquet and Lee (1975a) and Yetimoglu et al. (1994) reported the same observation. On the other hand, Omar et al. (1993a) and Das and Omar (1994) indicated that the magnitude of settlement ratio \( \frac{s}{B} \) at ultimate bearing capacity increased along with an increase of the ultimate bearing capacity for tests on reinforced sand over unreinforced sand.

### 4.1. Effect of reinforcement’s top spacing

The optimum location (top layer spacing) of the first reinforcement layer was investigated for both confined and unconfined conditions. For embedded footing \( (D_f/B = 1.0) \), Fig. 3a and b shows that the BCR at 3% settlement ratio and the ultimate loads generally increased with increasing the top layer spacing ratio \( \frac{u}{B} \) up to a maximum value at \( \frac{u}{B} = 0.33 \) for both GG1 and GG3 geogrid, after which it decreased. Top layer spacing ratio is defined as the ratio of top layer spacing \( u \) to footing width \( B \). The optimum location of the top layer is then estimated to be about 51 mm, which is equivalent to \( 0.33B \), and seems not to be related to the modulus of geogrid.

For the surface footing condition \( (D_f/B = 0.0) \), the variations of BCRs obtained at 3% settlement ratio and the ultimate loads for different top layer spacing \( u \) are shown in Fig. 4a and b. Fig. 4a shows that the BCR values at the ultimate loads for GG1 geogrid reinforced sand generally decreased as top layer spacing increased. This behavior is different from that for the embedded footing \( (D_f/B = 1.0) \), in which the BCR first increased to an optimum value and then decreased. The same phenomenon was also obtained for model tests with GG3 geogrid reinforcement, as can be seen in Fig. 4b.

### Table 2

<table>
<thead>
<tr>
<th>( D_f/B )</th>
<th>( B/L )</th>
<th>Reinforcement configuration</th>
<th>( u, ) (mm)</th>
<th>( h, ) (mm)</th>
<th>Ultimate ( q_\text{cr} ), (kPa)</th>
<th>BCR</th>
<th>Ultimate ( q_\text{cr} ), (kPa)</th>
<th>BCR</th>
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<tr>
<td>1.0</td>
<td>1.0</td>
<td>Unreinforced</td>
<td>...</td>
<td>...</td>
<td>3639</td>
<td>...</td>
<td>2604</td>
<td>...</td>
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<tr>
<td>( N=1 ), GG1</td>
<td>25</td>
<td>...</td>
<td>4261</td>
<td>1.17</td>
<td>2718</td>
<td>1.04</td>
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<td></td>
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<tr>
<td>( N=1 ), GG3</td>
<td>0–203</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
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<tr>
<td>( N=2 ), GG1</td>
<td>51</td>
<td>51</td>
<td>5171</td>
<td>1.42</td>
<td>3296</td>
<td>1.27</td>
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<tr>
<td>( N=3 ), GG1</td>
<td>51</td>
<td>25</td>
<td>5554</td>
<td>1.53</td>
<td>3265</td>
<td>1.25</td>
<td></td>
<td></td>
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<tr>
<td>( N=3 ), GG3</td>
<td>51</td>
<td>76</td>
<td>5362</td>
<td>1.47</td>
<td>3367</td>
<td>1.29</td>
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<tr>
<td>( N=4 ), GG1</td>
<td>51</td>
<td>76</td>
<td>5133</td>
<td>1.41</td>
<td>3100</td>
<td>1.19</td>
<td></td>
<td></td>
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<tr>
<td>( N=4 ), GG2</td>
<td>51</td>
<td>...</td>
<td>5458</td>
<td>1.50</td>
<td>3393</td>
<td>1.30</td>
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<tr>
<td>( N=3 ), GG2</td>
<td>51</td>
<td>51</td>
<td>5362</td>
<td>1.47</td>
<td>3335</td>
<td>1.28</td>
<td></td>
<td></td>
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<tr>
<td>( N=4 ), GG2</td>
<td>51</td>
<td>51</td>
<td>5362</td>
<td>1.47</td>
<td>3389</td>
<td>1.30</td>
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<td>( N=1 ), GT1</td>
<td>51</td>
<td>...</td>
<td>4884</td>
<td>1.34</td>
<td>2997</td>
<td>1.15</td>
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<tr>
<td>( N=2 ), GT1</td>
<td>51</td>
<td>51</td>
<td>5171</td>
<td>1.42</td>
<td>3333</td>
<td>1.28</td>
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<td>( N=3 ), GT1</td>
<td>51</td>
<td>51</td>
<td>5458</td>
<td>1.50</td>
<td>2821</td>
<td>1.08</td>
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<td>( N=4 ), GT1</td>
<td>51</td>
<td>...</td>
<td>5554</td>
<td>1.53</td>
<td>2849</td>
<td>1.09</td>
<td></td>
<td></td>
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<tr>
<td>( N=1 ), GGT1</td>
<td>51</td>
<td>...</td>
<td>5362</td>
<td>1.47</td>
<td>3038</td>
<td>1.17</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( N=2 ), GGT1</td>
<td>51</td>
<td>51</td>
<td>5745</td>
<td>1.58</td>
<td>3472</td>
<td>1.33</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( N=3 ), GGT1</td>
<td>51</td>
<td>51</td>
<td>5937</td>
<td>1.63</td>
<td>3541</td>
<td>1.36</td>
<td></td>
<td></td>
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<tr>
<td>( N=4 ), GGT1</td>
<td>51</td>
<td>51</td>
<td>5937</td>
<td>1.63</td>
<td>3553</td>
<td>1.36</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| 1.0       | 0.6       | Unreinforced              | ...         | ...         | 3562            | ... | 2253            | ... |
| \( N=4 \), GG1 | 51       | 51                         | 4711        | 1.32        | 2673            | 1.19|
| \( N=4 \), GG2 | 51       | 51                         | 4596        | 1.29        | 2750            | 1.22|
| \( N=4 \), GT1 | 51       | 51                         | 4711        | 1.32        | 2384            | 1.06|
| \( N=4 \), GGT1 | 51       | 51                         | 5401        | 1.52        | 2855            | 1.27|

