An experimental study on characteristics and behavior of reinforced soil foundation

Qiming Chen
Louisiana State University and Agricultural and Mechanical College

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AN EXPERIMENTAL STUDY ON
CHARACTERISTICS AND BEHAVIOR OF
REINFORCED SOIL FOUNDATION

A Dissertation
Submitted to the Graduate Faculty of the
Louisiana State University and
Agricultural and Mechanical College
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Doctor of Philosophy

in

The Department of Civil and Environmental Engineering

By
Qiming Chen
B.S., Nanjing Architecture and Civil Engineering Institute, China, 1997
M.S., Tongji University, China, 2000
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ABSTRACT

This research study aims at investigating the potential benefits of using the reinforcement to improve the bearing capacity and reduce the settlement of shallow foundations. To implement this objective, a total of one hundred seventeen tests were performed to study the behavior of reinforced soil foundation. The test results showed that the inclusion of reinforcement can significantly improve the soil’s bearing capacity and reduce the footing settlement. The geogrids with higher tensile modulus performed better than geogrids with lower tensile modulus. The strain developed along the reinforcement is directly related to the settlement. The test results also showed that the inclusion of reinforcement can redistribute the applied load to a wider area, thus minimizing stress concentration and achieving a more uniform stress distribution. The redistribution of stresses below the reinforced zone can result in reducing the consolidation settlement of the underlying weak clayey soil. Insignificant strain measured in the geogrid beyond its effective length of 4.0–6.0B indicated that the geogrid past this length provides negligible reinforcement effect.

The scale effect on the results of model footing tests was studied using FEA program ABAQUS. Finite element analysis results indicate that the scale effect of reinforced soil foundation is mainly related to the reinforced ratio \( R_r \) of the reinforced zone.

The failure mechanisms of reinforced soil foundation were proposed based on the literature review and experimental test results. Stability analysis including the effect of reinforcement has been conducted based on these proposed failure mechanisms. The new bearing capacity formula with the contribution of reinforcements to an increase in bearing capacity was then developed for reinforced soil foundation. A reasonable estimation on the tensile force along the reinforcement was proposed. The predicted bearing capacities of reinforced soil foundation by using the methods of this study are generally in good agreement with the field test results of previous research for reinforced sand and this study for reinforced silty clay. The proposed methods also provide a good prediction of laboratory model test results of this study.
CHAPTER 1 INTRODUCTION

1.1 Background
Geosynthetics are made from various types of polymers and generally used to provide one or more of the following functions: separation, reinforcement, filtration, or drainage. The most common types of geosynthetics include Geotextiles, Geogrids, Geomembranes, Geosynthetic Clay Liners, Geonets, and Geopipes (Koerner, 1997). One potential application is the use of geosynthetics as reinforcing materials.

In the past half century a significant progress has been achieved in the research and application of reinforced soil earth structures. The concept of reinforced soil is based on the existence of tensile strength of reinforcement and soil-reinforcement interaction due to frictional, interlocking and adhesion properties. It was first commercially introduced in the construction industry by French architect Henri Vidal in 1965. Since then, this technique has been widely used in geotechnical engineering practice. The reinforcing materials range from stiff metal to flexible geosynthetic materials and can be classified as either extensible reinforcements or inextensible reinforcements (McGown et al., 1978).

Geosynthetics have been widely used as reinforcing materials in many geotechnical engineering applications, such as mechanically stabilized earth (MSE) walls, slopes, embankments, pavements, and reinforced soil foundations. Among these applications the use of geosynthetics to reinforce soils to support foundations has recently received more attention.

1.2 Problem Statement
In many cases, shallow foundations are built on top of existing cohesive soil deposits or embankment soils of low to medium plasticity, resulting in low bearing capacity and/or excessive settlement problems. This can cause structural damage, reduction in the durability, and/or deterioration in the performance level. Conventional treatment methods were to replace part of the weak cohesive soil by an adequately thick layer of stronger granular fill, increase the dimensions of the footing, or a combination of both methods. However, an alternative and more economical solution is the use of geosynthetics to reinforce soils. This can be done by either reinforcing cohesive soil directly or replacing the poor soils with stronger granular fill in combination with the inclusion of geosynthetics. The resulting composite zone (reinforced soil
mass) will improve the load carrying capacity of the footing and provide better pressure distribution on top of the underlying weak soils, hence reducing the associated settlements.

One potential application is the use of reinforced soil foundation (RSF) in the design of approach slab for highway engineering applications to minimize the resulting differential settlements. Excessive differential settlement of the concrete approach slab currently causes the significant bridge “bump” problem. This results in uncomfortable rides, dangerous driving conditions, and frequent repairs. One proposed solution is to use rigid approach slab, and transfer the traffic loads to two ends of the slab. Accordingly, a shallow foundation is needed at the end of approach slab far from the bridge to carry that part of load (Figure 1.2.1). To achieve better bearing capacity and/or to prevent excessive settlement, the soil underneath the footing needs to be reinforced.

![Figure 1.2.1 Reinforced soil foundation applied to approach slab](image)

Figure 1.2.1 Reinforced soil foundation applied to approach slab

The benefits of the inclusion of reinforcements within soil mass to increase the bearing capacity and reduce the settlement of soil foundation have been widely recognized. Many hypotheses have been postulated about the failure mode of RSF. However, the mechanism of reinforcement is still not fully understood in RSF. As compared to other reinforced soil applications, the development of design method and theory for RSF is relatively slow. These restrictions, on the other hand, inhibit the further development of reinforcement technology. Therefore, it is important to investigate the reinforcement mechanism for reinforcing intrusion materials for foundation applications.
1.3 Scope and Objectives of the Study

The main objective of this research study is to investigate the potential benefits of using the reinforced soil foundations to improve the bearing capacity and reduce the settlement of shallow foundations on soils. These include (1) examining the influences of different variables and parameters contributing to the improved performance of RSF, (2) investigating the stress distribution in soil mass with and without reinforcement and the strain distribution along reinforcements, (3) understanding the failure mechanism of reinforced soil, (4) developing regression model to estimate the bearing capacity of RSFs, and (5) conducting stability analysis of reinforced soil foundation and developing a step by step procedure for the design of reinforced soil foundation.

To implement objectives of this study, four series of tests were conducted. These include: small-scale laboratory tests on silty clay soil, small-scale laboratory tests on sandy soil, small-scale laboratory tests on crushed limestone, and large-scale field tests on silty clay soil. The parameters investigated in these tests include (i) top layer spacing ($u$), (ii) number of reinforcement layers ($N$), (iii) total depth of reinforcement ($d$), (iv) vertical spacing between reinforcement layers ($h$), (iv) the type and stiffness of reinforcement, (v) the embedment of the footing ($D_f$), (vi) the shape of the footing, and (vii) the type of soil. The experimental study also includes the investigation of the stress distribution in the soil mass with and without reinforcement and the strain distribution along reinforcement. The vertical stress distribution in the soil was measured by placing earth pressure cells at pre-specified locations/depth within the soil. The strain distribution along the reinforcement was recorded using electrical resistance strain gauges that were instrumented at different locations along the reinforcements.

Based on the test results, statistical analyses were performed to develop regression models to predict the bearing capacity of reinforced soil. The scale effects associated with the reduced-scale model tests can be significant as compared to full scale field foundations. Accordingly, a study of the scale effect was conducted by using finite element analysis.

Based on the results of this study, existing analytical solutions were examined and new methods were proposed to calculate the bearing capacity of RSF for different soil type. Typical reinforcement configuration parameters for soil foundation are recommended for design purpose.
1.4 Outline

This dissertation is divided into eight chapters. The following is a brief summary of the contents in each chapter.

Chapter 2 presents an extensive literature review related to experimental study, analytical study, and numerical analysis of reinforced soil foundation.

Chapter 3 describes the materials used in this study and the experimental testing programs, both for small-scale laboratory model tests and large-scale field model tests.

Chapter 4 presents full details of test results and analytical discussion. The comparison of the results of this experimental study to the results of previous studies by different researchers is also provided in this chapter.

In Chapter 5, statistical analyses of the test results are first performed. Regression models are then developed to estimate the bearing capacity of reinforced soil foundations.

Chapter 6 numerically studies the scale effect using axisymmetric finite element analysis.

In Chapter 7, the Stability analysis of reinforced soil foundation includeing the effect of reinforcement is conducted. Existing analytical solutions are examined and new methods are proposed to calculate the bearing capacity of RSF for different soil type.

Finally, Chapter 8 summarizes and concludes this research work and provides some suggestions for future research.
CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

During the past thirty years, many research works have been done to investigate the behavior of reinforced soil foundations (RSF). All these works indicated that the use of reinforcements can significantly increase the bearing capacity and reduce the settlement of soil foundations. Different researchers attempted to evaluate the benefits of using RSFs through bearing capacity ratio (BCR), which is defined as the ratio of the bearing capacity of the RSF to that of the unreinforced soil foundation. Many of these research efforts were aimed at investigating the parameters and variables that would contribute to the BCR value. The results of experimental studies available in the literature showed that better improvements were obtained when the reinforcement is placed within a certain depth beyond which no significant improvement will occur. The parameters studied by researchers include: (1) top layer spacing \( u \), (2) number of layers \( N \), (3) total depth of reinforcement \( d \), (4) vertical spacing between reinforcement \( h \), (5) length of reinforcement \( l \), (6) type and stiffness of reinforcement, (7) soil type, (8) embedment depth of footing \( D_f \), and (9) shape of footing. Figure 2.1.1 depicts a typical geosynthetic reinforced soil foundation and the descriptions of various geometric parameters.

![Figure 2.1.1 Geosynthetic reinforced soil foundation parameters](image-url)
2.2 Experimental Study

Since after Binquet and Lee (1975a) conducted an experimental study to evaluate the bearing capacity of metal strips on reinforced sand soil, numerous experimental studies on the bearing capacity of footings on reinforced sandy soil (e.g., Akinmusuru and Akinbolade, 1981; Fragaszy and Lawton, 1984; Guido et al., 1985; Guido et al., 1986; Huang and Tatsuoka, 1990; Khing et al., 1993; Omar et al., 1993a,b; Shin et al., 1993; Das and Omar, 1994; Yetimoglu et al., 1994; Adams and Collin, 1997; Gabr et al., 1998; Gabr and Hart, 2000; Gnanendran and Selvadurai, 2001; Shin et al., 2002; Michalowski and Shi, 2003), clayey soil (e.g., Ingold and Miller, 1982; Saki and Das, 1987; Mandal and Sah, 1992; Ramaswamy and Purushothaman, 1992; Shin et al., 1993; Das et al., 1996), aggregate (DeMerchant, et al., 2002; James and Raymond, 2002; Sharma, et al., 2004), and pond ash (e.g. Ghosh et al., 2005; Bera et al., 2005) have been reported. The following sections will present the experimental work on RSF

2.2.1 Footings on Reinforced Sandy Soil

2.2.1.1 Geogrid Reinforcement

Khing et al. (1993) conducted a series of model tests on strip footings supported by geogrid reinforced sand. The tests were conducted in a 304.8 mm wide, 1,100 mm long, and 914 mm deep steel box. A hardwood with dimensions of 304.8 mm long, 101.6 mm wide, and 25.4 mm thick was used as model footing. The foundation soil consisted of a uniform fine rounded silica sand having an effective particle size \(D_{10}\) of 0.34 mm, a uniformity coefficient \(Cu\) of 1.53, and a coefficient of curvature \(Cc\) of 1.10. The model tests were conducted at an average dry unit weight of 17.14 kN/m³ \((\text{Dr} = 70\%)\). The corresponding friction angle determined by direct shear tests was about 40.3°.

Khing et al. (1993) reported that the top layer of geogrid showed similar behavior as a rigid rough base with top layer spacing ratio \((u/B)\) greater than unity. The test results indicated that placing the geogrid at a depth ratio \((d/B)\) greater than 2.25 resulted in no improvement on the bearing capacity of strip footing. To achieve maximum benefit, the minimum length ratio \((l/B)\) of the geogrid should be equal to 6. The BCR calculated at a limited settlement ratio \((s/B)\) of 0.25, 0.5 and 0.75 was approximately 67~70% of the ultimate BCR. The test results indicated that the ultimate BCR could be increased up to 4.0 for six layers of reinforcement. They also pointed out that the BCR calculated based on the ultimate bearing capacity and not the limited settlement ratio \((s/B)\) could be misleading for foundation application.
Omar et al. (1993a) investigated the influence of width to length ratio ($B/L$) of footings on the BCR with geogrid reinforcement. They used four model footings with dimensions of 76.2 mm × 76.2 mm, 76.2 mm × 152.4 mm, 76.2 mm × 228.6 mm, and 76.2 mm × 304.8 mm which correspond to width to length ratios ($B/L$) of 1, 0.5, 0.333, and 0.0 (strip footing), respectively. A 0.91 m wide, 0.91 m long, and 0.91 m deep box was used for rectangular footings, while model tests on strip footing were conducted in a 304.8 mm wide, 1.1 m long, and 914 mm deep box. The foundation soil consisted of a uniform fine rounded silica sand having an effective particle size ($D_{10}$) of 0.34 mm, a uniformity coefficient ($C_u$) of 1.53, and a coefficient of curvature ($C_c$) of 1.10. The model tests were conducted at an average dry unit weight of 17.14 kN/m$^3$ ($D_r = 70\%$). The corresponding friction angle determined by direct shear tests was about 41°.

The results of the model tests indicated that the influence depth of reinforcement decreased with increasing the width to length ratio ($B/L$) of the footing. It was about 2B for strip footing and 1.2 B for square footing. The influence depth is the total depth of reinforcement below which the rate of increase in BCR is negligible with an additional reinforcement layer. The maximum BCR also decreased with the increase of the B/L of the footing for $u/B$ and $h/B$ ratios of 0.33 and 0.33 with optimum reinforcement layout. Omar et al. (1993a) also suggested the following empirical relationships for the optimum layout of the reinforcement.

$$\frac{d_{cr}}{B} = \begin{cases} 
2 - 1.4\left(\frac{B}{L}\right) & (0 \leq \frac{B}{L} \leq 0.5) \\
1.43 - 0.26\left(\frac{B}{L}\right) & (0.5 \leq \frac{B}{L} \leq 1) 
\end{cases}$$  \quad (2.1)

$$\frac{b_{cr}}{B} = 8 - 3.5\left(\frac{B}{L}\right)^{0.51}$$ \quad (2.2)

$$\frac{l_{cr}}{B} = 3.5\left(\frac{B}{L}\right) + \frac{L}{B}$$ \quad (2.3)

Where $d_{cr}$ is the influence depth of placing geogrid, $b_{cr}$ and $l_{cr}$ are effective width and length of geogrid. The maximum ultimate BCR achieved in their studies ranges from 3 to 4.5 as $B/L$ varies from 0 to 1.0.

Das and Omar (1994) studied the effects of footing width on BCR of model tests on geogrid reinforced sand. Six different sizes of model strip footings with widths of 50.8 mm, 76.2 mm 101.6 mm, 127 mm, 152.4 mm, and 177.8 mm were used in model tests. The length of all footings is 304.8 mm. Model tests were performed in a box with dimensions of 1.96 m (length) ×
0.305 m (width) × 0.914 m (height). The foundation soil consisted of sand having an effective particle size \((D_{10})\) of 0.34 mm, a uniformity coefficient \((C_u)\) of 1.53, and a coefficient of curvature \((C_c)\) of 1.10. The sand was poured into the box with the relative density of 55%, 65%, and 75%.

From test results, they observed that the settlement ratio \((s/B)\) at ultimate load was about 6-8% for unreinforced soil foundation and 16-23% for RSF at ultimate load. The test results also showed that the magnitude of bearing capacity ratio \((BCR)\) increased from 2.5–4.1 to 3–5.4 with the decrease of the relative density. Based on the test results, they reached the conclusion that the magnitude of BCR decreased from 4.1–5.4 to 2.5–3 with the increase of the footing width and was practically constant \((BCR = 2.5, 2.9, \text{and } 3.0 \text{ for reinforced sand at the relative density of } 75\%, 65\%, \text{and } 75\%, \text{respectively})\) when the width of footings is equal to or greater than 130 ~ 140 mm.

Yetimoglu et al. (1994) investigated the bearing capacity of rectangular footings on geogrid reinforced sand using both laboratory model tests and numerical analyses. The model tests were conducted in a 70 cm wide, 70 cm long, and 100 cm deep steel box. A 127 mm long, 101.5 mm wide, and 12.5 mm thick rectangular steel plate was used as model footing. The foundation soil consisted of a uniform sand having an effective particle size \((D_{10})\) of 0.15 mm, a uniformity coefficient \((C_u)\) of 2.33, and a coefficient of curvature \((C_c)\) of 0.76. The model tests were conducted at an average dry unit weight of 17.16 kN/m³ \((D_r = 70–73\%)\). The corresponding friction angle determined by direct shear tests was about 40º.

The test results showed that the settlement ratio \((s/B)\) was about 0.03 ~ 0.05 at failure for all the unreinforced and reinforced sand, while the BCR ranged from 1.8 to 3.9. Therefore it seems the settlement of the footing at failure might not be influenced significantly by the geogrid reinforcement. This observation of test results is different from that of Das and Omar’s (1994). Based on both the model test results and numerical study, the following findings were reported: (1) the optimum top layer spacing ratio \((u/B)\) was found to be around 0.3 and 0.25 in reinforced sand with single layer and multi-layer reinforcement, respectively, (2) the optimum vertical spacing ratio \((h/B)\) between reinforcement layers was determined as 0.2 to 0.4 depending on the number of reinforcement layers, (3) the influence depth was approximately 1.5B and the effective width ratio \((b/B)\) of reinforcement was around 4.5, (4) increasing the reinforcement stiffness beyond a certain limit would result in insignificant increase in the BCR value. Yetimoglu et al. (1994) believed the disagreement in the results reported by different researchers
might be attributed to the different material properties used in their model tests. As Yetimoglu et al. (1994) pointed out, and Jewell et al. (1984) and Milligan and Palmeira (1987) indicated, the geogrid-soil interaction was influenced significantly by the ratio of minimum grid aperture size ($d_{\text{min}}$) to mean particle size ($D_{50}$). Based on this study, they reached the conclusion that the reinforcement layout had a very significant effect on the bearing capacity of reinforced foundation, especially for the number of reinforcement layers.