| 0.0       | 1.0       | Unreinforced              | ...         | ...         | 937             | ... | 688             | ... |
| \( N=1 \), GG1 | 25       | ...                        | 1382        | 1.47        | 891             | 1.30|
| \( N=1 \), GG3 | 31–76    | ...                        | ...         | ...         | ...             | ... | ...             | ... |
| \( N=2 \), GG1 | 51       | 51                         | 1241        | 1.32        | 1032            | 1.50|
| \( N=3 \), GG1 | 51       | 51                         | 1335        | 1.42        | 1080            | 1.57|
| \( N=4 \), GG1 | 51       | 51                         | 1335        | 1.42        | 1120            | 1.63|
| \( N=2 \), GT1 | 51       | 51                         | 1171        | 1.25        | 742             | 1.08|
| \( N=3 \), GT1 | 51       | 51                         | 1265        | 1.35        | 955             | 1.39|
| \( N=4 \), GT1 | 51       | 51                         | 1265        | 1.35        | 995             | 1.45|
top layer spacing was obtained for geogrid reinforced sand for surface footing condition. As will be shown later in Section 5, the contribution of geosynthetic reinforcement to the bearing capacity of sand is proportional to the tensile force mobilized in geosynthetic reinforcement and the depth of geosynthetic reinforcement. The tensile force usually decreases with the increase of depth of geosynthetic reinforcement. As such, the variation of BCRs at the ultimate loads for different top layer depth ($u$) really depends on the magnitude and variation of reinforcement tensile force with the reinforcement depth.