Adams and Collin (1997) performed several series of large scale field tests. The tests were conducted in a 6.9 m (length) × 5.4 m (width) × 6 m (height) concrete box with 0.3×0.3 m, 0.46×0.46 m, 0.61×0.61 m, and 0.91×0.91 m square footings. Poorly graded fine concrete mortar sand was selected for tests. The sand had a mean particle size ($D_{50}$) of 0.25 mm, and a uniformity coefficient ($C_u$) of 1.7. The parameters investigated in the tests included the number of reinforcement layers, spacing between reinforcement layers, the top layer spacing, plan area of the reinforcement, and the density of soil.

The test results indicated that three layers of geogrid reinforcement could significantly increase the bearing capacity and that the ultimate bearing capacity ratio (BCR) could be increased to more than 2.6 for three layers of reinforcement. However the amount of settlement required for this improvement is about 20 mm (s/B = 5%) and may be unacceptable on some foundation application. The results also showed that the beneficial effects of reinforcement at low settlement ratio (s/B) could be achieved maximally when top layer spacing is less than 0.25B. They found that the improvement of the RSF was related to the density of sand. Large settlement ratio was required to mobilize the reinforcement in loose sand. The researchers also pointed out the fact that a general failure was less likely to happen if the top layer spacing was less than 0.4B. Adams and Collin (1997) recommended future research to be oriented towards determining the relation between the footing size and the thickness of the reinforced soil mass and comparing the behavior of the different reinforced soils.

Gabr, et al. (1998) used the plate load tests with instrumentation to study the stress distribution in geogrid-reinforced sand. The model tests were conducted in a 1.52 m wide, 1.52 m long, and 1.37 m deep steel box. A 0.33 m wide square footing was used in the test. The foundation soil consisted of Ohio River sand having a uniformity coefficient ($C_u$) of 8, and a coefficient of curvature ($C_c$) of 1. The model tests were conducted at a unit weight ranged from 17.3 kN/m$^3$ to 17.9 kN/m$^3$. The corresponding friction angle determined by triaxial tests was about 38.6º.
Their results showed a better attenuation of the stress distribution due to the inclusion of the reinforcement. The stress distribution angle ($\alpha$) estimated using the measured stress beneath the center of the footing indicates higher values of the angle ($\alpha$) for reinforced sand as compared to unreinforced sand. The stress distribution angle ($\alpha$) decreased with increasing the surface pressure; while the rate of reduction for unreinforced sand is higher than that for reinforced sand.

Gabr and Hart (2000) evaluated the test results in terms of elastic modulus instead of the bearing capacity as traditionally used by other researchers. Test results showed the load-settlement response with the inclusion of geogrid was stiffer than that without geogrid. The elastic modulus of reinforced sand decreased with increasing the top layer spacing ($u$) for SR1 geogrid. On the other hand, optimum top layer spacing ratio of $u/B = 0.65$ was observed for the stiffer geogrid SR2. For one layer of SR1 geogrid, the ratio of modulus constant of reinforced sand ($E_{ref}$) to that of unreinforced sand ($E_{unref}$), $E_{ref}/E_{unref}$, decreased from 1.6 to 1.05 and 1.5 to 1.15 at $s/B = 1.5\%$ and 3\%, respectively as $u/B$ increased from 0.5 to 1.0. At $s/B = 1.5\%$ and 3\%, $E_{ref}/E_{unref}$ for one layer of stiffer geogrid SR2 ranged from 1.0 to 1.3 and 0.96 to 1.3, respectively.

Shin et al. (2002) investigated the influence of embedment on BCR for geogrid reinforced sand. The model tests were conducted in a 174 mm wide, 1000 mm long, and 600 mm deep steel box. A 172 mm long, 67 mm wide and 77 mm thick wood was used as a model strip footing. The foundation soil consisted of a poorly graded sand having an effective particle size ($D_{10}$) of 0.15 mm, a specific gravity of 2.65, and the uniformity coefficient ($C_u$) and coefficient of curvature ($C_c$) equal to 1.51 and 1.1, respectively. The model tests were conducted at an average relative density of $D_r = 74\%$. The corresponding friction angle determined by direct shear tests was about 38°. The top layer spacing ratio ($u/B$), vertical spacing ratio between reinforcement ($h/B$) layers, and length ratio ($l/B$) of the reinforcement were constants for all tests and had a value of 0.4, 0.4, and 6, respectively.

The model test results showed the influence depth for placing reinforcement was about 2B. The BCR at ultimate bearing capacity increased with the increase of the embedment depth of the footing. For footing with embedment depth ratio ($D_r/B$) of 0, 0.3 and 0.6, the ultimate BCR increased from 1.13 to 2.0, 1.25 to 2.5, and 1.38 to 2.65 as the number of reinforcement layer increased from 1 to 6. The BCR values measured at a settlement ratio ($s/B$) less than 5% were smaller than those at ultimate bearing capacity. The BCR with embedment was greater than that without embedment. Although, the magnitude of the ratio of BCR at settlement less than 0.05B to BCR at ultimate bearing capacity decreased with the increase of the depth of the footing.
Guido et al. (1985) conducted an experimental study on the geotextile reinforced sand foundation. Their model tests were conducted in a square Plexiglas box with dimensions of 1.22 m (width) × 0.92 m (height). A 0.31 m wide square footing was used in the test. They conducted 70 plate bearing tests. Twenty-five tests were conducted on a uniformly sand having a mean particle size ($D_{50}$) of 0.18 mm, an effective particle size ($D_{10}$) of 0.086 mm, and a uniformity coefficient ($C_u$) of 2.50 (Case 1). Forty-five tests were conducted on a uniformly sand having a mean particle size ($D_{50}$) of 0.15 mm, an effective particle size ($D_{10}$) of 0.086 mm, and a uniformity coefficient ($C_u$) of 1.90 (Case 2). The model tests for case 1 were conducted at a dry unit weight of 14.80 kN/m$^3$ ($D_r = 50\%$) which corresponds to the friction angle of 35°, while the model tests for case 2 were conducted at a dry unit weight of 14.26 kN/m$^3$ ($D_r = 50\%$), which corresponds to the friction angle of 36°.

They reported that the failure mode of soil switched from general shear failure to local shear failure with the inclusion of the reinforcement. A certain amount of deformation, which was about 0.017B in their study, was needed to mobilize the geotextile. They also reported that the top layer spacing ($u$) and vertical spacing between layers ($h$) depended on each other and should be considered together. Their test results showed that the improvement in bearing capacity was negligible when the reinforcement was placed below an influence depth of 1.0B. The BCR could be increased up to 2.8 for five layers of geotextile with $u/B$ of 0.28 and $h/B$ of 0.18. Increasing the length ratio ($l/B$) of the reinforcement beyond 3.0 resulted in insignificant improvement on the BCR, for two reinforcement layers and $u/B$ of 0.25 and $h/B$ 0.25. The BCR increased with increasing tensile strength.

Guido et al. (1986) conducted an experimental study on the comparison of the bearing capacity of geogrid and geotextile reinforced earth slabs. Their model tests were conducted in a square Plexiglas box with dimensions of 1.22 m (width) × 0.92 m (height). A 305 mm wide square footing was used in the test. The foundation soil consisted of sand having an effective particle size ($D_{10}$) of 0.086 mm, a uniformity coefficient ($C_u$) of 1.90, and a coefficient of curvature ($C_c$) of 1.23. All the model tests were conducted at a dry unit weight of 14.39 kN/m$^3$ ($D_r = 55\%$) with friction angle of 37°. The geogrid and geotextile used in the tests were tensar SS1 geogrid and Du Pont TyPar 3401 geotextile. The ratio of soil-reinforcement friction to soil-soil friction determined by direct shear test was 0.985 for geotextile at a relative density of 55%.
The geogrid failed by tension in pull-out test at a normal stress of 50 kPa and a relative density of 55%.

Guido et al. (1986) showed that the BCR decreased with the increase of $u/B$, but the increase was not significant for $u/B$ greater than 1.0. Decreasing the vertical spacing of the reinforcement resulted in an increase in the BCR values. Their test results also showed that the improvement in bearing capacity was negligible when the number of reinforcement layers was increased beyond 3 layers which correspond to an influence depth of 1.0B for $u/B$, $h/B$, and $l/B$ ratios of 0.5, 0.25, and 3. Negligible improvement on the BCR was observed while increasing the length ratio ($l/B$) of the reinforcement beyond 2 for geogrid reinforced sand and 3 for geotextile reinforced sand with two reinforcement layers and $u/B$ and $h/B$ ratios of 0.25 and 0.25, respectively. Better performance of geogrid reinforced sand than geotextile reinforced sand was observed in their tests. Generally, the BCRs for the geogrid reinforced sand were approximately 10% greater than those for geotextile reinforced sand. The BCR achieved in their studies for geogrid reinforced sand ranged from 1.25 to 2.8.

**2.2.1.3 Other Reinforcing Materials**

Binquet and Lee (1975a) conducted a series of small-scale model tests to simulate three different foundation conditions: (1) a deep homogeneous sand foundation, (2) sand above a deep soft layer of clay or peat (simulated by using a 2.25 in. thick layer of Pack Lite foam rubber), and (3) sand above a soft pocket of material such as clay (simulated by using a 2 in. thick of finite soft pocket Sears foam rubber). Their model tests were conducted in a 60 in. (1,500 mm) long, 20 in. (510 mm) wide, and 13 in. (330 mm) high box. The model footing was a 3 in. (76 mm) wide strip footing. The foundation soil consisted of Ottawa No. 90 sand having a uniformity coefficient ($C_u$) of 1.5 and a coefficient of curvature ($C_c$) of 0.75. All the model tests were conducted at a dry density of 1500 kg/m$^3$. The corresponding friction angles for triaxial and plane conditions were 35° and 42°, respectively. The reinforcement was the household aluminum foil prepared at 0.5 in. (13 mm) wide strips placed along the length of the box, at a linear density of 42.5%, a tensile strength of 0.57kN/m, and a vertical spacing of 1.0 in. (25 mm). The pull out test results showed the peak and residual soil-tie friction angles were 18° and 10°, respectively.

The test results presented by Binquet and Lee (1975a) indicated that the bearing capacity could be improved by a factor ranging from 2 to 4 by reinforcing the soil foundation. They reported that a minimum critical number of reinforcement layers would be required, and that
increasing the number of layers would definitely result in better improvements. Their test results also suggested that the reinforcement placed below the influence depth which was about 2B in their study had negligible effect on the increase of the bearing capacity. The depth to the first reinforcement layers was found to be another significant factor to the success of process. Based on their model tests, and in most cases, placing the first layer at u = 1.0 in. (25 mm) (u/B = 0.3) below the base of the footing resulted in the greatest improvement. They observed that the broken locations of reinforcements were below the edges or toward the center of the footing rather than near the classical slip surface.

Akinmusuru and Akinbolade (1981) investigated the influences of the horizontal spacing, vertical spacing, top layer spacing, and the number of layers on the bearing capacity of reinforced soil. Their model tests were conducted in a square wooden box with dimensions of 39.4 in. (1.0 m) (width) × 28 in. (0.7 m) (height). The model footing was a 0.5 in. (13 mm) thick square steel plate with side length of 4 in. (100 mm). The foundation soil consisted of sand having a mean particle size \(D_{50}\) of 0.43 mm, and an effective size \(D_{10}\) of 0.14 mm. All the model tests were conducted at a dry density of 109 pcf (1700 kg/m³) which corresponded to a friction angle of 38º. The reinforcement was the rope fiber, locally referred to as “iko”. They were prepared at 0.4 in. (10 mm) wide and 0.001 in. (0.03 mm) thick strips, with a breaking strength of 11.6 ksi (80 N/mm²). The pull out test results showed the soil-tie friction angle was 12º.

Akinmusuru and Akinbolade (1981) showed that the ultimate bearing capacity could be improved by a factor up to 3 times that of unreinforced soil. The bearing capacity of reinforced soil foundation increased with increasing the linear density of reinforcement, by reducing the horizontal spacing of the reinforcement. The influence of the vertical spacing was somehow similar to that of the horizontal spacing. The optimum top layer spacing was determined as 0.5 B from their test results. They also showed that the improvement in bearing capacity was negligible when the number of reinforcement layers was increased beyond 3 layers which correspond to an influence depth of 1.75B.

Fragaszy and Lawton (1985) investigated the influence of the reinforcement length and the soil density on the amount of improvement accomplished by using reinforcement in soil foundations. Their model tests were conducted in a rectangular fiberboard box with inside dimensions of 0.56 m (width) × 1.22 m (length) × 0.36 m (height). The model footing was a 7.6 cm × 15.2 cm rectangular steel plate. The foundation soil consisted of sand having a mean
particle size \( (D_{50}) \) of 0.4 mm, a uniformity coefficient \( (C_u) \) of 1.5 and a coefficient of curvature \( (C_c) \) of 0.75. The friction angles corresponding to the densities of 1470, 1540, and 1590 kg/m\(^3\) (relative densities of 31%, 70%, and 90%) were 36.5°, 38°, and 39°, respectively. The reinforcement was the household aluminum foil prepared at 2.54 cm wide and 0.0254 mm thick strips placed at a linear density of 47% and a tensile strength of 1.34 kN/m. All the model tests were conducted with three layers of reinforcement placed at top layer spacing of 2.54 cm \( (u/B \approx 0.33) \) and vertical spacing of 2.54 cm \( (h/B \approx 0.33) \).

Their test results indicated that the amount of improvement in the bearing capacity was dependent on the design criteria. Fragaszy and Lawton (1985) used two criteria: the bearing capacity at a settlement ratio \( (s/B) \) of 0.04 and 0.10. The settlement ratio \( s/B \) is defined here as the ratio of the settlement \( s \) to the width of footing \( B \). At a settlement ratio of 0.04, the bearing capacity ratio (BCR) increased for 1.2 to 1.5 with increasing the density of the soil from 1,490 kg/m\(^3\) to 1,590 kg/m\(^3\), while at a settlement ration of 0.10, the bearing capacity ratio almost kept constant (1.6~1.7) no matter how much the soil density was. Fragazy and Lawton (1985) also showed that the BCR increased from 1.25 to 1.7 with increasing the length of reinforcement from 3 to 7 \( B \), after which the improvement became negligible. They also found out that the design method developed by Binquet and Lee (1975b) is very sensitive to the magnitude of the interface friction coefficient of soil-reinforcement.

Huang and Tatsuoka (1990) conducted a systematic study of bearing capacity of reinforced sand. A 40 cm wide, 183 cm long and 74 cm high sand box was used in their model tests. The model footing was a 10 cm wide strip footing. The foundation soil consisted of Toyoura sand having a mean particle size \( (D_{50}) \) of 0.16 mm, and a uniformity coefficient \( (C_u) \) of 1.46. Five groups of tests were conducted in their study. Short reinforcement having a length of \( L \) equal to \( B \) with different number of layers was used in the first test group. In the second group of tests, the effects of length of reinforcement were studied. The third group of tests were used to study the effects of the number layers. The covering ratio of reinforcement was studied in the fourth group of tests. Four types of reinforcement with different rigidity and rupture strength were used in fifth groups of tests; three of them were Phosphor bronze, the other one was aluminum foil.

The results of their model tests indicated that the bearing capacity could be increased even with short reinforcement of \( L \) equal to \( B \). For surface footings on sand reinforced to a depth of \( d \) less than 0.9\( B \) at a covering ratio (CR) of 18%, the bearing capacity was similar to the footings on unreinforced sand with an embedment depth of \( D \) equal to \( d \). The failure of reinforced sand
with short reinforcement occurred beneath the reinforced zone; and the shear strain distribution beneath the reinforced zone was similar to that beneath the footing with an embedment depth of $D$ equal to $d$. The intensive shear zone along the lateral face of reinforced zone occurred in the case with short reinforcement was spread into larger area for the case with reinforcement longer than $B$. The change of the strain fields was not significant for reinforcement length greater than $2B$. So the effective length of reinforcement was determined as $2.0B$ in their study. Based on these observations, Huang and Tatsuoka (1990) identified two mechanisms that describe the increase in the bearing capacity of reinforced soil foundation: deep footing mechanism and wide-slab mechanism. Their test results also showed there could be an effective covering ratio of reinforcement above which the increase of the bearing capacity would be insignificant. Huang and Tatsuoka (1990) also reported that the reinforcement stiffness had negligible effects on the bearing capacity of strip footing on reinforced sand, unless the reinforcement failed by rupture.

2.2.2 Footings on Reinforced Clayey Soil

2.2.2.1 Geogrid Reinforcement

Ramaswamy and Purushothaman (1992) conducted an experimental study of model footings on the geogrid reinforced clayey soil foundation. A 40 mm-diameter circular footing was used in the test. The foundation soil consisted of clay (CL) having 100% passing the 0.075 mm opening sieve with a specific gravity of 2.66, and liquid and plastic limits equal to 31 and 18, respectively. The maximum dry density of the soil was 1800 kg/m$^3$ with an optimum moisture content of 18% as determined by the Standard Proctor test. Three moisture contents, 14%, 18%, and 20%, were used in the model tests. The corresponding dry densities were 1725 kg/m$^3$, 1810 kg/m$^3$, and 1765 kg/m$^3$, respectively.

The results by Ramaswamy and Purushothaman (1992) showed that the optimum top layer spacing ratio was about 0.5 and the effective length ratio ($l/D$) of the reinforcement was about 4. The BCR increased from 1.15 to 1.70 as the number of layers increased from 1 to 3. The bearing capacity of both unreinforced and reinforced clay decreased with increasing the moisture content. The BCR of reinforced clay with two layers of geogrid at optimum moisture content (=1.47) was higher than those at wet and dry sides (=1.11 and 1.26, respectively).

Mandal and Sah. (1992) conducted a series of model tests on model footings supported by geogrid reinforced clay. The tests were conducted in a 460 mm wide, 460 mm long, and 460 mm deep steel box. A hardwood with dimensions of 100 mm long, 100 mm wide, and 48 mm thick
was used as model footing. The foundation soil consisted of clay (CL) having liquid and plastic limits equal to 72 and 41, respectively. The model tests for clay were conducted at a moisture content of 28%. The corresponding undrained shear strength was about 27 kN/m².

The model test results showed that a maximum BCR was obtained at $u/B=0.175$, while the minimum settlement reduction factor (SRF) at the ultimate bearing pressure of unreinforced clay was obtained at $u/B=0.25$. The settlement reduction factor (SRF) is defined here as the ratio of the immediate settlement of the footing on a reinforced clay to that on an unreinforced clay at a specified surface pressure. The maximum BCR of 1.36 was achieved at $u/B = 0.175$ in their study. With the use of geogrid reinforcement the settlement could be reduced up to 45%.

Shin et al. (1993) conducted an experimental study of strip footings on the geogrid reinforced clay. Their model tests were conducted in a 304.8 mm wide, 1.09 m long, and 0.91 m deep steel box. A 76.2 mm wide strip footing was used in the test. The foundation soil consisted of clay (CL) having 98% passing the 0.075 mm opening sieve with a specific gravity of 2.74, and liquid and plastic limits equal to 44 and 24, respectively. Two moisture contents, 42.5% and 37.7%, were used in the model tests.

Their results indicated that the optimum top layer spacing ratio was about 0.4 and the effective length ratio ($l/B$) of the reinforcement was about 4.5 to 5. The BCR values increased from 1.06 ~ 1.1 to 1.4 ~ 1.45 as the number of layers increased from 1 to 5 and almost kept constant thereafter. The influence depth ratio ($d/B$) of reinforcement was about 1.8B. The increase of BCR with the decrease of the undrained shear strength was observed in their study, but it seems that the undrained shear strength has no effect on the magnitude of the influence depth, based on their limited test results.

Das et al. (1996) conducted a comparative study of strip footings on geogrid reinforced sand and clay. Two 304.8 mm wide, 1.1 m long and 0.91 m high boxes were used in their model tests. The model footings were 76.2 mm wide strip footings. The clay used in the tests had liquid and plastic limits equal to 44 and 24, respectively. The model tests for sand were conducted at a dry unit weight of 17.14 kN/m³ ($D_r = 70\%$) which corresponds to the friction angle of 41° from direct shear tests; while the model tests for clay were conducted at a moisture content of 42.5%, a degree of saturation of 97%, and a wet unit weight of 17.4 kN/m³. The corresponding undrained shear strength determined by laboratory vane shear device was about 3.1 kN/m².

Based on the test results, Das et al. (1996) reported that different optimum parameters for layout of the reinforcement in both soils as shown in the Table 2.1.
Table 2.1 Optimum parameters for layout (Das et al. 1996)

<table>
<thead>
<tr>
<th></th>
<th>Sand</th>
<th>Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>$u/B$</td>
<td>0.3</td>
<td>0.4</td>
</tr>
<tr>
<td>$d/B$</td>
<td>2</td>
<td>1.75</td>
</tr>
<tr>
<td>$l/B$</td>
<td>8</td>
<td>5</td>
</tr>
</tbody>
</table>

Das et al. (1996) also observed that the ultimate bearing capacity could be increased with the inclusion of the reinforcement up to a BCR value of 4.0 and 1.45 for sand and clay, respectively. However, the settlement in reinforced sand is much larger than that in unreinforced sand at the ultimate bearing capacity; while in clay, the difference is insignificant. With increasing the number of layers from 1 to 6, the BCR of reinforced sand and clay varied from 2.1 to 4.0 and 1.1 to 1.45, respectively. They attributed the substantially smaller value of BCR observed in clay to the reason that the passive force between the soil and rib of the geogrid could not be developed in saturated clay because of a zero friction angle. Based on their and other researcher’s limited results, they pointed out that it seems the BCR would keep constant for similar types and layout of geogrid, no matter the values of the relative density of sand or the undrained shear strength of the clay.

2.2.2.2 Geotextile Reinforcement

Saki and Das (1987) conducted an experimental study on the geotextile reinforced clayey soil foundation. Their model tests were conducted in a box with dimensions of 0.652 m (length) × 0.0762 m (width) × 0.61 m (height). A 76.2 mm wide strip footing was used in the test. The foundation soil consisted of clay having 100% passing 2.0 mm opening sieve, 86% passing 0.425 mm opening sieve, 62% passing 0.075 mm opening sieve, and liquid and plastic limits equal to 35% and 24%, respectively. All the model tests were conducted on the moist clay with an undrained shear strength of 22.5 kN/m$^2$, a moisture content of 25.1%, and degree of saturation of 96%.

Their test results demonstrated the following: (1) the most geotextile reinforcement benefit was obtained at a top layer spacing ratio ($u/B$) of 0.35 to 0.4, (2) for $u/B$ of 0.33 and $h/B$ of 0.33, the BCR increased from 1.1 to 1.5 as number of layers increased from 1 to 3 and remained practically constant thereafter. The influence depth of placing geotextile was then determined as 1.0 $B$, and (3) the most effective length of geotextile was equal to 4 times the width of the strip footing.
2.2.3 Footings on Reinforced Aggregate

DeMerchant, et al. (2002) conducted an experimental study of plate load tests on the geogrid reinforced light weight aggregate. Their model tests were conducted in a 2.2 m wide, 3.2 m long, and 1.6 m deep pit. A 305 mm diameter circular footing was used in the test. The foundation soil consisted of light weight aggregate having a grain size distribution between 19 and 4.7 mm, a uniformity coefficient ($C_u$) of 1.4 and a specific gravity in the range of 1.25 to 1.4. The friction angles corresponding to the dry densities of 735 and 832 kg/m$^3$ were 39.5º and 44.5º (from triaxial test), respectively.

The model test results showed that the effective length ratio ($l/B$) of reinforcement was around 4 and the influence depth was approximately 1B. As the top layer spacing ratio ($u/B$) increased from 0.25 to 0.75, the subgrade modulus at $s/B = 2\%$ for single layer of BX1100 and BX1200 geogrid decreased from 82.2 to 52.6 kN/m$^3$ and 49.3 to 38.8 kN/m$^3$, respectively; The subgrade modulus is defined as the applied pressure divided by the corresponding settlement. It was also observed by authors that geogrid with lower stiffness performs better than geogrid with higher stiffness until certain settlement, after which the opposite performance was showed in their study.

Sharma, et al. (2004) investigated the bearing capacity and bulge characteristics of reinforced aggregate piles in clayey soil. A 40 mm-diameter circular plate was used in the test. The foundation soil consisted of crushed stone aggregate having a uniformity coefficient ($C_u$) of 1.52, and a coefficient of curvature ($C_c$) of 1.10. The model tests were conducted at a relative density of 60%. The corresponding peak friction angle of the dry aggregate, as determined by direct shear tests, was about was 38º.

The results of the model tests indicated that the bearing capacity increased from 1.3 to 1.96 and maximum bulge diameter of the reinforced aggregate piles decreased from 6.60 cm to 6.25 cm with an increase of the number of layers from 2 to 5. Their test results also showed that the bearing capacity increased and maximum bulge diameter of the reinforced aggregate piles decreased with a decrease in the vertical spacing.

2.2.4 Footings on Reinforced Pond Ash

Ghosh et al. (2005) investigated the bearing capacity of square footings on reinforced pond ash. Their model tests were conducted in a 0.6 m long, 0.6 m wide and 0.4 m high wooden tank. A 80 mm× 80 mm square footing was used in the model tests. The foundation soil consisted of pond
ash have a specific gravity of 2.16, a uniformity coefficient \((C_u)\) of 3.0, and a coefficient of curvature \((C_c)\) 1.46. The maximum dry density of the soil was 1061 kg/m\(^3\) with an optimum moisture content of 37\% as determined by Standard Proctor test. The corresponding cohesion and internal friction angle of pond ash at optimum moisture content and maximum dry density were 32 kPa and 36\(^\circ\) (from triaxial test), respectively.

Based on the model test results, Ghosh et al. (2005) reached the following conclusions: (1) optimum top layer spacing ratio \((u/B)\) was found to be 0.3125 at any settlement ratio \((s/B)\), (2) the influence depth of placing reinforcement was about 1.75\(B\), (3) The effective length ratio of reinforcement was found to be between 5\(B\) and 7\(B\), and (4) the BCR due to the geotextile reinforcement increases with the increase of friction ratio \((f)\), which defined as the ratio of soil-reinforcement interface friction angle to soil friction angle. Their tests results also indicated the rate of increase of bearing capacity with increase of the number of layers for \(s/B \geq 10\%\) is more significant than for \(s/B \leq 5\%\). For \(u/B\) of 0.3125, the BCR at \(s/B = 10\%\) can be increased from 1.3 to 2.3 with increasing the number of layers of geotextile from 1 to 7.

Based on their experimental model test data of 80 mm square footing on pond ash reinforced by geotextile, Bera et al. (2005) performed the regression analysis of the bearing capacity of reinforced pond ash. A non-linear power model was chosen to relate the bearing capacity of reinforced pond ash \((q_r)\) to the bearing capacity of unreinforced pond ash \((q_{un})\), settlement ratio \((s/B)\), top layer spacing ratio \((u/B)\), number of layers \((N)\), vertical spacing ratio between reinforcement layers \((h/B)\), and length ratio of reinforcement \((l/B)\), friction ratio \((f)\). All possible regression technique was used to select the best model. For the convenience of analysis, nonlinear models were transformed to linear model using logarithmic transformation technique. On the basis of analysis of the results, Bera et al. (2005) obtained the best model for predicting bearing capacity of square footing on pond ash reinforced by geotextile as follows:

\[
q_r = 1.9165q_{un}^{0.777} \left(\frac{s}{B}\right)^{0.3647} N^{0.1901} f^{0.1465} \left(\frac{l}{B}\right)^{0.1111} 0.8105\left(\frac{u}{F}\right) 0.5171\left(\frac{h}{F}\right)
\]

\[(2.4)\]

**2.2.5 Summary of Literature Findings**

The optimum top layer spacing ratio \((u/B)\), the optimum vertical spacing ratio \((h/B)\), the effective length of reinforcement \((l/B)\), and the influence depth ratio \((d/B)\) obtained by different researchers are summarized in Table 2.2 for sand and Table 2.3 for clay.
Table 2.2 Summary of optimum parameters for reinforcement layout in sand

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcement type</td>
<td>Geogrid</td>
<td>geotextile</td>
<td>aluminum foil</td>
<td>rope fiber</td>
<td>phosphor bronze</td>
<td>Geogrid</td>
<td>geotextile</td>
<td>aluminum foil</td>
<td>rope fiber</td>
<td>phosphor bronze</td>
</tr>
<tr>
<td>(u/B)_{opt}</td>
<td>–</td>
<td>No</td>
<td>No</td>
<td>0.25 ~ 0.3</td>
<td>0.3</td>
<td>No</td>
<td>0.3</td>
<td>–</td>
<td>0.5</td>
<td>–</td>
</tr>
<tr>
<td>(h/B)_{opt}</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>0.2 ~ 0.4</td>
<td>–</td>
<td>No</td>
<td>–</td>
<td>–</td>
<td>No</td>
<td>–</td>
</tr>
<tr>
<td>(d/B)_{cr}</td>
<td>1</td>
<td>2.25</td>
<td>2</td>
<td>1.4</td>
<td>1.5</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>–</td>
<td>1.75</td>
</tr>
<tr>
<td>(b/B)_{cr}</td>
<td>2</td>
<td>6</td>
<td>8</td>
<td>4.5</td>
<td>4.5</td>
<td>8</td>
<td>3</td>
<td>–</td>
<td>6 ~ 7</td>
<td>–</td>
</tr>
</tbody>
</table>

Table 2.3 Summary of optimum parameters for reinforcement layout in clayey soil

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcement type</td>
<td>square</td>
<td>circular</td>
<td>strip</td>
<td>strip</td>
<td>strip</td>
</tr>
<tr>
<td>(u/B)_{opt}</td>
<td>0.175</td>
<td>0.5</td>
<td>0.4</td>
<td>0.4</td>
<td>0.35 ~ 0.4</td>
</tr>
<tr>
<td>(h/B)_{opt}</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>(d/B)_{cr}</td>
<td>–</td>
<td>–</td>
<td>1.8</td>
<td>1.75</td>
<td>1</td>
</tr>
<tr>
<td>(b/B)_{cr}</td>
<td>–</td>
<td>4</td>
<td>4.5 ~ 5</td>
<td>5</td>
<td>4</td>
</tr>
</tbody>
</table>


2.3 Analytical Study

Compared to the number of experimental studies, theoretical analysis of bearing capacity of footings on reinforced soil is relatively scarce. The proposed reinforcement mechanisms in the literature can be categorized as follows:

(1) Rigid boundary (Figure 2.3.1a): If the depth to the first layer of reinforcement ($u$) is greater than a specific value, the reinforcement would act as a rigid boundary and the failure would occur above the reinforcement. Binquet and Lee (1975b) were the first who reported this finding. Experimental study conducted by several researches (Akinmusuru and Akinbolade, 1981; Mandal and Sah, 1992; Khing et al., 1993; Omar et al., 1993b; Ghosh et al., 2005) confirmed this finding subsequently.

(2) Membrane effect (Figure 2.3.1b): With the applied load, the footing and soil beneath the footing move downward; the reinforcement is deformed and tensioned. Due to its stiffness, the curved reinforcement develops an upward force to support the applied load. A certain amount of settlement is needed to mobilize tensioned membrane effect and the reinforcement should have enough length and stiffness to prevent it from failing by pull-out and tension. Binquet and Lee (1975b) were perhaps the first who applied this reinforcement mechanism to develop a design method for a strip footing on reinforced sand with the simple assumption made for the shape of reinforcement after deformation. Kumar and Saran (2003) extended this method to a rectangular footing on reinforced sand.

(3) Confinement effect (lateral restraint effect) (Figure 2.3.1c): Due to relative displacement between soil and reinforcement, the friction force is induced at the soil-reinforcement interface. Furthermore, the interlocking can be developed by the interaction of soil and geogrid. Consequently, lateral deformation or potential tensile strain of the reinforced soil is restrained. As a result, vertical deformation of soil is reduced. Since most soils are stress-dependent materials, improved lateral confinement can increase the modulus/compressive strength of soil, and thus improve the bearing capacity. Huang and Tatsuoka (1990) substantiated this mechanism by successfully using short reinforcement having a length ($L$) equal to the footing width ($B$) to reinforce sand in their experimental study. Michalowski (2004) applied this reinforcing mechanism in the
limit analysis of reinforced soil foundation and derived the formula for calculating the ultimate bearing capacity of strip footings on reinforced soils.

2.3.1 Binquet and Lee’s Method

Based on the results and observations of the reduced-scale laboratory model tests, Binquet and Lee (1975b) identified three possible failure mechanism of reinforced soil foundation depending on the tensile strength and configuration of reinforcement: (1) shear failure above uppermost layer of reinforcement which occurs likely when the top layer spacing of reinforcement (u) is greater than 2B/3 (Figure 2.3.2a), (2) pull-out failure (ties pullout) which is likely for the cases where top layer spacing of reinforcement is less than 2B/3 and three or less layers of reinforcement are used, or the length of reinforcement is too short (Figure 2.3.2b), and (3) tension failure (ties break) which is
likely for top layer spacing of reinforcement less than 2B/3, four or more layers of reinforcement, and long reinforcement (Figure 2.3.2c).

a) \( u/B > 2/3 \) : Shear above reinforcements.

b) \( u/B < 2/3 \) & \( N < 2 \) or \( 3 \), or short ties: tie pullout.

c) \( u/B < 2/3 \), long ties and \( N > 4 \): Upper tie break.

Figure 2.3.2 Three modes of failure (after Binquet and Lee, 1975b)

By considering pull-out failure and tension failure, Binquet and Lee (1975b) proposed a design method for strip footing on sand.

According to Binquet and Lee method, the stress distributions within the soil are shown in Figure 2.3.3, which are assumed to be independent of whether or not soil is reinforced. It was assumed that the soil in ZONE 1 moved down with the application of
load, while the soil in ZONE2 moved outward. The boundary (lines a-c and a’-c’) between the ZONE1 and ZONE 2 could be obtained by connecting the points of different depth in the soil at which shear stress, \( \tau_{xz} \), is maximum. The locus of these points could be readily calculated from the elastic theory.

After deformation, the reinforcement at the boundary points was assumed to take the shape as shown in the Figure 2.3.4 b.

Force equilibrium of the element, ABCD, for unreinforced case required (Figure 2.3.4a).

\[
F_t - F_b - S = 0
\]  
(2.5)

where \( F_t \) and \( F_b \) are vertical normal forces acting on the top and bottom faces of the element, \( S \) is the shear force acting on the side of the element.

Force equilibrium of the element for reinforced case required (Figure 2.3.4b).

\[
F_r - F_{br} - S_r - T_t = 0
\]  
(2.6)

where \( F_r \) and \( F_{br} \) are vertical normal forces acting on the top and bottom faces of the element, \( S_r \) is the shear force acting on the side of the element, and \( T_t \) is force developed in the reinforcement.