Similar results on surface footing condition were also reported by Guido et al. (1986) on sand reinforced by geogrid and geotextile, and by Omar et al. (1993b) on sand reinforced by geogrid. On the other hand, a study conducted by Yetimoglu et al. (1994) indicated that the optimum top layer spacing, at which maximum bearing capacity was obtained, was about $0.3B$ for sand reinforced with single layer of geogrid. Vinod et al. (2009) reported that the optimum top layer spacing for loose sand reinforced with braided coir rope was about $0.4B$. However, the literature review showed that no such information is available for embedded footings.

4.2. Effect of number of reinforcement layers

A series of laboratory model footing tests were conducted on the sand reinforced with multiple layers of four different types of geosynthetics placed at a spacing of 51 mm (i.e. $u/B = h/B = 0.33$) for both surface footing and embedded footing conditions. The variations of BCRs obtained at settlement ratio of $s/B = 3\%$ and the ultimate loads for different numbers of reinforcement layers ($N$) and reinforcement depth ratios ($d/B$) are shown in Fig. 5a through d for embedded footings, and in Fig. 6a through b for surface footings. The reinforcement depth ratio is defined as the ratio of the total depth of reinforcement ($d$) to footing width ($B$).

As expected, the bearing capacity increased as the number of reinforcement layers increased. However, the significance of an additional reinforcement layer decreased as the number of layers increased. It can be seen from these Figures that the BCRs increase with $N$ and $d/B$, and appear to become almost
constant after \( N = 3 \) which are located at a depth of \( 1.0B \) for both surface and embedded footings for all types of reinforcement. Accordingly, the influence depth can be estimated to be \( 1.25B \). The influence depth is defined as the depth below the footing below which the inclusion of an additional reinforcement layer contributes negligibly to the increase in BCR. This result suggests that the type and modulus of reinforcement within the examined range have minimal effect on the influence depth. The influence depth also seems to be independent of footing embedment depth.

Similar to these findings, Guido et al. (1986) reported that both the geogrid and the geotextile placed below \( 1.0B \) could not improve the bearing capacity of sand. Omar et al. (1993a) indicated that the influence depth was approximately \( 2.0B \). Basudhar et al. (2007) and Latha and Somwanshi (2009a) showed that influence depth was approximately \( 2.0B \). Shin et al. (2002) reported that the influence depth was unrelated to the embedment depth.

### 4.3. Effect of vertical spacing of reinforcement layers

The effect of vertical spacing of reinforcement layers on BCRs was investigated for embedded footing \( (D_f/B = 1.0) \) condition by using three layers of GG1 with a top layer spacing of \( 51 \text{ mm} \) \((0.33B)\) and vertical spacing varied from \( 0.167B \) to \( 0.5B \). Fig. 7 depicts the variation of BCRs obtained at 3% settlement ratio and the ultimate loads for different vertical spacing ratio \( (h/B) \), which is defined as the ratio of the vertical spacing of reinforcement layers \( (h) \) to the footing width \( (B) \). It is obvious that the BCR values decreased as the vertical spacing of reinforcement layers increased with maximum BCR at \( h = 0.167B \). No optimum vertical spacing was obtained for the geogrid reinforced sand tested.

Similar results were also reported by Akinmusuru and Akinbolade (1981) on sand reinforced by rope fiber, and by Guido et al. (1986) on both geogrid and geotextile reinforced sand. On the other hand, a study conducted by Yetimoglu et al. (1994) showed that the optimum vertical spacing of reinforcement layers, at which maximum bearing capacity was obtained, was about \( 0.2B \) for reinforced sand. As stated before, there is an influence depth for placing geogrid. The effect of vertical spacing is not independent. Instead, it is a function of top layer spacing \( (u) \) and number of layers \( (N) \), and may also be a function of reinforcement size. However, for the sand and geogrid reinforcement tested in this study, the smaller the spacing, the higher the BCR. For design purpose, engineers need to balance between reducing spacing and increasing geogrid modulus. The authors believe a value of \( h/B = 0.2 \) can be a reasonable value for use in the design of reinforced sand foundation.
4.4. Effect of footing depth and shape

The effect of embedment depth on the BCR of reinforced sand was investigated by conducting two sets of model tests, one without embedment depth \((D_f/B = 0.0)\), and one at an embedment depth equal to the footing width \((D_f/B = 1.0)\). The tests conducted in the present study indicated that at the same settlement ratio, the BCRs for surface footings were generally greater than those for embedded footings \((D_f/B = 1.0)\) (Fig. 8).