At the same settlement (\( F_b = F_{br} \))

\[
T_t = F_r - F_t - S_r + S
\]  
(2.7)

Using the Boussinesq’s solution, Binquet and Lee (1975b) derived the following relation for the reinforcement force, \( T_t \), developing at any depth, \( z \):

\[
T_t(z, N) = \frac{1}{N} \left[ J \left( \frac{z}{B} \right) B - I \left( \frac{z}{B} \right) h \right] q_0 \left( \frac{q_r}{q_0} - 1 \right)
\]  
(2.8)

where \( z \) is the depth of reinforcement; \( N \) is the number of reinforcement layers; \( h \) is vertical spacing between layers; \( B \) is the width of the footing; \( q_0 \) is the footing bearing pressure of unreinforced soil foundation; \( q_r \) is the footing bearing pressure of reinforced soil foundation; and \( I \) and \( J \) are dimensionless force and can be computed as:

\[
J \left( \frac{z}{B} \right) = \int_0^x \sigma_z \left( \frac{z}{B} \right) dx 
\]

\[
I \left( \frac{z}{B} \right) = \frac{\tau_{xz max} \left( \frac{z}{B} \right)}{q_r}
\]  
(2.9)
where $X_0$ is the distance of the point at which $\tau_{xz}$ is a maximum; $\sigma_z$ is the vertical stress at the depth $z$; $\tau_{x\text{max}}$ is the maximum shear stress at the depth $z$.

They also expressed allowable tensile resistance, $R_y$, and the pullout resistance, $T_f$ of the reinforcement as follow:

$$R_y = \frac{wN_Rt_f}{FS_y}$$  \hspace{1cm} (2.10)

$$T_f(z) = 2f \cdot LDR\left[M\left(\frac{z}{B}\right)Bq_0\left(\frac{q_R}{q_0}\right) + \gamma(L_0 - X_0)(z + D)\right]$$  \hspace{1cm} (2.11)

where $w$ is the width of a single tie, $t$ is the thickness of a single tie, $N_R$ is the number of ties per unit length of strip footing, the product of $w$ and $N_R$ is called the linear density of reinforcement ($LDR$), $f_y$ is the yield or breaking strength of the tie material, $FS_y$ is the factor of safety against reinforcement breakage, $f$ is allowable soil-tie coefficient of friction expressed as $(\tan \phi_f/FS_f)$, $\phi_f$ is the soil-tie friction angle, $FS_f$ is the factor of safety for the pullout, and $M$ is a dimensionless force.

---

Figure 2.3.3 Stress distributions below strip footing and failure mechanism (after Binquet and Lee, 1975b)
2.3.2 Huang and Tatsuoka’s Method

Based on the results and observations of the laboratory model tests of strip footing on reinforced sand, Huang and Tatsuoka (1990) described two possible failure modes of reinforced soil foundation: (1) local failure in the unreinforced zone beneath the reinforced zone (Figure 2.3.5a), and (2) local failure within the reinforced zone (Figure 2.3.5b). This type of failure generally included pull-out failure of reinforcement, tension failure of reinforcement, and compression failure in soil. They also proposed a simple method for predicting the bearing capacity increase ($\Delta q$) of strip footings in reinforced sand.

Figure 2.3.5 Two failure modes of reinforced sand (after Huang and Tatsuoka, 1990)
For failure mode 1:

\[
\Delta q = K_p \frac{\gamma_d}{2} \left[ \frac{2d + b + 2s_2}{2} - \frac{c + s_1}{2} \right] + \frac{2}{B} \left[ \sum_{i=1}^{N} T_{i,t} \cdot \tan \phi \cdot N_i \right]
\] (2.12)

For failure mode 2:

\[
\Delta q = K_p \frac{\sum_{i=1}^{N} T_{\text{av},i} \cdot N_i}{d}
\] (2.13)

Where, \( K_p = \tan^2 (45^\circ + \phi/2) \), \( \phi \) is the internal friction angle of sand in the corresponding plane strain compression (PSC) test at \( \delta = 90^\circ \) (\( \delta \) is the angle between major stress (\( \sigma_1 \)) direction and the bedding plane), \( \gamma_d \) is the dry unit weight of sand, \( d \) is the total depth of reinforcement, \( b \) and \( s_2 \) are the height of block B beneath the reinforced zone and the settlement of footing at failure for reinforced sand, \( c \) and \( s_1 \) are the height of block beneath the footing and the settlement of footing at failure for unreinforced sand, \( N \) is the number of reinforcement layers, \( T_{i,t} \) is the tensile force in each in strip in the layer \( i \) at the lateral face of the block \( A \), \( N_i \) is the number of reinforcements per unit length in the layer \( i \), \( B \) is the width of the footing, and \( T_{\text{av},i} \) is the average tensile force at the layer \( i \) in the block \( A \). The features of this method are summarized in Table 2.4.

Table 2.4 Terms to be checked for reinforced soil (Huang and Tatsuoka, 1990)

<table>
<thead>
<tr>
<th>Terms Checked</th>
<th>Checked</th>
</tr>
</thead>
<tbody>
<tr>
<td>Insufficient length</td>
<td>No (never occurs)</td>
</tr>
<tr>
<td>Bond failure between soil and reinforcement</td>
<td>Yes</td>
</tr>
<tr>
<td>Insufficient covering ratio</td>
<td>Yes</td>
</tr>
<tr>
<td>Insufficient ( \phi ) in soil</td>
<td>Yes</td>
</tr>
<tr>
<td>Insufficient rupture strength</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Local failure within the reinforced zone

- Pull-out failure of reinforcement
- Compressive failure in soil
- Tension failure of reinforcement

Local failure within the unreinforced zone

- Compressive failure in soil
- Insufficient \( \phi \) in soil
2.3.3 Huang and Menq’s Method

Huang and Menq (1997) evaluated the reinforced soil foundation based on a failure mechanisms proposed by Schlosser et al. (1983) as shown in Figure 2.3.6. According to this failure mechanism, two reinforcing effects, i.e. deep footing and wide-slab effects, would contribute to the increase of bearing capacity. The basic concept of this failure mechanism is that the bearing capacity of footing (width: B) on reinforced soil foundation is equivalent to that of a wider footing (width: B+ΔB) at the depth of d (total depth of reinforcement) on unreinforced soil foundation.

![Figure 2.3.6 Failure mechanism of reinforced soil foundation (after Huang and Menq, 1997)](image)

For unreinforced soil foundation:

\[
q_u(\text{unreinforced}, D_f=0) = \eta \times \gamma \times B \times N_q
\]  

(2.14)

For reinforced soil foundation:

\[
q_u(R) = q_u(\text{unreinforced}, D_f) = \eta \times \gamma \times (B + \Delta B) \times N_q + \gamma \times d \times N_q
\]  

(2.15)

Where \(q_u(\text{unreinforced}, D_f=0)\) is the ultimate bearing capacity of unreinforced soil foundation with footing on surface; \(q_u(R)\) is the ultimate bearing capacity of reinforced soil foundation; \(D_f\) is the embedment of footing; \(\eta\) is an coefficient depending on footing shape; \(\gamma\) is the dry unit weight of soil; \(B\) is the width of footing; \(N_q\), \(N_q\) are bearing capacity factors; \(d\) is the total depth of reinforcement; \(\Delta B\) is the increase of footing width due to the inclusion of reinforcement, \(=(2 \times d) \tan \alpha\); \(\alpha\) is the stress distribution angle due to the wide-slab effect as shown in Figure 2.3.6. Based on experimental data from different researchers, Huang and Menq (1997) did regression analysis and obtained the following equations to estimate the stress distribution angle, \(\alpha\).

\[
\tan \alpha = 0.680 - 2.071h / B + 0.743CR + 0.03l / B + 0.076N
\]  

(2.16)
Where $h$ is the vertical spacing between reinforcement layers; $CR$ is the covering ratio of reinforcement = the area of reinforcement divided by the area of soil covered by reinforcement; $l$ is the length of reinforcement; $N$ is the total number of reinforcement layers.

2.3.4 Wayne et al’s Method

Wayne et al. (1998) suggested four possible failure modes for reinforced soil foundations as shown in Figure 2.3.7. The control failure mode depends on the reinforcement configuration and soil conditions.

If the depth to the first layer of reinforcement ($u$) is greater than a specific value, the reinforcement would act as a rigid boundary and the failure would occur above the reinforcement (Figure 2.3.7a). This kind of failure can be avoided by placing top layer of reinforcement close to the footing.

If the vertical spacing between reinforcement layers ($h$) is too large, the failure would occur between the reinforcements (Figure 2.3.7b). This kind of failure mode can be excluded by keeping a proper vertical spacing of reinforcement layers.

Punching failure along reinforced zone (Figure 2.3.7c) can occur when reinforcement is not long enough and the reinforced zone is very strong. For this kind of failure, the reinforced mass acts as a rigid deep footing, and the thickness of reinforced zone can be treated as embedment depth of footing. The bearing capacity of reinforced soil foundation can then be calculated by classic bearing capacity formula presented by researchers such as Vesic (1973).

Punching failure through reinforced zone (Figure 2.3.7d) commonly occurs in reinforced soil foundations with proper reinforcement configuration. For this kind of failure, the reinforced soil foundation was treated as a two layer soil system by Wayne et al. (1998), i.e. stronger soil underlying weaker soil. Meyerhof and Hanna formula was so modified to include the contribution of reinforcement to the increase in bearing capacity as shown in equation 2.17.

$$q_{u(b)} = q_b + 2c_f(B + L)\frac{d}{BL} + \gamma_h H^2 \left[1 + 2\frac{D}{d}\right] K_s(B + L)\frac{\tan \phi}{BL} + 2(B + L)\frac{T}{BL} - \gamma_h d$$

(2.17)
Where $q_b$ is the ultimate bearing capacity of the foundation below the reinforced zone; $c_t$ is the cohesion of the upper layer; $\gamma_t$ is the unit weight of the upper layer; $d$ is the thickness of the upper layer; $B$ is the width of the footing; $L$ is the length of the footing; $D_f$ is the embedment depth of the footing; $\phi$ is the friction angle of the upper layer; $K_s$ is the punching shear coefficient for upper layer, which is the function of the friction angle and dependent on the ultimate bearing capacities of surface footing on upper and lower soil layers; $T$ is the uplift or restraining force of the reinforcements.

### 2.3.5 Michalowski’s Method

Michalowski (2004) conducted stability analysis of reinforced soil foundation based on upper-bound theorem. Two failure mechanisms, i.e. pull-out failure and tension failure, were considered in his study. Applying superposition principle in the analysis, Michalowski (2004) suggested the following formulas for calculating the bearing capacity of strip footings on soil reinforced with horizontal layers of geosynthetics.

*(i) For tension failure:*

$$p = cN_c + qN_q + \frac{1}{2}\gamma BN_f + k_iM'_t$$

where $k_i = \frac{T_i}{h}$, $M'_t = (1 + \sin \phi)e^{(\pi/2+\phi)\tan \phi}$, $T_i$ is tensile strength of reinforcement, $h$ is vertical spacing between reinforcement layers

*(ii) For pull out failure:*

1. Single layer of reinforcement:

$$p = \frac{1}{1 - \mu \frac{u}{B} M_p} \left[ c(N_c + f_c M_c) + q(N_q + \mu M_q) + \gamma B \left( \frac{1}{2} N_f + \mu \frac{d}{B} M_f \right) \right]$$

Where $N_c = (N_q - 1)\cot \phi$, $N_q = \tan \left( \frac{\pi}{4} + \frac{\phi}{2} \right) e^{\tan \phi}$, $N_f = e^{0.66 + 5.11 \tan \phi}$, $f_c$ is the ratio of soil-reinforcement interface cohesion to the soil cohesion.
Figure 2.3.7 Possible failure modes of reinforced soil foundation (after Wayne et al., 1998)
2. Multi-layer of reinforcement:

\[ p = \frac{1}{1 - \mu M_p} \left[ c(N_c + n f^c M_c) + q(N_q + n \mu M_q) + \gamma B \left( \frac{1}{2} N_c + \mu M_q \sum_{i=1}^{n} \frac{d_i}{B} \right) \right] \] (2.20)

Where \( n \) is number of layers, \( d_i \) is depth of the \( i \)th layer, \( = u + (i-1)h \), \( h \) is vertical spacing between reinforcement layers.

Numerical results obtained by Micholowski (2004) showed that placing the reinforcement above point B and C in Figure 2.3.8 would mobilize the maximum benefit of the reinforcement. The expressions to estimate the bearing capacity coefficients for such cases were given approximately by Michalowski (2004) and summarized in Table 2.5.

![Figure 2.3.8 Mechanism of foundation soil collapse (after Michalowski, 2004)](image-url)

Table 2.5 Bearing capacity coefficients due to reinforcement (Michalowski, 2004)

<table>
<thead>
<tr>
<th>Number of layers</th>
<th>( M_c = M_q = M_p )</th>
<th>( M_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single layer</td>
<td>( 1.6(1 + 8.5 \tan^{13} \phi) )</td>
<td>( 1.5 - 1.25 \times 10^{-2} \phi )</td>
</tr>
<tr>
<td>Two layers</td>
<td>( 1.1(1 + 10.6 \tan^{13} \phi) )</td>
<td>( 4.075 - 6.25 \times 10^{-3} \phi )</td>
</tr>
<tr>
<td>Three layers</td>
<td>( 0.9(1 + 10.6 \tan^{13} \phi) )</td>
<td>( 0.5 - 6.25 \times 10^{-2} \phi )</td>
</tr>
</tbody>
</table>

2.4 Numerical Analysis

Numerical analysis is a powerful mathematic tool that enables us to solve complex engineering problems. The finite element method is the most popular and well-established numerical analysis technique. It has been widely used in many civil engineering applications both for research and design of real engineering problems. One such application is the numerical analysis of reinforced soil foundation problem.
The performance of reinforced soil structures depends not only on soil and reinforcement properties but also on the interaction between the soil and reinforcement. For this reason, finite element procedure becomes complex as compared to the simulation of regular soil foundation. In the past, while a lot of research work has been conducted to simulate the reinforced soil in pullout loading conditions, finite element analysis for investigating the behavior of reinforced soil foundation can also be found in several literatures (e.g., Yetimoglu et al., 1994; Kurian et al., 1997; Yamamoto and Otani, 2002; Maharaj, 2002). Numerical modeling of reinforced soil foundation presented by researchers can be categorized into two groups:

The first group models the reinforcement and soil as two separate components (e.g., Yetimoglu et al., 1994; Kurian et al., 1997; Maharaj, 2002). The reinforcement is generally treated as a linear elastic material in these studies. The soil model used by different researchers includes Ducan-Chang model (Kurian et al., 1997), Drucker-Prager model (Maharaj, 2002), and Modified Duncan hyperbolic model (Yetimoglu et al., 1994). Soil-reinforcement interface are generally modeled using two approaches: constraint approach and contact elements. The constraint approach generally assumes that separation is not allowed between the soil and reinforcement in normal direction, while in tangential direction slip can occur. In the use of the contact element, the normal stiffness is often given a very high value to prevent interpenetration of nodes.

The second group treats reinforced soil as an equivalent homogeneous continuum media (e.g., Yamamoto and Otani, 2002). Yamamoto and Otani (2002) used Drucker-Prager model for reinforced sand and included the effect of the reinforcement in pseudo cohesion, \( c_R \).

\[
c_R = \frac{T \sqrt{K_p}}{2h}
\]

Where \( c_R \) is pseudo cohesion, \( T \) is the mobilized tension of geosynthetics, \( h \) is the vertical spacing of geosynthetics, \( K_p \) is passive pressure coefficient, \( = \tan^2(45^\circ + \phi/2) \).

Kurian et al. (1997) investigated the settlement of footing on reinforced sand by using 3D finite element analysis. The results of the analysis were then compared with those from laboratory model tests. 8-node brick element was used to discretize the soil, while 3D truss element was used to discretize the reinforcement. A 3D soil-reinforcement
interface friction element developed on the basis of Goodman element was used in the analysis. Both the reinforcement and the interface elements were geometrically 3D line elements. The stress-strain behavior of sand was modeled by Ducan-Chang model, while the footing and the reinforcements were assumed to be linearly elastic. The sand used in their study had an effective particle size ($D_{10}$) of 0.23 mm, a uniformity coefficient ($C_u$) of 1.34, and a Poisson’s ratio of 0.3. The friction angle determined by triaxial tests was about 38°.

Kurian et al. (1997) reported that there was a clear reduction of settlement in the reinforced sand at higher loads as compared to unreinforced sand. The numerical results also indicated that a small increase in settlement occurred in reinforced sand at the initial stage of loading process. A possible explanation of this phenomenon given by Kurian et al. (1997) was that the normal load was too small to mobilize enough friction between soil and reinforcement, i.e. a weak plane was initially presented with the inclusion of reinforcement. The relative movement between soil and reinforcement increased with the increase of load and decreased with increase of reinforcement depth. The maximum shear stress at the soil-reinforcement interface occurred at a relative distance ($x/B$) of about 0.5 from the center of the footing. The tension developed in reinforcement was maximum at the center and gradually decreased towards to the end of the reinforcement. As compared to unreinforced sand, the vertical stress contours shifted downwards in reinforced sand, i.e. spreading the stress deeper.

Maharaj (2002) investigated the influences of top layer spacing, vertical spacing of reinforcement layers, size of the reinforcement and the number of layers on the settlement of strip footing on reinforced clay using two dimensional nonlinear finite element analysis. The footing and soil were discretized into four node isoparametric finite elements while the reinforcement was discretized into four node one dimensional isoparametric elements. Drucker-Prager yield criteria was used to model clay. The footing and reinforcement were assumed to be linear elastic material. The clay used in the model had a Poisson’s ratio of 0.45 and an elasticity modulus of 13,000 kN/m². The cohesion intercept and friction angle of clay were 10.84 kN/m² and 0°. The stiffness of reinforcement used in the model ranged from 500 kN/m to 20,000 kN/m.
Based on numerical study, the following findings were reported by Maharaj (2002): (1) in the case of single layer of reinforcement, the optimum top layer spacing ratio \((u/B)\) was found to be around 0.125 in reinforced clay, (2) the effective length ratio \((l/B)\) of reinforcement was around 2.0, (3) the influence depth depended on the stiffness of reinforcement, and (4) the increase in geosynthetics’ stiffness resulted in reducing the settlement of the footing.

Yamamoto and Otani (2002) investigated the bearing capacity and failure mechanisms of reinforced granular material by using rigid-plastic finite element analysis. The numerical results were then compared with In their analysis, the reinforced soil was treated as a composite material. A Drucker-Prager model was used for modeling this composite material. The cohesion and the friction angle of soil were determined as 0.49 kN/m² and 25° by back analysis of the model tests results.

Their numerical study indicated that at the same settlement ratio \((s/B)\) the area of plastic flow for the reinforced foundation was wider and deeper compared with that of unreinforced foundation. Therefore, the bearing capacity of reinforced foundation was improved. The area of plastic flow became even wider as the length of reinforcement increased. The similar phenomenon was obtained for the contour line distribution of the equivalent plastic strain rate.
CHAPTER 3 EXPERIMENTAL TESTING PROGRAM

3.1 Introduction

The testing program designed in this study aimed at investigating the potential benefits of using the reinforced soil foundations to improve the bearing capacity and to reduce the settlement of shallow foundations on soils. For this purpose, two series of tests, small-scale laboratory tests and large-scale field model tests were conducted to investigate the influence of different parameters involved in the design. The experimental study also includes the investigation of the stress distribution in the soil mass with and without the inclusion of reinforcement, and the strain distribution along the reinforcement.

3.2 Testing Materials Properties

3.2.1 Soil

Three different types of geomaterial (sand, silty clay, and Kentucky crushed limestone soils) were used in the present study. The physical properties of sand are summarized in Table 3.1, and the grain-size distribution curve of sand is shown in Figure 3.2.1. Based on Unified Soil Classification System (USCS) and American Association of State Highway and Transportation Officials (AASHTO) classification system, this sand is classified as SP and A-1-b, respectively. The friction angles at different dry densities determined by large scale direct shear test are summarized in Table 3.2.

In many coastal areas of the United States, high quality embankment soils are not locally available and marginal cohesive soils are often encountered. The silty clay soil used in the present study was a marginal embankment soil with low to medium plasticity that is often encountered in embankments in Southern Louisiana. The physical properties of the silty clay are summarized in Table 3.3. The soil has a maximum dry density of 1670 kg/m$^3$ and an optimum moisture content of 18.75% as determined by standard proctor test. From the atterberg limits test, the silty clay is classified as CL according to the USCS, and A-6 according to the AASHTO classification system. The shear strength parameters determined by large scale direct shear test at optimum moisture content of 18.75% with different densities are summarized in Table 3.4.

Figure 3.2.2 depicts the grain-size distribution curve of Kentucky crushed limestone. Table 3.5 summarizes the physical properties of crushed limestone. The shear strength parameter obtained from large scale direct shear test at optimum moisture content of 7.5% and maximum
dry density of 2268 kg/m³, as determined by standard proctor test, is φ = 53° and c = 0 kPa. This crushed limestone is classified as GW and A-1-a according to USCS and AASHTO classification system, respectively.

Table 3.1 Properties of sandy soil

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective particle size ($D_{10}$)</td>
<td>0.226 mm</td>
</tr>
<tr>
<td>Mean particle size ($D_{50}$)</td>
<td>0.45 mm</td>
</tr>
<tr>
<td>Uniformity coefficient ($C_u$)</td>
<td>2.07</td>
</tr>
<tr>
<td>Coefficient of curvature ($C_c$)</td>
<td>1.25</td>
</tr>
<tr>
<td>Maximum dry density#</td>
<td>1620 kg/m³</td>
</tr>
<tr>
<td>Optimum moisture content#</td>
<td>4.8%</td>
</tr>
</tbody>
</table>

\# Standard Proctor test

Table 3.2 Friction angle versus dry density of sandy soil

<table>
<thead>
<tr>
<th>Dry Density (kg/m³)</th>
<th>Friction Angle (φ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1592</td>
<td>40.0º</td>
</tr>
<tr>
<td>1620</td>
<td>41.5º</td>
</tr>
<tr>
<td>1646</td>
<td>42.7º</td>
</tr>
<tr>
<td>1686</td>
<td>44.1º</td>
</tr>
<tr>
<td>1714</td>
<td>45.0º</td>
</tr>
<tr>
<td>1764</td>
<td>47.8º</td>
</tr>
</tbody>
</table>

Table 3.3 Properties of silty clay/embankment soil

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid limit</td>
<td>31</td>
</tr>
<tr>
<td>Plastic index</td>
<td>15</td>
</tr>
<tr>
<td>Silt content</td>
<td>72%</td>
</tr>
<tr>
<td>Clay content</td>
<td>19%</td>
</tr>
<tr>
<td>Maximum dry density#</td>
<td>1670 kg/m³</td>
</tr>
<tr>
<td>Optimum moisture content#</td>
<td>18.75%</td>
</tr>
</tbody>
</table>

\# Standard Proctor test
Table 3.4 Cohesion and friction angle versus dry density of silty clay

<table>
<thead>
<tr>
<th>Dry Density (kg/m³)</th>
<th>Moisture Content (%)</th>
<th>Cohesion (kPa)</th>
<th>Friction Angle (φ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1525</td>
<td>18.75</td>
<td>5.06</td>
<td>25.96</td>
</tr>
<tr>
<td>1670</td>
<td>18.75</td>
<td>13.19</td>
<td>25.11°</td>
</tr>
<tr>
<td>1763</td>
<td>18.75</td>
<td>24.58</td>
<td>24.13°</td>
</tr>
</tbody>
</table>

Table 3.5 Properties of Kentucky crushed limestone

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective particle size ($D_{10}$)</td>
<td>0.465 mm</td>
</tr>
<tr>
<td>Mean particle size ($D_{50}$)</td>
<td>5.56 mm</td>
</tr>
<tr>
<td>Uniformity coefficient ($C_u$)</td>
<td>20.26</td>
</tr>
<tr>
<td>Coefficient of curvature ($C_c$)</td>
<td>1.37</td>
</tr>
<tr>
<td>Maximum dry density$^#$</td>
<td>2268 kg/m³</td>
</tr>
<tr>
<td>Optimum moisture content$^#$</td>
<td>7.5%</td>
</tr>
</tbody>
</table>

$^\#$ Standard Proctor test

Figure 3.2.1 Grain-size distribution curve of sand
3.2.2 Reinforcement

Nine types of geosynthetics (eight geogrid types and one geotextile type), one type of steel wire mesh and one type of steel bar mesh were used in the present study. The physical and mechanical properties of these reinforcements as provided by the manufacturers are summarized in Table 3.6

3.3 Testing Program and Sample Preparation Techniques

3.3.1 Small-Scale Laboratory Tests

The small-scale laboratory tests were conducted at the Geotechnical Engineering Research Laboratory (GERL) of the Louisiana Transportation Research Center (LTRC). The model tests were conducted inside a steel box with dimensions of 1.5 m (length) × 0.91 m (width) × 0.91 m (height). The model footings used in the tests were 25.4 mm thick steel plates with dimensions of 152 mm × 152 mm and 152 mm × 254 mm. The footings were loaded with a hydraulic jack against a reaction steel frame (Figure 3.3.1). The testing procedure was performed according to the ASTM D 1196-93 (ASTM 1997), where the load increments were applied and maintained until the rate of settlement was less than 0.03 mm/min for three consecutive minutes. The load and the corresponding footing settlement were measured using a ring load cell and two dial gauges, respectively.

The soil was placed and compacted in lifts inside the steel box. The thickness of each lift varies from 25 mm to 102 mm depending on reinforcement spacing. The amount of soil needed for each lift was calculated first. The test samples were prepared by hand mixing pre-weighted
Table 3.6 Properties of reinforcement

<table>
<thead>
<tr>
<th>Type</th>
<th>Reinforcement</th>
<th>Polymer Type</th>
<th>T&lt;sub&gt;a&lt;/sub&gt;, kN/m</th>
<th>J&lt;sub&gt;b&lt;/sub&gt;, kN/m</th>
<th>Aperture Size, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>MD&lt;sup&gt;a&lt;/sup&gt;</td>
<td>CD&lt;sup&gt;d&lt;/sup&gt;</td>
<td>MD&lt;sup&gt;a&lt;/sup&gt;</td>
</tr>
<tr>
<td>GG1</td>
<td>Mirafi BasXgrid11 geogrid&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Polyester</td>
<td>7.3</td>
<td>7.3</td>
<td>365</td>
</tr>
<tr>
<td>GG2</td>
<td>Tensar BX6100 geogrid&lt;sup&gt;2&lt;/sup&gt;</td>
<td>Polypropylene</td>
<td>3.6</td>
<td>5.1</td>
<td>182</td>
</tr>
<tr>
<td>GG3</td>
<td>Tensar BX6200 geogrid&lt;sup&gt;2&lt;/sup&gt;</td>
<td>Polypropylene</td>
<td>5.5</td>
<td>7.4</td>
<td>274</td>
</tr>
<tr>
<td>GG4</td>
<td>Tensar BX1100 geogrid&lt;sup&gt;2&lt;/sup&gt;</td>
<td>Polypropylene</td>
<td>4.1</td>
<td>6.6</td>
<td>205</td>
</tr>
<tr>
<td>GG5</td>
<td>Tensar BX1200 geogrid&lt;sup&gt;2&lt;/sup&gt;</td>
<td>Polypropylene</td>
<td>6.0</td>
<td>9.0</td>
<td>300</td>
</tr>
<tr>
<td>GG6</td>
<td>Tensar BX1500 geogrid&lt;sup&gt;2&lt;/sup&gt;</td>
<td>Polypropylene</td>
<td>8.5</td>
<td>10.0</td>
<td>425</td>
</tr>
<tr>
<td>GG7</td>
<td>Tenax MS330 Geogrid&lt;sup&gt;3&lt;/sup&gt;</td>
<td>Polypropylene</td>
<td>6.1</td>
<td>9.0</td>
<td>305</td>
</tr>
<tr>
<td>GG8</td>
<td>Mirafi Miragrid 8XT geogrid&lt;sup&gt;4&lt;/sup&gt;</td>
<td>Polyester</td>
<td>16</td>
<td>16</td>
<td>80</td>
</tr>
<tr>
<td>GT1</td>
<td>Mirafi HP570 geotextile&lt;sup&gt;5&lt;/sup&gt;</td>
<td>Polypropylene</td>
<td>14</td>
<td>19.3</td>
<td>700</td>
</tr>
<tr>
<td>SWM</td>
<td>Steel Wire Mesh</td>
<td>Stainless Steel</td>
<td>236</td>
<td>447</td>
<td>11780</td>
</tr>
<tr>
<td>SBW</td>
<td>Steel Bar Mesh</td>
<td>Steel</td>
<td>970</td>
<td>970</td>
<td>48480</td>
</tr>
</tbody>
</table>

<sup>a</sup>Tensile Strength (at 2% strain), <sup>b</sup>Tensile Modulus (at 2% strain), <sup>c</sup>Machine Direction, <sup>d</sup>Cross machine direction

<sup>1</sup>http://www.mirafi.com/products/product_basx_index2.html
<sup>2</sup>http://www.tensarcorp.com/uploadedFiles/SPECTRA_MPDS_BX_8.05.pdf
<sup>4</sup>http://www.mirafi.com/products/product_xt_index2.html
<sup>5</sup>http://www.mirafi.com/products/product_hp_index2.html

soil and water. Then, the soil was poured into the box, leveled, and compacted using a 203 mm × 203 mm plate adapted to a vibratory jack hammer to the predetermined height. The jackhammer delivers compaction energy of 58.3 m·N and blows at a rate of 1400 per minute. The compaction started on one side and proceeded to the other side.

The quality control of compaction and repeatability of test sections were a major source of discrepancy for such material. Accordingly, 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. The nuclear density gauge and the geogauge stiffness device were used to measure the density and stiffness modulus for each lift.

As indicated in literature review, several parameters are crucial for design of reinforced soil foundation (RSF). The purpose of these model tests was to examine the influences of the following parameters on the benefit of RSF:

i) depth ($u$) to the first reinforcement layer,
ii) number of reinforcement layers ($N$),
iii) vertical spacing between reinforcement layers ($h$),
iv) the type and tensile modulus of reinforcement,
v) the embedment of the footing ($D_f$),
vi) the shape of footing, and
vii) the type of soil.
Tables 3.7 through 3.9 present the detail test factorial of laboratory model tests used in this research study. The laboratory experimental study also included the investigation of the stress distribution in soil with and without reinforcement and the strain distribution along the reinforcement. The vertical stress distribution in the soil was measured by Model 4800 VW earth pressure cells (4 in. diameter) from Geokon Inc. installed within the soil mass. 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 reinforcements. Figure 3.3.2 and 3.3.3 depict typical layouts of instrumentation (pressure cells and strain gage) used for laboratory model tests on silty clay and sand soils, respectively; while the corresponding plane layouts of pressure cells are shown in Figure 3.3.4 and 3.3.5, respectively.

Three lead-wire quarter-bridge technique was used in this study. Compared to two-wire quarter-bridge, three-wire quarter-bridge can eliminate initial imbalance problem of the Wheatstone bridge, reduce desensitization due to leadwire resistance, and cancel the influence of temperature changes in the leadwire on measured strain.

The pressure cells and strain gauges were connected to the terminal board of Geokon model 8032 16/32 channel multiplexer. The channels are protected against voltage surges with tripolar plasma surge arrestors and bipolar surge arrestors. The Geokon Model 8020 MICRO-10 Datalogger was connected to multiplexers to read the sensors. Multilogger software package from Canary Systems Inc. was used to manage the data acquisition hardware (Datalogger) which was connected directly to the PC. Figure 3.3.6 depicts the whole instrumentation system set-up.

3.3.2 Large-Scale Field Tests

The large-scale model tests were performed in an outdoor test pit constructed next to the LTRC building. The test pit has a dimension of 3.658 m (12 ft) (length) × 3.658 m (12 ft) (width) × 1.829 m (6 ft) (height). The side walls of the test pit were built using the silty clay soil and have a slope of 1:1, as shown in Figure 3.3.7. Reaction steel piles were installed on each side of the test foundation and connected with a steel beam. A hydraulic jack against the steel beam provided downward load. A load cell was placed between the jack and the foundation to measure the applied load. The settlement was measured using dial gages mounted on reference beams, as shown in Figure 3.3.8. The model footing used in the field tests was 203 mm (8 in.) thick steel-reinforced precast concrete block with dimensions of 457 mm (1.5 ft) × 457 mm (1.5 ft).
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Footing Dimensions</th>
<th>Embedment mm</th>
<th>Reinforcement configuration</th>
<th>$u$ mm</th>
<th>$h$ mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>CNR*</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>Unreinforced</td>
<td>...</td>
<td>...</td>
</tr>
<tr>
<td>CGG11-1</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td></td>
<td>25</td>
<td>...</td>
</tr>
<tr>
<td>CGG11-2</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td></td>
<td>51</td>
<td>...</td>
</tr>
<tr>
<td>CGG11-3</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td></td>
<td>76</td>
<td>...</td>
</tr>
<tr>
<td>CGG11-4</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=1, BasXgrid11</td>
<td>102</td>
<td>...</td>
</tr>
<tr>
<td>CGG11-5</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td></td>
<td>127</td>
<td>...</td>
</tr>
<tr>
<td>CGG11-6</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td></td>
<td>152</td>
<td>...</td>
</tr>
<tr>
<td>CGG11-7</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td></td>
<td>203</td>
<td>...</td>
</tr>
<tr>
<td>CGG12</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=2, BasXgrid11</td>
<td>51</td>
<td>51</td>
</tr>
<tr>
<td>CGG13</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=3, BasXgrid11</td>
<td>51</td>
<td>51</td>
</tr>
<tr>
<td>CGG14</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=4, BasXgrid11</td>
<td>51</td>
<td>51</td>
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<tr>
<td>CGG15*</td>
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<td>N=5, BasXgrid11</td>
<td>51</td>
<td>51</td>
</tr>
<tr>
<td>CGG21</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=1, BX6100</td>
<td>51</td>
<td>51</td>
</tr>
<tr>
<td>CGG22</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=2, BX6100</td>
<td>51</td>
<td>51</td>
</tr>
<tr>
<td>CGG23</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=3, BX6100</td>
<td>51</td>
<td>51</td>
</tr>
<tr>
<td>CGG24*</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=4, BX6100</td>
<td>51</td>
<td>51</td>
</tr>
<tr>
<td>CGG25*</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=5, BX6100</td>
<td>51</td>
<td>51</td>
</tr>
<tr>
<td>CGG31</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=1, BX6200</td>
<td>51</td>
<td>...</td>
</tr>
<tr>
<td>CGG32</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=2, BX6200</td>
<td>51</td>
<td>51</td>
</tr>
<tr>
<td>CGG33-1*</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=3, BX6200</td>
<td>51</td>
<td>25</td>
</tr>
<tr>
<td>CGG33-2*</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=3, BX6200</td>
<td>51</td>
<td>51</td>
</tr>
<tr>
<td>CGG33-3*</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=3, BX6200</td>
<td>51</td>
<td>76</td>
</tr>
<tr>
<td>CGG33-4*</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=3, BX6200</td>
<td>51</td>
<td>102</td>
</tr>
<tr>
<td>CGG34*</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=4, BX6200</td>
<td>51</td>
<td>51</td>
</tr>
<tr>
<td>CGG35*</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=5, BX6200</td>
<td>51</td>
<td>51</td>
</tr>
<tr>
<td>CGT11</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=1, HP570</td>
<td>51</td>
<td>...</td>
</tr>
<tr>
<td>CGT12</td>
<td>152 mm×152 mm</td>
<td>0</td>
<td>N=2, HP570</td>
<td>51</td>
<td>51</td>
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<td>h mm</td>
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<td>--------------</td>
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<td>51</td>
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<td>51</td>
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<td>N=2, HP570</td>
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<td>N=3, HP570</td>
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<td>N=4, HP570</td>
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<tr>
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<td>...</td>
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<td>N=1, BasXgrid11</td>
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Table 3.8 (continued)