The present study also showed that the BCRs at the ultimate bearing capacity for surface footings were generally smaller than those for embedded footings \((D_f/B = 1.0)\) (Table 2). This finding may be expected in the light of the fact that the settlement ratios \((s/B)\) at the ultimate bearing capacity for embedded footing \((D_f/B = 1.0)\) are greater than those for surface footing. Similar to the finding of the present study, Shin et al. (2002) and Patra et al. (2005) reported that the magnitude of BCRs at the ultimate bearing capacity for strip footing increased with increasing \(D_f/B\). However, Shin et al. (2002) also indicated that for \(s/B < 5\%\) the BCRs for surface footings were less than those for embedded footings.

The effect of footing shape on the BCR of reinforced sand was also investigated by conducting two sets of model tests, one with a square footing \((B/L = 1.0)\), and one with a rectangular footing \((B/L = 0.6)\). The test results (Table 2) showed that the ultimate bearing capacity of unreinforced
sand for square footings \((B/L = 1.0)\) was greater than that for rectangular footings \((B/L = 0.6)\), which is consistent with the theoretical analysis of unreinforced soil by using bearing capacity formula suggested by Vesic (1973). The test results also indicated that the BCRs at the ultimate bearing capacity for square footing \((B/L = 1.0)\) were greater than those obtained for rectangular footing \((B/L = 0.6)\) (Table 2). A similar trend was identified for the BCRs at 3% settlement ratio, while the opposite trend was observed for the BCRs at the residual loads (post-failure stage). On the other hand, Omar et al. (1993a) reported that the BCRs at the ultimate bearing capacity decreased with increasing the \(B/L\). It should be pointed out that in their study, the model tests were conducted in surface footing conditions, in which the ultimate bearing capacity of unreinforced sand decreases with increasing \(B/L\) according to theoretical analysis by using bearing capacity formula suggested by Vesic (1973).

4.5. Effect of tensile modulus and type of geosynthetic reinforcement

Four different types of reinforcement with different tensile moduli were used in the model footing tests. The properties of these reinforcements were presented earlier in Table 1. As seen in Fig. 2, the performance of the GG1 and GG2 geogrids is very similar until the ultimate bearing capacity is reached. After this point, the sand reinforced by the GG1 geogrid, which has a higher modulus and smaller aperture size than the GG2 geogrid, performs appreciably better than sand reinforced by the GG2 geogrid. This point is more clearly demonstrated in Figs. 9 and 10. Similar results were reported by Huang and Tatsuoka (1990) for strip footing on reinforced sand.

![Fig. 7. BCR versus \(h/B\) for three layers of GG1 (\(B/L\): 1.0; \(D_f/B\): 1.0).](image1)

![Fig. 8. BCR versus \(s/B\) for both embedded and surface footing. (a) \(N = 3\) and (b) \(N = 4\).](image2)