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<th>Instrument Type</th>
<th>Notes</th>
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<td>152</td>
<td>N=1,BasXgrid11</td>
<td></td>
</tr>
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<td>152</td>
<td>N=1,BasXgrid11</td>
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</tr>
<tr>
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<td>152 mm×152 mm</td>
<td>152</td>
<td>N=1,BasXgrid11</td>
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</tr>
<tr>
<td>SDGG81-3</td>
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<td>N=1,BasXgrid11</td>
<td></td>
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<tr>
<td>SDGG81-4</td>
<td>152 mm×152 mm</td>
<td>152</td>
<td>N=1,BasXgrid11</td>
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</tr>
<tr>
<td>SDGG12</td>
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<td>152</td>
<td>N=2,BasXgrid11</td>
<td></td>
</tr>
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<td>152 mm×152 mm</td>
<td>152</td>
<td>N=3,BasXgrid11</td>
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<td>152 mm×152 mm</td>
<td>152</td>
<td>N=3,BasXgrid11</td>
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<tr>
<td>SDGG13-3</td>
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<td>152</td>
<td>N=3,BasXgrid11</td>
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<td>152 mm×152 mm</td>
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<td>N=4,BasXgrid11</td>
<td></td>
</tr>
<tr>
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<td>N=1,BX6100</td>
<td></td>
</tr>
<tr>
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<td>152</td>
<td>N=2,BX6100</td>
<td></td>
</tr>
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<td>N=4,BX6100</td>
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</tr>
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<td>152</td>
<td>N=1,HP570</td>
<td></td>
</tr>
<tr>
<td>SDGT12*</td>
<td>152 mm×152 mm</td>
<td>152</td>
<td>N=2,HP570</td>
<td></td>
</tr>
<tr>
<td>SDGT13*</td>
<td>152 mm×152 mm</td>
<td>152</td>
<td>N=3,HP570</td>
<td></td>
</tr>
<tr>
<td>SDGT14*</td>
<td>152 mm×152 mm</td>
<td>152</td>
<td>N=4,HP570</td>
<td></td>
</tr>
<tr>
<td>SDGGT11</td>
<td>152 mm×152 mm</td>
<td>152</td>
<td>N=1,Composite</td>
<td></td>
</tr>
<tr>
<td>SDGGT12</td>
<td>152 mm×152 mm</td>
<td>152</td>
<td>N=2,Composite</td>
<td></td>
</tr>
<tr>
<td>SDGGT13</td>
<td>152 mm×152 mm</td>
<td>152</td>
<td>N=3,Composite</td>
<td></td>
</tr>
<tr>
<td>SDGGT14*</td>
<td>152 mm×152 mm</td>
<td>152</td>
<td>N=4,Composite</td>
<td></td>
</tr>
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<td>SDFGG14*</td>
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<td>152</td>
<td>N=4,BasXgrid11</td>
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<td>152 mm×254 mm</td>
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<td>N=4,BX6100</td>
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<td>SDFGT14*</td>
<td>152 mm×254 mm</td>
<td>152</td>
<td>N=4,HP570</td>
<td></td>
</tr>
<tr>
<td>SDFGGT14*</td>
<td>152 mm×254 mm</td>
<td>152</td>
<td>N=4,Composite</td>
<td></td>
</tr>
</tbody>
</table>

* Instrumented with pressure cell
# Instrumented with strain gage
The soil selected for large-scale model tests was the silty clay soil. Figures 3.3.8 and 3.3.9 show the front and side view of the test setup. The large-scale tests were performed according to the ASTM D 1196-93 (ASTM 1997). A total of 6 large-scale field tests were conducted. Table 3.10 presents the test factorial for this research study.

The soil was placed and compacted in lifts. The amount of soil needed for each lift was calculated first. The test samples were prepared by using tiller to mix the pre-weighted soil and water. Then, the soil was poured into the test pit, raked level, and compacted using a MultiQuip plate compactor and a wacker-packer tamper to the predetermined height to achieve the desired densities. The MultiQuip plate compactor delivers 3,450 pounds (1,565 kg) of compaction force.
Figure 3.3.2 Typical layout of instrumentation for laboratory model tests on silty clay soil

Figure 3.3.3 Typical layout of instrumentation for laboratory model tests on sand soil
FIG 3.3.4 Plane layout of pressure cells for laboratory model tests on silty clay soil

FIG 3.3.5 Plane layout of pressure cells for laboratory model tests on sand soil
Figure 3.3.6 Instrumentation system

and has a 500 mm × 526 mm (19.7 in. × 20.7 in.) plate. The wacker-packer tamper delivers compaction force of 3,300 lbs (1,497 kg), blows at a rate of 700 per minute, and has a 254 mm × 305 mm (10 in. × 12 in.) plate.

The compaction-quality control processes were accomplished by conducting three passes with the MultiQuip plate compactor followed by six passes with the wacker-packer tamper. As we did early in laboratory test, the nuclear density gauge and the geogauge stiffness device were used to measure the density and stiffness modulus for each lift.

The purpose of these large-scale tests was to study the behavior of reinforced soil foundation under the field condition. The stress distribution in the soil with and without reinforcement and the strain distribution along the reinforcement were also evaluated in this series of field tests. Figure 3.3.10 depicts typical layout of instrumentations (pressure cells and strain gage) used for large-scale field tests.
Figure 3.3.7 Large-scale field test setup, loading, and reaction system

Figure 3.3.8 Large-scale field test setup – front view
Table 3.10 Test factorial for large-scale field tests on silty clay soil

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Soil</th>
<th>Footing Dimensions</th>
<th>Embedment mm</th>
<th>Reinforcement configuration</th>
<th>u mm</th>
<th>h mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLNR*</td>
<td>Silty clay</td>
<td>457 mm×457 mm</td>
<td>0</td>
<td>None</td>
<td>...</td>
<td>...</td>
</tr>
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<td>Silty clay</td>
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<td>N=4, BX6100</td>
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<td>203</td>
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<td>305</td>
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<td>N=4, BX6200</td>
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<td>203</td>
</tr>
<tr>
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<td>N=5, BX6200</td>
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<td>152</td>
</tr>
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<td>N=4, BX1500</td>
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<td>203</td>
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</table>

* Instrumented with pressure cell  
# Instrumented with strain gage
Figure 3.3.10 Typical layout of instrumentation for large-scale field tests on silty clay soil
CHAPTER 4 ANALYTICAL DISCUSSION OF TEST RESULTS

4.1 Introduction

Two terms are used in the present study to evaluate the benefits of using RSF. The bearing capacity ratio (BCR) is defined as the ratio of the bearing capacity of the RSF to that of the unreinforced; and the settlement reduction factor (SRF) is defined as the ratio of the settlement of the RSF to that of the unreinforced.

Two different types of load-settlement behavior were observed in the model footing tests. For the first type of load-settlement curve as show in Figure 4.1.1a, the failure point is not well defined. The benefits of using RSF are then evaluated in terms of BCR at a specific settlement (BCRs) and SRF at a specific surface pressure. Figure 4.1.1b depicts the second type of load-settlement curve which has a well defined failure point. For this type of load-settlement behavior, BCR at a specific settlement (BCRs), BCR at the ultimate bearing capacity (BCRu) and SRF at a specific surface pressure are used to evaluate the improved performance of RSF.

![Figure 4.1.1 Definition of BCR and SRF](image)

The optimal values for reinforcement layout and the effect of types of reinforcement and soil are determined based on BCR and SRF in this study. This analytical discussion also includes the characterization of RSF, stress distribution in soil with and without reinforcement, and strain distribution along the reinforcement. The results of this experimental study are also compared with results of previous studies by different researchers.
4.2 Small-Scale Laboratory Tests on Reinforced Silty Clay

The main objective of this research was to investigate the potential benefits of using the RSFs to improve the bearing capacity and reduce the settlement of shallow foundations on cohesive soils of low to medium plasticity. For this purpose, extensive laboratory model tests were conducted on geosynthetic reinforced clayey soils. The parameters investigated in the model tests include the top layer spacing ($u$), the number of reinforcement layers ($N$), the vertical spacing between reinforcement layers ($h$), the tensile modulus and type of reinforcement, and shape of footing. The experimental study also includes investigating the stress distribution in clay and the strain distribution along the reinforcement.

Three types of geogrids: BasXgrid11, BX6100 and BX6200, and one type of geotextile, HP570, were used as reinforcement in the tests. The physical and mechanical properties of these reinforcements are presented earlier in Table 3.6.

The dry densities measured by nuclear density gauge varied from 1,640 to 1,709 kg/m$^3$ for geogrid reinforced silty clay embankment soil and from 1,601 to 1,644 kg/m$^3$ for geotextile reinforced silty clay embankment soil. Both had moisture contents ranging from 18 to 18.5%. The corresponding geogauge stiffness moduli were in the range of 100 to 120 MPa for silty clay soil in all tests with/without reinforcement inclusion.

The results of the laboratory model tests for silty clay embankment soil are summarized in Table 4.1. In this table the BCRs obtained at settlement ratios ($s/B$) of 3%, 10% and 16%, are presented. The settlement ratio ($s/B$) is defined as the ratio of footing settlement ($s$) to footing width ($B$). The results of the model footing tests are also graphically presented in Figures 4.2.1 through 4.2.7. Figure 4.2.1 presents the pressure-settlement curves measured for model footing tests with a single layer of BasXgrid11 placed at different top layer spacing. The measured pressure-settlement curves for model footing tests with different numbers of reinforcing layers are presented in Figures 4.2.2 through 4.2.6. Figure 4.2.7 depicts the pressure-settlement curves obtained from model footing tests using three layers of BX6200 placed at different vertical spacing. Investigating the pressure-settlement curves, one can see that the pressure keeps increasing with an increase in the settlement for both unreinforced and reinforced silty clay. This settlement pattern resembles a typical punching-shear failure. Mandal and Sah (1992) also reported the same observation for 100mm-wide square footing on clay reinforced by one layer of
geogrid. Since the failure point is not well defined, the bearing capacity is obtained at different settlement ratios and used to calculate the corresponding BCRs.

Table 4.1 Summary of laboratory model tests for silty clay embankment soil

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<tr>
<th>Test No.</th>
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<th>u mm</th>
<th>h mm</th>
<th>s/B = 3% q, kPa</th>
<th>BCR</th>
<th>s/B = 10% q, kPa</th>
<th>BCR</th>
<th>s/B = 16% q, kPa</th>
<th>BCR</th>
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<td>...</td>
<td>...</td>
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<td>570</td>
<td>...</td>
<td>687</td>
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<td>358 1.00</td>
<td>586</td>
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<td>737</td>
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**Table 4.1 (continued)**

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* Instrumented with pressure cell

4.2.1 Effect of Reinforcement Top Layer Spacing

The optimum location of the first reinforcement layer (top layer spacing) for silty clay embankment soil was investigated using a 152 mm-wide square footing and with BasXgrid11 geogrid reinforcement. The measured pressure-settlement curves for soil without reinforcement and for soils with one layer of reinforcement placed at different top spacing are presented in Figure 4.2.1.

At settlement ratios of s/B=3%, 10%, and 16%, Figure 4.2.1 shows that the bearing capacities of the reinforced silty clay soil increase from 359 kPa, 587 kPa, and 729 kPa to 365 kPa, 590 kPa, and 745 kPa, respectively.
Figure 4.2.2 Pressure-settlement curves for model footing tests with different number of layers of BasXgrid11 geogrid in silty clay (B×L: 152 mm×152 mm)

Figure 4.2.3 Pressure-settlement curves for model footing tests with different number of layers of BX6100 geogrid in silty clay (B×L: 152 mm×152 mm)
Figure 4.2.4 Pressure-settlement curves for model footing tests with different number of layers of BX6200 geogrid in silty clay (B×L: 152 mm×152mm)

Figure 4.2.5 Pressure-settlement curves for model footing tests with different number of layers of HP570 geotextile in silty clay (B×L: 152 mm×152mm)
Figure 4.2.6 Pressure-settlement curves for model footing tests with different number of layers of BX6100 geogrid in silty clay (B×L: 152 mm×254mm)

Figure 4.2.7 Pressure-settlement curves for model footing tests with three layers of BX6200 placed at different vertical spacing in silty clay (B×L: 152 mm×152mm)
kPa, 609 kPa, and 770 kPa, respectively, as the top layer spacing \((u)\) increases from 25 mm to 51 mm. It then decreases to 352 kPa, 526 kPa, and 715 kPa as \(u\) increases from 51 mm to 203 mm.

The bearing capacity reached its maximum value when \(u\) equals to 51 mm. Figure 4.2.8 depicts the variation in BCR values of the loads corresponding to settlement ratio \(s/B=3\%\), 10\%, and 16\% as a function of the top layer spacing ratio \((u/B)\). Top layer spacing ratio \((u/B)\) is defined as the ratio of top layer spacing \((u)\) to footing width \((B)\). Figure 4.2.8 shows that the BCRs at different settlement ratios increase with increasing the top layer spacing ratios \((u/B)\) up to a maximum value at \(u/B = 0.33\), after which it decreases. Based on the laboratory test results, the optimum location of the top layer is then estimated to be about 51 mm, which is equivalent to 0.33B.

This finding is similar to that reported by Sakit and Das (1987) and Shin et al. (1993). Sakit and Das (1987) reported that the maximum value of BCR was obtained at a depth of 0.35B for strip footing on both single and multilayer geotextile reinforced clay. Shin et al. (1993) indicated that the optimum location of the top layer was at a depth of about 0.4B for strip footing on clay with four layers of geogrid. On the other hand, based on a 40 mm-diameter circular footing on clay reinforced by one layer of reinforcement, Ramaswamy and Purushothaman (1992) obtained a maximum BCR at \(u/B=0.5\). Mandal and Sah (1992) indicated that the optimum top layer spacing was about 0.175B for 100 mm-wide square footing on clay with a single layer of geogrid.

This discrepancy on the optimum location of the top layer reinforcement may be attributed to the different properties of soil and reinforcement used by different researchers.

Figure 4.2.8 BCR versus \(u/B\) for one layer of BasXgrid11 at different settlement ratios \((s/B)\) in silty clay soil \((B\times L: 152 \text{ mm}\times 152\text{ mm})\)
4.2.2 Effect of Number of Reinforcement Layers

A series of laboratory model footing tests were conducted on the silty clay embankment soil reinforced with multiple layers of four different types of geosynthetics placed at a spacing of 51 mm. Figure 4.2.2 through 4.2.6 present the pressure-settlement curves of these model tests. As expected, the bearing capacity increased with increasing the number of reinforcement layers. For example, at settlement ratios of \( s/B = 3\% \), 10\%, and 16\%, the bearing capacities of a 152 mm-wide square footing on silty clay reinforced by BX6200 geogrid increase from 433 kPa, 691 kPa, and 877 kPa to 538 kPa, 931 kPa, and 1246 kPa, respectively as number of reinforcement layers increases from 1 to 5. However, the significance of an additional reinforcement layer decreases with the increase in the number of layers. This effect becomes negligible below the influence depth. The influence depth is the total depth of reinforcement below which the rate of increase in BCR is negligible with an additional reinforcement layer. This tendency can be seen from Figures 4.2.2 through 4.2.6, in which the pressure-settlement curves for the model tests are very close for four and five layers of geogrid, and three and four layers of geotextile. The virtually identical measured stresses at the same depth under the center of the footing for four and five layers of BX6200 geogrid reinforced silty clay also confirm this finding. This will be further discussed in a later section 4.2.6 when the stress distribution in the silty clay embankment soil is discussed. The variations of BCRs obtained at settlement ratios of \( s/B = 3\% \), 10\%, and 16\% for different numbers of reinforcement layers (\( N \)) and reinforcement depth ratios (\( d/B \)) are shown in Figures 4.2.9 for 152 mm-wide square footing, and Figure 4.2.10 for 152 mm×254mm rectangular footing. The reinforcement depth ratio is defined as the ratio of the total depth of reinforcement (\( d \)) to footing width (\( B \)). It can be seen from these Figures that the BCRs increase with \( N \) and \( d/B \), and appear to become almost constant after \( N=4 \) (\( d/B=1.33 \)) for geogrid reinforced silty clay and after \( N=3 \) (\( d/B=1.25 \)) for geotextile reinforced silty clay. Accordingly, the influence depth can be estimated to be 1.5B for geogrid reinforced silty clay and 1.25 B for geotextile reinforced silty clay. The influence depth seems to be independent of the footing shape based on the test results of this study. Similar to this finding, Saki and Das (1987) indicated that the geotextile placed below 1.0B could not improve the bearing capacity of clay. Shin et al. (1992) reported the influence depth for a strip footing on geogrid reinforced clay was approximately 1.8B.
Figure 4.2.9 BCR versus $N$ and $d/B$ at different settlement ratios (s/B) for silty clay soil (B×L: 152 mm×152 mm)
Figure 4.2.9 (continued)
4.2.3 Effect of Vertical Spacing of Reinforcement Layers

The effect of vertical spacing of reinforcement layers was investigated using 152 mm-wide footing and three layers of BX6200 geogrid reinforcement with a top layer spacing of 51 mm (0.33B). The vertical spacing of reinforcement varied from 0.167B to 0.667B. At settlement ratios of $s/B=3\%$, 10\%, and 16\%, Figure 4.2.7 shows that the bearing capacity of the reinforced silty clay decreases from 552 kPa, 978 kPa, and 1227 kPa to 470 kPa, 768 kPa, and 1001 kPa, respectively as the vertical spacing ($h$) increases from 25 mm to 102 mm. Figure 4.2.11 depicts the variation in the BCR values of the loads corresponding to settlement ratios of $s/B=3\%$, 10\%, and 16\% as a function of the 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 decrease with increasing vertical spacing of reinforcement layers with maximum BCR at $h = 0.167B$ in the present study. No optimum vertical spacing was obtained for the BX6200 geogrid reinforced silty clay tested. Similar results were reported by Ingold and Miller (1982) on geogrid reinforced clay. 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 modulus and size. Guido et al. (1985) indicated that it is difficult to fully understand the effect of vertical spacing on bearing
capacity separately without considering other influencing factors. However, for the silty clay and geogrid reinforcement tested in this study, one can realize that the smaller the spacing, the higher the BCR. In design, engineers have to balance between using smaller spacing and using higher modulus geogrid. The effect of geogrid modulus will be discussed later in Section 4.2.5. The author believes a value of $h/B = 0.2$ can be a reasonable value for use in the design of reinforced silty clay.

![Figure 4.2.11 BCR versus $h/B$ at different settlement ratios (s/B) for three layers of BX6200 in silty clay (B×L: 152 mm×152 mm)](image)

**4.2.4 Effect of Footing Shape**

The effect of footing shape on the BCR of reinforced soils was investigated by conducting two sets of model tests, one with a 152 mm×152 mm square footing, and one with a 152 mm×254 mm rectangular footing. The test results show that the bearing capacity of unreinforced silty clay for 152 mm-wide square footing is greater than that for 152 mm×254 mm rectangular footing (Table 4.1), which is consistent with the theoretical analysis by using bearing capacity formula suggested by Vesic (1973). Similarly trend was also found in reinforced silty clay. The comparison of BCRs obtained for these two different shape footings is shown in Figure 4.2.12. From this figure, it is clear to see that the BCRs for 152 mm-wide square footing are generally greater than those obtained for 152 mm×254 mm rectangular footing.
<table>
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<th>h (mm)</th>
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<tr>
<td>152 x 254</td>
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</table>

(a) $s/B = 3\%$

(b) $s/B = 10\%$

(c) $s/B = 16\%$

Figure 4.2.12 BCR versus type of reinforcement for two different size footing
4.2.5 Effect of Tensile Modulus and Type of Reinforcement

Four different types of reinforcement with different strength/modulus were used in the model footing tests. These include BasXgrid11 geogrid, BX6100 geogrid, BX6200 geogrid, and HP570 geotextile. The properties of these reinforcements were presented earlier in Table 3.6. Figures 4.2.13 to 4.2.18 rearrange the pressure-settlement curves obtained for different types of reinforcements to compare the results of model tests with same reinforcement configuration. The BX6100 and BX6200 geogrids are made of the same material and have similar aperture size, but BX6200 has higher tensile strength/modulus than BX6100 (Table 3.6). As seen in Figures 4.2.13 through 4.2.17, the silty clay reinforced by BX6200 geogrid performed better than that reinforced by BX6100 geogrid. It is also noted that the behavior of these two geogrids is very similar until a certain amount of settlement is reached enough to mobilize the geogrid. The Figures also show that BasXgrid11 geogrid, which has the highest stiffness and smallest aperture size out of the three types of geogrid used in present study, has the best performance. As shown in Figure 4.2.19, this study demonstrates that the performance of reinforced silty clayey soil improves with increasing geogrid tensile modulus. However, the effect of the tensile modulus seems to be a function of settlement. In support of this finding, lower stresses were measured for higher tensile modulus geogrids at the same depth under the center of the footing as will be discussed later in section 4.2.6.

The BCRs at different settlement ratios \((s/B)\) for model tests with multiple layers of different types of reinforcement are presented in Figures 4.2.20a to 4.2.20d. It can be seen that the BCR generally increases with the increase of settlement ratio \((s/B)\). At relatively low settlement ratio \((s/B)\), the increase of the bearing capacity of silty clay soil reinforced with geogrids is more significant than those with HP570 geotextile of higher tensile modulus. However, with the increase of settlement ratio \((s/B)\), the BCRs of HP570 geotextile reinforced silty clay increased more quickly than those of geogrid reinforced silty clay. Figures 4.2.13 through 4.2.19 also show that the silty clay reinforced by geogrids performs better than that reinforced by HP570 geotextile at relative low settlement, while the response of HP570 geotextile reinforced silty caly is stiffer than geogrid reinforced silty clay after reaching a certain amount of settlement, which depends on the number of reinforcement layers. This behavior can be attributed to the slack of woven geotextile. 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 silty clay-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 stiffness out of four types of reinforcement used in this study, the reinforcing effect of
geotextile would be more appreciably mobilized.

It is interesting to mention here that the change of stresses in HP570 geotextile reinforced
silty clay and geogrid reinforced silty clay with the increase of footing pressure has the same
trend (see further discussion in section 4.2.6).

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 silt clay foundation than geotextile. 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 pressure (q) for the model tests
with multiple layers of different types of reinforcements are presented in Figures 4.2.21a through
4.2.21d. It is obvious that the inclusion of the reinforcement would reduce the immediate
settlement significantly. With three or more layers of reinforcement, the settlement can even be
reduced by 50% at relatively medium footing pressure (400kPa). The geogrids with higher
tensile modulus provide the better reduction in immediate settlement than the lower tensile
modulus geogrids. The SRF values of geotextile reinforced silty clay are generally lower than
those of geogrid reinforced silty clay. The SRF values associated with geotextile at low stresses
with N equal to/less than 3 are even greater than 1.0. This behavior may be attributed to the slack
effect of woven geotextile. The rate of decrease of SRF with the increase of footing pressure for
geotextile reinforced silty clay is higher compared to that for geogrid reinforced silty clay. In all
cases, the SRFs become stabilized at a footing pressure of 500 kPa and higher.

4.2.6 Stress Distribution in Silty Clay
Several laboratory model tests were conducted to evaluate the stress distributions in the silty
clayey soil with and without reinforcement inclusion induced by footing load. Pressure cells
were placed at specified locations/depth for this purpose. The induced vertical stress distributions
along the center line of the footing at the depth of 254 mm (1.67B) are shown in Figure 4.2.22
for 152 mm-wide footing with different number of layers of BX6200 geogrid and in Figure
Figure 4.2.13 Pressure-settlement curves for model footing tests with one layer of different types of reinforcements (B×L: 152 mm×152mm)

Figure 4.12.14 Pressure-settlement curves for model footing tests with two layers of different types of reinforcements (B×L: 152 mm×152mm)
Figure 4.2.15 Pressure-settlement curves for model footing tests with three layers of different types of reinforcements (B×L: 152 mm×152mm)

Figure 4.2.16 Pressure-settlement curves for model footing tests with four layers of different types of reinforcements (B×L: 152 mm×152mm)
Figure 4.2.17 Pressure-settlement curves for model footing tests with five layers of different types of reinforcements (B×L: 152 mm×152mm)

Figure 4.2.18 Pressure-settlement curves for model footing tests with five layers of different types of reinforcements (B×L: 152 mm×254mm)
Figure 4.2.19 BCR versus type of reinforcement for silty clay
(B×L: 152 mm×152mm)
Figure 4.2.20 BCR versus settlement ratio (s/B)
(B×L: 152 mm×152mm)
Figure 4.2.20 (continued)
Figure 4.2.21 SRF versus applied footing pressure (q)
(B×L: 152 mm×152 mm)
Figure 4.2.21 (continued)
4.2.23 for 152 mm × 254 mm rectangular footing with different number of layers of BX6100 geogrid. Figures 4.2.24 and 4.2.25 present the induced vertical stress distributions at depths of 152 mm (1B) and 254 mm (1.67B), respectively for 152 mm-wide footing with three layers of BX6200 geogrid placed at different vertical spacing. The induced vertical stress distributions at a depth of 254 mm (1.67B) for 152 mm-wide footing and 152 mm × 254 mm rectangular footing with five layers of different types of reinforcement are presented in Figure 4.2.26 and 4.2.27. The profiles of induced vertical stress with depth below the center of the footing are shown in Figure 4.2.28 for 152 mm-wide footing with three layers of BX6200 geogrid placed at different vertical spacings. Here, the vertical stress distributions are only presented for two footing pressure levels, the vertical stress distributions at the other applied footing pressures are presented in Figures A.1 through A.7 on Appendix A. It is noted that the stresses measured here by the pressure cells are the total vertical stresses induced by the applied footing load, while the stresses induced by the weight of soil are not included.

As can be seen from these Figures, the induced maximum stresses beneath the center of the footing in reinforced silty clay are appreciably reduced compared to those in unreinforced silty clay. For three layers of BX6200 geogrid at different vertical spacing, the stress at a depth of 152 mm (1.0B) can be reduced up to 49% and up to 19% at a surface pressure of 47 kPa and 468 kPa, respectively. For 152 mm-wide square footing with five layers of different types of reinforcement, the reduction in stress at a depth of 254 mm (1.67B) ranges from 18% to 69% and from 18% to 36% at a surface pressure of 47 kPa and 468 kPa, respectively; while the reduction in stress at the same depth for 152 mm × 254 mm rectangular footing varies from 5% to 35% and from 15% to 26% at a surface pressure of 28 kPa and 422 kPa, respectively. Figures 4.2.13 and 4.2.27 also show that the load was redistributed and an improved stress distribution was achieved for 152 mm × 254 mm rectangular footing due to the inclusion of reinforcement. For 152 mm-wide square footing, due to the pressure cell next to the center of footing having a distance of 1.33B from the center of footing, this redistribution of load was not shown from the measurement data, but it is believed to be existed. This point was confirmed in the field tests (see later discussion in section 4.5). The reduction in stress distribution at center will be resulted in reducing the consolidation settlement of silty clay which is directly related to the induced stress. Generally, for the same applied footing pressure, the vertical stresses under the center decrease with increasing number of layers (Figures 4.2.22 and 4.2.23) and decreasing vertical spacing of reinforcement layers (Figures 4.2.24, 4.2.25, and 4.2.26).
For geogrids with the same material and aperture size, the higher modulus geogrid (BX6200) results in more significant reduction of center stresses than the lower modulus geogrid (BX6100) does. BasXgrid11, which has the highest modulus and smallest aperture size among the three geogrids, provided the best attenuation of the stresses under the center of footing; while HP570 geotextile, which has higher modulus than all three geogrids, showed better attenuation of the center stresses than geogrids. It seems that the improvement of stress distribution in reinforced silty clay is somehow related to the modulus of reinforcement. It is also noted that the improved performance of reinforced soil is not always compatible with the improved stress distribution. As shown earlier, at relatively low footing pressure, geogrid reinforced silty clay performs better than geotextile reinforced silty clay, but the induced stresses under the center of the footing in geogrid reinforced silty clay are higher than those in geotextile reinforced silty clay, even at relative low footing pressure. This observation is in agreement with work by Leng (2002). They attributed this performance to better tension membrane effect in geotextile than geogrids.

Interestingly, negative stresses (stresses less than self weight of the soil) are measured in unreinforced silty clay approximately at 3.5B for 152 mm-wide square footing and at 2.0L for 152 mm × 254mm rectangular footing as measured from the center of footing. This result indicates that the soil is pushed upward at a distance of around 3.5B (2.0L) from the center of footing. By contrast, the similar behavior is only observed in reinforced silty clay with three layers of reinforcement placed at a spacing of 76mm and 103 mm, but the values of measured negative stresses are much smaller than those in unreinforced silty clay. Apparently, the stresses at the same locations for reinforced silty clay with appropriate reinforcement configuration are positive, which means an increase in vertical earth pressure. This increase in vertical earth pressure caused due to the inclusion of reinforcement can prevent soil from moving upward at locations far away from the footing, and thus improve the bearing capacity of silty clay. This phenomenon is known as “surcharge effect”, since this effect is equivalent to adding a surcharge load.

Figures 4.2.29 through 4.2.34 depict the variation of the stress influence factor (I) at depths of 152 mm and 254 mm, respectively below 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. As shown in the Figures, under the same footing pressure, the (I) factor decreases with the increase of the number of reinforcement layers. For example, under the footing pressure of 937 kPa, the stress influence factor in BX6200
geogrid reinforced silty clay with 152 mm-wide square footing is reduced from 0.36 to 0.32 as the number of reinforcement layers increases from three to five. However, the decrease is not significant for BX6200 geogrid reinforced silty clay between four and five layers of reinforcement. In general, the stress influence factor (I), as shown in Figures 4.2.31 and 4.2.32, increases with the increase in the vertical spacing of reinforcement layers. Figures 4.2.29 through 4.2.34 show that the stress influence factors (I) increase with the increase of the footing pressures, so the stress influence factor (I) seems to be a load dependent value instead of a constant value as indicated by the elastic solutions such as the Boussinesq solution.

4.2.7 Strain Distribution along the Reinforcement
Two laboratory model tests were conducted to evaluate the strain distribution along the reinforcements. One model test was for square footing with dimensions of 152 mm × 152 mm (B×L), while the other model test was for rectangular footing with dimensions of 152 mm × 254 mm (B×L). Four layers of BX6100 geogrid placed at a spacing of 51 mm were used in both tests. The geogrids with instrumentation were placed at the top and bottom layers (at depths of 51 mm and 203 mm, respectively). The distributions of strains along the centerline of the BX6100 geogrid measured at different settlement ratios(s/B) are presented in Figures 4.2.35, 4.2.36, and 4.2.37. The measured tensile strain is maximum at the point beneath the center of the footing and becomes almost negligible at about 2.5~3.0B from the center of footing. This indicates that the geogrid beyond the effective length (5.0 ~ 6.0B) results in insignificant mobilized tensile strength, and thus provides negligible effects on the improved performance of reinforced silty clayey soil foundation. The corresponding tensions developed in geogrid are shown in Figures A.8 through A.10 on Appendix A.

It is interesting to mention here that compressive strains were measured in the geogrid beyond 1.0~2.0B (L) from the center of footing. This means that the geogrid past this length cannot restrain lateral soil shear flow and works as an anchorage unit to prevent geogrid from failing by pull out. This measured compressive strain is consistent with the limit analysis point of view (Michalowski, 2004).

4.3 Small-Scale Laboratory Tests on Reinforced Sand
The main objective of this research was to investigate the potential benefits of using the RSFs to improve the bearing capacity and reduce the settlement of shallow foundations on sand. For this purpose, extensive laboratory model tests were conducted on geosynthetic reinforced sandy soils.
Figure 4.2.22 Vertical stress distribution along the center line of footing at a depth of 254 mm for multi-layer of BX6200 geogrid reinforced section (B×L: 152 mm×152mm)
Figure 4.2.23 Vertical stress distribution along the center line of footing at a depth of 254 mm for multi-layer of BX6100 geogrid reinforced section (B×L: 152 mm×254 mm)
Figure 4.2.24 Vertical stress distribution along the center line of footing at a depth of 152 mm for three layers of BX6200 geogrid at different vertical spacing (B×L: 152 mm×152mm)
Figure 4.2.25 Vertical stress distribution along the center line of footing at a depth of 254 mm for three layers of BX6200 geogrid at different vertical spacing (B×L: 152 mm×152 mm)
(a). Applied footing pressure q=47 kPa

(b). Applied footing pressure q=468 kPa

Figure 4.2.26 Vertical stress distribution along the center line of footing at a depth of 254 mm for five layers of different types of reinforcement (B×L: 152 mm×152mm)
Figure 4.2.27 Vertical stress distribution along the center line of footing at a depth of 254 mm for five layers of different types of reinforcement (B×L: 152 mm×254 mm)
Figure 4.2.28 Profiles of vertical stress with the depth below the center of footing
(B×L: 152 mm×152mm)
Figure 4.2.29 Stress influence factor ($I$) at a depth of 254 mm (1.67B) below the center of footing versus applied footing pressure for multi-layer of BX6200 geogrid (B×L: 152 mm×152 mm)

Figure 4.2.30 Stress influence factor ($I$) at a depth of 254 mm (1.67B) underneath the center of footing versus applied footing pressure for multi-layer of BX6100 geogrid (B×L: 152 mm×254 mm)
Figure 4.2.31 Stress influence factor ($I$) at a depth of 152 mm (1.0B) below the center of footing versus applied footing pressure for three layers of BX6200 geogrid at different vertical spacing (B×L: 152 mm×152mm)

Figure 4.2.32 Stress influence factor ($I$) at a depth of 254 mm (1.67B) below the center of footing versus applied footing pressure for three layers of BX6200 geogrid at different vertical spacing (B×L: 152 mm×152mm)
Figure 4.2.33 Stress influence factor (I) at a depth of 254 mm (1.67B) below the center of footing versus applied footing pressure for five layers of different types of reinforcement (B×L: 152 mm×152mm)

Figure 4.2.34 Stress influence factor (I) at a depth of 254 mm (1.67B) below the center of footing versus applied footing pressure for five layers of different types of reinforcement (B×L: 152 mm×254mm)
Figure 4.2.35 Strain distribution along the center line of BX6100 geogrid
(B×L: 152 mm×152 mm)
Figure 4.2.36 Strain distribution along the center line of BX6100 geogrid in the width direction of footing (B×L: 152 mm×254 mm)
Figure 4.2.37 Strain distribution along the center line of BX6100 geogrid in the length direction of footing (B×L: 152 mm×254mm)
Due to the fact that in engineering practice 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 include the top layer spacing \((u)\), the number of reinforcement layers \((N)\), the vertical spacing between reinforcement layers \((h)\), the tensile modulus and type of reinforcement, embedment depth \((D_f)\), and shape of footing. The experimental study also includes investigating the stress distribution in sand and the strain distribution along the reinforcement.