![Fig. 9. BCR versus type of reinforcement. (a) @ ultimate and (b) @\(s/B = 12\%).](image3)
figures show that the BCR generally increased as the settlement ratio \((s/B)\) increased. Before the ultimate bearing capacity was reached, the BCRs of geotextile reinforced sand were smaller than those of geogrid reinforced sand, except for one layer. However, the rate of increase of BCRs with the increase of settlement for geotextile reinforced sand was higher compared to that for geogrid reinforced sand. Consequently, at the post-failure stage, the BCRs of geotextile reinforced sand were much greater than those of geogrid reinforced sand. This point can also be clearly seen in Fig. 9. Furthermore, the bearing capacity of geotextile reinforced sand at low settlement level \((s/B < 2\% \text{ for embedded footing and } s/B < 1.5\% \text{ for surface footing})\) was even less than that of unreinforced sand (i.e., BCR < 1). Guido et al. (1985) reported that for \(s/B < 1.7\%\), the response of unreinforced sand was stiffer than that of geotextile reinforced sand. This behavior can be attributed to the slack effect of woven geotextile, which initially creates planes of weakness at the sand–geotextile interface. The slack of woven geotextile can be caused by stretching of woven, test setup, or both. At low settlement level, the friction and adhesion developed at the sand–geotextile interface starts to stretch the geotextile. With the increase of settlement, the slack of woven geotextile would be removed gradually; and finally, the geotextile would be fully stretched. After reaching a certain amount of settlement, because of its highest tensile modulus out of four types of geosynthetic reinforcement used in this study, the reinforcing effect of geotextile would be more appreciably mobilized. Interestingly, the ultimate bearing capacity of geotextile reinforced sand was somewhat higher than that of geogrid reinforced sand for embedded footings, while it was obviously lower for surface footings. Figs. 9 and 10 also show that the sand reinforced by GGT1 composite, which acts as a combination of “plane textile” and “grid reinforcement,” performed better than that reinforced by either geogrid or geotextile alone. This better performance of GGT1 composite becomes more pronounced at the post-failure stage. This is because GGT1 can take advantage of both interlocking effect of geogrid and high tensile modulus of geotextile.

Because of a serviceability requirement, foundations are always designed at a limited settlement level. From an engineering practice point of view, geogrid reinforcement is generally considered to perform better for soil foundation than geotextile. Similar to this finding, Guido et al. (1986) and Lee and Manjunath (2000) reported that the performance of geogrid reinforced sand was far better than geotextile reinforced sand. But just as Guido et al. (1986) stated, the selection of the type of reinforcement in engineering practice is a project-dependent issue. For example, some projects require that geosynthetics only function as reinforcement; while in other projects, geosynthetics are required to function as both reinforcement and separator or filter, in which relatively poor reinforcement is also acceptable.
The settlement reduction factors (SRF) at different footing pressures \( (q) \) for the model tests with multiple layers of different types of reinforcement are presented in Fig. 11a and b. The SRF is defined here as the ratio of the immediate or elastic settlement of the footing on reinforced sand to that on unreinforced sand at a specified footing pressure. It is obvious that the inclusion of the reinforcement would reduce the immediate settlement, except for the geotextile at a footing pressure less than 2000 kPa. This behavior can be attributed to the slack effect of woven geotextile as described earlier in this section. With two or more layers of geogrid, the settlement can be reduced by 20% at all pressure levels. This study showed that modulus of geogrid has minimal effect on reducing the settlement in sand. The rate of decrease of SRF with the increase of applied footing pressure for geotextile reinforced sand is higher compared to that for geogrid reinforced sand. The GGT1 composite is the most effective at reducing the footing settlement.

4.6. Stress distribution in sand

Several laboratory model tests were conducted to evaluate the stress distribution in sand with and without reinforcement inclusion. Pressure cells were placed at specified locations/depth for this purpose, as shown in

Fig. 12. Stress distribution along the center line of footing at a depth of 254 mm \((1.67B)\) below the footing \((B/L: 1.0; D_f/B: 1.0)\). (a) Applied Footing Pressure \( q=689 \text{ kPa} \) and (b) Applied Footing Pressure \( q=1839 \text{ kPa} \).

Fig. 13. Stress distribution along the center line of footing at a depth of 254 mm \((1.67B)\) below the footing \((B/L: 1.0; D_f/B: 0.0)\). (a) Applied Footing Pressure \( q=94 \text{ kPa} \) and (b) Applied Footing Pressure \( q=750 \text{ kPa} \).

Fig. 1. The measured stress distributions along the center line of the footing at a depth of 254 mm \((1.67B)\) below the footing for both embedded rectangular footing and surface square footing are shown in Figs. 12 and 13, respectively. Fig. 14 depicts the variation of the stress influence factor \((I)\) under the center of the footing with applied footing pressures. The stress influence factor \((I)\) is defined here as the ratio of the induced stress at a certain location/depth in soil to the footing pressure. It should be noted that the stresses measured here by the pressure cells are the total vertical stresses induced by the applied load, not including the stresses induced by the weight of soil.

As shown in these Figures, the soil reinforcement resulted in redistribution of the applied load to a more uniform pattern, thus avoiding stress concentration and achieving improved stress distribution. The induced maximum stresses beneath the center of the footing in reinforced sand were appreciably reduced compared to those in unreinforced sand, especially for the surface footing condition. For embedded rectangular footings with four layers of reinforcement, the reduction in maximum stress ranges from 6% to 13% and from 8% to 15% at a footing pressure of 689 kPa and 1839 kPa, respectively. For
surface square footings with four layers of reinforcement, the reduction in stress varies from 43% to 56% and from 31% to 34% at a footing pressure of 94 kPa and 750 kPa, respectively. As mentioned earlier, weak clayey soil is encountered in many foundation applications, and one treatment method is to replace part of the weak cohesive soil with an adequately thick layer of stronger granular fill. Granular fill in combination with geosynthetics reinforcement can form a composite zone. The resulting reinforced soil mass, as indicated in this study, distributes loads uniformly below the reinforced zone, i.e., over underlying weak clayey soils. This redistribution of load will result in reducing the thickness of stronger granular fill and hence prevent deep excavation, which is expensive and labor intensive.

Among the geogrids used, the geogrid (GG1) with higher modulus resulted in a better reduction of center stresses than the geogrid (GG2) with lower modulus. The GT1 geotextile, which has a higher tensile modulus than the geogrids used in the present study, showed better attenuation of the stresses under the center of footing than the geogrids. The GGT1 composite provided the best attenuation of the center stresses among the four types of reinforcement used in the present study. It seems that the improvement in stress distribution in reinforced sand is somehow related to the tensile modulus of geosynthetic reinforcement. It is also noted that the improved performance of reinforced sand was not always compatible with the improved stress distribution. As shown earlier, before the ultimate bearing capacity reached, geogrid reinforced sand generally performed better than geotextile reinforced sand, however the induced stresses under the center of the footing in geogrid reinforced sand were higher than those in geotextile reinforced sand. This observation is in agreement with work by Leng (2002). They attributed this performance to the better tension membrane effect in geotextiles than in geogrids.

Interestingly, negative stresses, which mean that the vertical stresses at the measured points are less than the weight of soil, were measured in unreinforced sand for surface footings at approximately 2.5B from the center of footing. This result indicates that the sand was pushed upward at a distance of around 2.5B from the center of footing in unreinforced sand. In the meantime, no negative stresses were measured in reinforced sand for surface footings at approximately 2.5B from the center of footing. This means the inclusion of reinforcement can develop a “surcharge effect” to prevent soil from moving upward, and thus improve the bearing capacity of sand. This phenomenon is called “surcharge effect” because this effect is equivalent to adding a surcharge load.

Fig. 14 shows that the stress influence factor (I) is a load-dependent value instead of a constant value and it increases

![Fig. 14. Variation of stress influence factor with the applied footing pressure. (a) Embedded Footing (Df/B=1.0) and (b) Surface Footing (Df/B=0.0).](image)
with the increase of the footing pressures. This result is in agreement with work by Gabr et al. (1998). They attributed this behavior to the non-linear stress–strain characteristics of soil and load-dependent soil modulus. It is also indicated in Fig. 14 that I factors for embedded footings are smaller than those for surface footings. This behavior may be expected in the light of the heterogeneity of sand and can be attributed to the variation of sand modulus with confining pressure, which increases with the depth.

4.7. Strain distribution along reinforcement

One laboratory model test using an embedded square footing ($B/L=1.0 \ D/B=1.0$) was conducted to evaluate the strain distribution along the reinforcement. Four layers of GG2 placed at a spacing of 51 mm were used in the test. The geogrids with strain gauge instrumentation were placed at the top and bottom layers [at a depth of 51 mm ($0.33B$) and 203 mm ($1.33B$) below the footing, respectively]. The variations of strains along the centerline of the geogrid at different settlement ratios ($s/B$) are presented in Fig. 15. The tensile strain was the largest at the point beneath the center of the footing, and compressive strains were measured in the geogrid located beyond 0.85$B$ and 1.15$B$ from the center of footing for the geogrid placed at a depth of 51 mm and 203 mm below the footing, respectively. Similar results were also reported by Huang and Tatsuoka (1990) on reinforced sand and by James and Raymond (2002) on reinforced aggregate. The strain developed in the geogrid should be compatible with the strain in the surrounding soil. The greatest lateral movement of sand is expected to occur at the point beneath the center of the footing, and thus the strain in the surrounding soil. The greatest lateral movement of sand is expected to occur at the point beneath the center of the footing where the maximum tensile strain is measured. The measured tensile strain indicates that the geogrid inhibits the lateral movement of sand, and thus improves the sand’s bearing capacity. The compressive strain measured in the geogrid beyond a certain length means that the geogrid past this length is surrounded by soil undergoing compressive strain in the horizontal direction. This compressive zone was clearly shown by Michalowski (2004) through limit analysis. The negligible strain measured at about 3.0$B$ from the center of footing indicates that the geogrid beyond the effective length of $l_e=6.0B$ results in insignificant mobilized tensile strength, and thus provides negligible effects on the improved performance of reinforced sand foundation. The effective length of geogrid reported in the literature varied from 2.0$B$ to 8.0$B$ (e.g., Guido et al., 1986; Omar et al., 1993b; Latha and Somwanshi, 2009a)

5. Ultimate bearing capacity of reinforced sand

5.1. Bearing capacity equation

As indicated in the laboratory model tests, the reinforcement inclusion can lead to an increase in the soil’s bearing capacity. So, the contribution of reinforcements to the bearing capacity needs to be included in the bearing capacity calculation.

To include the contribution of reinforcement, the method of superposition can be used and an additional term, $\Delta q_T$, is added to include the effect of reinforced tensile force $T$. For example, the bearing capacity equation for strip footing will be given in the following form:

$$q_{u(R)} = q_{u(UR)} + \Delta q_T = cN_c + qN_q + 0.5\gamma BN_D + \Delta q_T$$

where $q_{u(UR)}$ is the bearing capacity of unreinforced soil foundation; $\Delta q_T$ is the increased bearing capacity due to the tensile force of the reinforcement; $c$ is the cohesion of soil; $q$ is the surcharge load; $\gamma$ is the unit weight of soil; and $N_c$, $N_q$, and $N_T$ are bearing capacity factors, which are dependent on the friction angle of soil $\phi$.

Based on the limit equilibrium stability analysis of RSF, the following expressions for $\Delta q_T$ can be obtained for reinforced sand [the detailed derivation can be found in Sharma et al. (2009)]:

Stripfooting: $\Delta q_T = \sum_{i=1}^{N} \frac{4T_i[u+(i-1)h]}{B^2}$

Squarefooting: $\Delta q_T = \sum_{i=1}^{N} \frac{12T_i[u+(i-1)h]r_T}{B^2}$
where $T_i$ is the tensile force in the $i$th reinforcement layer (kN); $r_T$ is given below:

$$r_T = \begin{cases} 
\frac{1-2u+(i-1)\frac{h}{2}}{B} \tan \left( \frac{\pi}{4} - \frac{\phi}{2} \right) & \text{for } u + (i-1)\frac{h}{2} < \frac{B}{2} \tan \left( \frac{\pi}{4} + \frac{\phi}{2} \right) \\
\frac{1}{2} \frac{u + (i-1)\frac{h}{2}}{2H_f} & \text{for } u + (i-1)\frac{h}{2} \geq \frac{B}{2} \tan \left( \frac{\pi}{4} + \frac{\phi}{2} \right) 
\end{cases}$$

where $H_f$ is the depth of failure surface and can be evaluated as

$$H_f = \frac{B}{2\cos(\pi/4+\phi/2)}\frac{d}{\tan \phi \cos \phi}$$

5.2. Comparisons between analytical solutions and model test results

The results of laboratory model footing tests conducted in this study provide valuable experimental data for use to compare the analytical solution described herein. The verification of analytical solutions with field model test data reported by Adams and Collin (1997) can be found in Sharma et al. (2009). The tensile force in Eq. (3) was obtained from the measured tensile strain in the experiment.

Fig. 16 presents the comparisons between the BCR values obtained from model footing tests on geosynthetics reinforced sand foundations in this study and those estimated from the analytical solution. The figure shows that the BCR values predicted by using Eq. (3) are generally in good agreement with the test results; with a maximum error of less than 7.2%, except for the surface footing with two layers of GT1 geotextile (overestimated by 19.3% error, which may be due to the slack of woven geotextile caused by test setup).

6. General comments

The benefits of using geosynthetic-reinforced sand foundations were demonstrated in this paper through increasing the soil’s bearing capacity and reducing the footing settlement. When the foundation is built on very weak soil (e.g., compressible, high plasticity clay soils), the reinforced soil mass, as a load transfer platform, creates a composite structure to distribute loads more uniformly over soft foundation soils, thus reducing the stress concentration, which will reduce the consolidation settlement of the underlying weak soil. This will be resulted on smaller foundation size and/or reducing the depth of needed excavation, which will have an economical impact through decreasing material and labor costs. This technology is still under evaluation in Louisiana and no case study is available for the geosynthetic reinforced sand foundation now, however case studies on the benefit of geosynthetic reinforced clayey soil foundation are underway and part of the early results can be found in Chen and Abu-Farsakh (2011).

Reinforcement technique has also been demonstrated effective for improving the seismic performance of earth structures (e.g., Wang et al., 2010). As such, future research on the behavior of reinforced soil foundations under seismic loads is recommended to further explore the benefits of using geosynthetic-reinforced soil foundations.

7. Conclusions

A series of laboratory model footing tests were conducted on geosynthetic reinforced sand foundation in this study to investigate the potential benefits of using reinforcement to improve the bearing capacity and reduce the settlement of shallow foundations on sandy soils. An instrumentation program with pressure cells and strain gauges was designed to investigate the stress distribution in soil mass with and without reinforcement and the strain distribution along the reinforcement. The model footing test results showed (1) an optimum top layer spacing of 0.33B (B: footing width) for the embedded square model footing ($D_0/B=1.0$) on geogrid reinforced sand; (2) an influence depth of 1.25B for placing geosynthetic reinforcement regardless of the type of reinforcement and embedment depth; (3) no optimum vertical spacing for the geogrid reinforced sand tested; (4) a reduction of immediate settlement by 20% at all footing pressure levels with two or more layers of geogrid; (5) better performance of geogrid as compared to geotextile for sandy soil foundation; (6) a redistribution of the foundation loads over a wider area (i.e., more uniformly) below the reinforced zone; (7) a “surcharge effect” brought by the inclusion of geosynthetic reinforcement for surface footing condition; and (8) an effective geosynthetic reinforcement length of 6.0B.

An analytical model to estimate the ultimate bearing capacity of reinforced soil foundation has been developed by the authors in previous studies (Sharma et al., 2009). The results of laboratory model footing tests conducted in this study were used to further verify this previously developed analytical solution; and the agreement between the results of model footing test and the analytical solution is very good with a maximum error of less than 7.2%, except for the surface footing with two layers of GT1 geotextile (overestimated by 19.3%, which may be due to the slack of woven geotextile caused by test setup).

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