Three types of geogrids: BasXgrid11, Miragrid 8XT and BX6100 and one type of geotextile, HP570, were used as reinforcement in the tests. A composite which is a combination of BX6100 geogrid and HP570 geotextile (i.e. HP570 geotextile is placed directly on the top of BX6100 geogrid to form a new reinforcement) was also used in the present study. The physical and mechanical properties of these geosynthetics as provided by the manufacturers were presented earlier in Table 3.6.

The measured dry densities for sand test sections with and without reinforcement inclusion varied from 1,690 to 1,763 kg/m\(^3\), with the moisture contents ranging from 4.5 to 5%. The corresponding geogauge stiffness moduli were in the range of 50 to 60 MPa.

The results of the laboratory model tests conducted using 152 mm × 152 mm model footing for no embedment depth are summarized in Table 4.2; while Table 4.3 and Table 4.4 present the results of laboratory model tests conducted at an embedment depth ratio \((D_f/B)\) of 1.0 for square and rectangular footings with dimensions of 152 mm × 152 mm and 152 mm × 254 mm, respectively. In these tables the bearing capacity ratios (BCRs) obtained at the ultimate capacity, at a settlement ratio \((s/B) = 3\%\), and at the residual are presented. The results of the model footing tests are also graphically shown in Figures 4.3.1 through 4.3.12. Figures 4.3.1 through 4.3.12 present the pressure-settlement curves for 152 mm×152 mm square footing at a footing embedment depth equal to 152 mm \((D_f = 1.0B)\). Figures 4.3.1 and 4.3.2 show the pressure-settlement curves measured for model footing tests with single layer of BasXgrid11 geogrid and Miragrid 8XT geogrid placed at different top layer spacing, respectively. The measured pressure-settlement curves for model footing tests with different number of reinforcing layers are presented in Figures 4.3.3 through 4.3.6. Figure 4.3.7 depicts the pressure-settlement curves obtained for model footing tests using three layers of BasXgrid11 geogrid placed at different vertical spacing. Figures 4.3.8 through 4.3.11 present the pressure-settlement curves for 152 mm×152 mm footing placed on surface (no embedment). Figures 4.3.8 and 4.3.9 show the
pressure-settlement curves measured for model footing tests with single layer of BasXgrid11 geogrid and Miragrid 8XT geogrid placed at different top layer spacings. The measured pressure-settlement curves for model footing tests with different number of reinforcing layers are presented in Figures 4.3.10 and 4.3.11. Figures 4.3.12 depicts the pressure-settlement curves obtained for model footing tests with four layers of different reinforcement for 152 mm×254 mm rectangular footing at a footing embedment depth equal to 152 mm (Df = 1.0B).

It can be seen from these figures that the magnitude of settlement ratio (s/B) at ultimate bearing capacity is about 7~10% for embedded footing and 4~7% for surface footing on both unreinforced and reinforced sands. It is clear that although the inclusion of geogrid/geotextile reinforcement can increase the ultimate bearing capacity of sand, However, the effect of reinforcement on footing settlement at ultimate load is minimal. Yetimoglu et al. (1994) reported the same observation for a rectangular footing on reinforced sand. On the other hand, Omar et al. (1993a) and, Das and Omar (1994) indicated that the magnitude of settlement ratio (s/B) at ultimate bearing capacity increased along with an increase of the ultimate bearing capacity for tests on reinforced sand over unreinforced sand.

4.3.1 Effect of Reinforcement Top Layer Spacing

The optimum location (top layer spacing) of the first reinforcement layer for sand was investigated for both confined (embedded footing) and unconfined (surface footing) conditions.

For footing embedment depth equal to 152 mm, the measured pressure-settlement curves for the model tests with one layer of reinforcement placed at different top layer spacing are presented in Figures 4.3.1 and 4.3.2 for sand reinforced with BasXgrid11 and Miragrid 8XT geogrids as well as without reinforcement. For BasXgrid 11, Figure 4.3.1 shows that the ultimate bearing capacity of the reinforced sand increases from 3878 kPa to 4596 kPa as the top layer spacing (u) increases from 0 mm to 51 mm, then it decreases to 4022 kPa as u increases from 76 mm to 203 mm. However, for Miragrid 8XT in which two pieces were used (one in each direction), the ultimate bearing capacity increases from 3878 kPa to 4501 kPa as the top layer spacing increases from 0 mm to 51 mm, then it decreases to 4118 kPa as u increases from 51 mm to 102 mm. The ultimate bearing capacity reached its maximum value when u equals to 51 mm. Figures 4.3.13a and 4.3.13b show that the BCR at 3% settlement ratio and the ultimate loads generally increases with increasing the top layer spacing ratio (u/B) up to a maximum value at u/B = 0.33 for both BasXgrid 11 and Miragrid 8XT geogrid, after which it decreases. The
Table 4.2 Summary of laboratory model tests for sand with 152 mm×152 mm square footing on surface

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Reinforcement configuration</th>
<th>u mm</th>
<th>h mm</th>
<th>Ultimate @ s/B = 3%</th>
<th>Residual @ s/B = 12%</th>
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<td>688</td>
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<td>1077 1.15 758 1.10 333 0.94</td>
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* Instrumented with pressure cell

Table 4.3 Summary of laboratory model tests for sand with 152 mm×152 mm square footing at an embedment depth of 152mm

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<tr>
<th>Test No.</th>
<th>Reinforcement configuration</th>
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<th>h mm</th>
<th>Ultimate @ s/B = 3%</th>
<th>Residual @ s/B = 12%</th>
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Table 4.3 (continued)

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<th>u mm</th>
<th>h mm</th>
<th>Ultimate</th>
<th>@ s/B = 3%</th>
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* Instrumented with pressure cell  
# Instrumented with strain gauge

Table 4.4 Summary of laboratory model tests for sand with 152 mm×254 mm rectangular footing at an embedment depth of 152mm

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Reinforcement configuration</th>
<th>u mm</th>
<th>h mm</th>
<th>Ultimate</th>
<th>@ s/B = 3%</th>
<th>Residual @ s/B = 12%</th>
</tr>
</thead>
<tbody>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>SDFNR*</td>
<td>Unreinforced</td>
<td></td>
<td></td>
<td>3562</td>
<td>2253</td>
<td>2360</td>
</tr>
<tr>
<td>SDFGG14*</td>
<td>N=4,BasXgrid11</td>
<td>51</td>
<td>51</td>
<td>4711 1.32</td>
<td>2673 1.19</td>
<td>3953 1.67</td>
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<tr>
<td>SDFGG24*</td>
<td>N=4, BX6100</td>
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<td>51</td>
<td>4596 1.29</td>
<td>2750 1.22</td>
<td>3152 1.34</td>
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<tr>
<td>SDFGT14*</td>
<td>N=4, HP570</td>
<td>51</td>
<td>51</td>
<td>4711 1.32</td>
<td>2384 1.06</td>
<td>4577 1.94</td>
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<tr>
<td>SDFGGT14*</td>
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<td>51</td>
<td>5401 1.52</td>
<td>2855 1.27</td>
<td>5264 2.23</td>
</tr>
</tbody>
</table>

* Instrumented with pressure cell
Figure 4.3.1 Pressure-settlement curves for model footing tests with single layer BasXgrid11 placed at different top layer spacing (B×L: 152 mm×152mm; D/B: 1.0)

Figure 4.3.2 Pressure-settlement curves for model footing tests with single layer Miragrid 8XT placed at different top layer spacing (B×L: 152 mm×152mm; D/B: 1.0)
Figure 4.3.3 Pressure-settlement curves for model footing tests with different number of layers of BasXgrid11 geogrid (B×L: 152 mm×152 mm; Df/B: 1.0)

Figure 4.3.4 Pressure-settlement curves for model footing tests with different number of layers of BX6100 geogrid (B×L: 152 mm×254 mm; Df/B: 1.0)
Figure 4.3.5 Pressure-settlement curves for model footing tests with different number of layers of HP570 geotextile (B×L: 152 mm×152 mm; Df/B: 1.0)

Figure 4.3.6 Pressure-settlement curves for model footing tests with different number of layers of Composite (B×L: 152 mm×152 mm; Df/B: 1.0)
Figure 4.3.7 Pressure-settlement curves for model footing tests with three layers of BasXgrid11 placed at different vertical spacing (B×L: 152 mm×152mm; Df/B: 1.0)

Figure 4.3.8 Pressure-settlement curves for model footing tests with single layer BasXgrid11 placed at different top layer spacing (B×L: 152 mm×152mm; Df/B: 0.0)
Figure 4.3.9 Pressure-settlement curves for model footing tests with single layer Miragrid 8XT placed at different top layer spacing (B×L: 152 mm×152mm; Df/B: 0.0)

Figure 4.3.10 Pressure-settlement curves for model footing tests with different number of layers of BasXgrid11 geogrid (B×L: 152 mm×152mm; Df/B: 0.0)
Figure 4.3.11 Pressure-settlement curves for model footing tests with different number of layers of HP570 geotextile (B×L: 152 mm×152mm; Df/B: 0.0)

Figure 4.3.12 Pressure-settlement curves for model footing tests with four layers of different types of reinforcements (B×L: 152 mm×254mm; Df/B: 1.0)
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 stiffness of geogrid.

For the surface footing condition, the pressure-settlement curves of the model tests for one layer of reinforcement placed at different top layer spacing are shown in Figures 4.3.8 and 4.3.9 for BasXgrid 11 and Miragrid 8XT geogrids compared to the case of no reinforcement. For BasXgrid 11, Figure 4.3.8 shows that the ultimate bearing capacity decreases from 1382 kPa to 949 kPa as the top layer spacing \( (u) \) increasing from 25 mm to 152 mm. This behavior is different from that for the footing with 152 mm embedment, in which the ultimate bearing capacity first increases to an optimum value and then decreases. The same phenomenon was also obtained for model tests with Miragrid 8XT geogrid reinforcement as can be seen in Figure 4.3.9. Figure 4.3.9 shows that the ultimate bearing capacity decreases from 1265 kPa to 1124 kPa as the top layer spacing \( (u) \) increasing from 31 mm to 76 mm. The variations of BCRs obtained at 3% settlement ratio and the ultimate loads for different top layer spacing \( (u) \) are presented in Figures 4.3.14a and 4.3.14b. The figures show that the BCR values for BasXgrid 11 geogrid and Miragrid 8XT geogrid reinforced sand generally decrease with increasing top layer spacing. No clear optimum top layer spacing was obtained for geogrid reinforced sand for surface footing condition. However, the use of \( u/B \) value of 0.33 seems to be reasonable based on the best judgment from the figures.

Similar to the finding of the present study on surface footing condition, Guido et al. (1986) reported BCRs decreased with the increase of \( u/B \) for 305 mm-wide square footing on both geogrid and geotextile reinforced sand. Omar et al. (1993b) indicated that as \( u/B \) increased, BCRs would decrease accordingly for 76.2 mm-wide square footing on sand with four layers of geogrid. On the other hand, a study conducted by Yetimoglu et al. (1994) using rectangular footings on geogrid reinforced sand indicated that the variation of BCRs with \( u/B \) was different for single and multi-layer geogrid reinforced sand. While the results for multilayer reinforced sand in their studies were similar to the present study, the results for single-layer reinforced sand showed that there was an optimum top layer spacing at which maximum bearing capacity was obtained. According to their studies, this optimum vertical spacing was about 0.3B. In the meanwhile, literature review showed that no such information is available for embedded footings.

The results of laboratory model tests on sand with one layer of reinforcement show that there are two different types of failure modes for reinforced soil, as described below:
a. When the top layer spacing is greater than 0.5B, the failure surface in the sands extends to the surface and the load decreases rapidly after failure. The settlement pattern for this kind of failure resembles typical general shear failure. For this kind of failure mode, it is believed that lateral soil shear flow happens above the reinforcement. The visual inspection of the reinforcement after tests also confirms this point. The deformation of the reinforcement beneath the footing is not clear. This failure mode was first reported by Binquet and Lee (1975b). In this model, the top layer of reinforcement acts as a rigid boundary.

b. When the top layer spacing is less than 0.5B, the failure surface in the sands never extends to the surface of the sands, indicating a punching type of failure. Here the load decreases slowly after failure. For this kind of failure mode, it is believed that lateral soil shear flow crosses the reinforcement. The visual inspection of the reinforcement after test shows the deformation of the reinforcement beneath the footing is significant. Similar to this finding, Yetimoglu et al. (1994) reported that a typical punching shear failure would occur for $u/B < 0.25B$

4.3.2 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 for both surface footing and embedded footing conditions. Figures 4.3.3 through 4.3.6 and Figures 4.3.10 through 4.3.11 present the pressure-settlement curves of these model tests. As shown for model tests on reinforced silty clay soil, the similar phenomenon was also observed in reinforced sand soil. The bearing capacity increased with increasing number of reinforcement layers. For example, the ultimate bearing capacities of sand reinforced by BX6100 geogrid for 152 mm-wide square footing with an embedment of 152 mm increase from 4884 kPa to 5362 kPa as the number of reinforcement layers increases from 1 to 4. However, the significance of an additional reinforcement layer decreases with the increase in number of layers. This effect becomes negligible below the influence depth. The variations of BCRs obtained at settlement ratio of $s/B = 3\%$, the ultimate loads, and the residual loads for different numbers of reinforcement layers ($N$) and reinforcement depth ratios ($d/B$) are shown in Figures 4.3.15a through 4.3.15d for embedded footing and Figures 4.3.16a through 4.3.16b for surface footing. 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 footing for all types of reinforcement. Accordingly, the influence depth can be estimated to be 1.25B. This result

![Graph (a) BasXgrid11](image1)

![Graph (b) Miragrid 8XT](image2)

Figure 4.3.13 BCR versus \( u/B \) for one layer of reinforcement
(B×L: 152 mm×152 mm; \( D_f/B = 1.0 \))
Figure 4.3.14 BCR versus $u/B$ for one layer of reinforcement

(B×L: 152 mm×152 mm; $D_f/B$: 0.0)
suggests that the type and stiffness 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), based on their study of a 305 mm-wide square footing on reinforced sand, 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 for a 76.2 mm-wide square footing on geogrid reinforced sand was approximately 1.2B. Shin et al. (2002), using strip footings on geogrid reinforced sand with embedment depth ratios $D_f/B=0.0, 0.3,$ and $0.6$, indicated that the influence depth extends to $2.0B$ and was unrelated to the embedment depth.

4.3.3 Effect of Vertical Spacing of Reinforcement Layers

The effect of vertical spacing of reinforcement layers in sand was investigated using three layers of BasXgrid11 with a top layer spacing of $51$ mm ($0.33B$) and vertical spacing varied from $0.167B$ to $0.5B$. Figure 4.3.7 shows that the ultimate bearing capacity of the reinforced sand decreases from $5554$ kPa to $5133$ kPa as the vertical spacing ($h$) increases from $25$ mm to $76$ mm. Figure 4.3.17 depicts the variation in the BCR values of the loads corresponding to settlement ratios $s/B=3\%$, the ultimate loads, and the residual loads as a function of the vertical spacing ratio ($h/B$). It is obvious that the BCR values decrease with increasing vertical spacing of reinforcement layers with maximum BCR at $h = 0.167B$. No optimum vertical spacing was obtained for the BasXgrid11 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) using rectangular footings on geogrid reinforced sand showed that there was an optimum vertical spacing of reinforcement layers at which maximum bearing capacity was obtained. According to their studies, the optimum vertical spacing was about $0.2B$ for reinforced sand with four layers of reinforcement at top layer spacing of $0.3B$. As indicated in the previous discussion of model tests on reinforced silty clay soil, in order to fully understand the effect of vertical spacing on bearing capacity separately, other influencing factors, such as the top layer spacing ($a$), number of layers ($N$), geogrid modulus, should be considered. Once again, for the sand and geogrid reinforcement tested in this study, one can realize that the smaller the spacing, the higher the BCR. For design purpose, engineers need balance between reducing
spacing and increasing geogrid modulus. The author believes a value of \( h/B = 0.33 \) can be a reasonable value for use in the design of reinforced sand.

Figure 4.3.15 BCR versus \( N \) and \( d/B \)
(B\times L: 152 mm\times 152 mm; \( D/B = 1.0 \))
(c). HP570 geotextile

(d). HP570/BX6100 Composite

Figure 4.3.15 (continued)
Figure 4.3.16 BCR versus $N$ and $d/B$
(B×L: 152 mm×152mm; $D/B = 0.0$)
4.3.4 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 (footing on surface), and one at an embedment depth equal to the footing width ($D_f = B = 152$ mm). The tests conducted in the present study indicated that at the same settlement ratio ($s/B$), the BCRs for surface footing are generally greater than those for 152 mm embedded footing (Figure 4.3.18). The BCRs also increases with increasing the settlement ratio. It is also shown in the present study that the BCRs at ultimate bearing capacity for surface footing are generally smaller than those for footing with 152 mm embedment (Figure 4.3.19). This finding may be expected in the light of the fact that the settlement ratios ($s/B$) at the ultimate bearing capacity for 152 mm embedded footing 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 ultimate bearing capacity for strip footing increased with increasing $D_f / B$. However, Shin et al. (2002) also indicated that for $s/B \leq 5\%$ the BCRs for surface footing are less than those for embedded footing, while the BCRs for footing at an embedment depth ratio of 0.3 are greater than those for footing at an embedment depth ratio of 0.6.

The effect of footing shape on the BCR of reinforced sand was also investigated by conducting two sets of model tests, one with a 152 mm×152 mm square footing, and one with a
152 mm×254 mm rectangular footing. The test results show that the ultimate bearing capacity of unreinforced sand for 152 mm-wide square footing is greater than that for 152 mm×254 mm rectangular footing (Table 4.3 and 4.4), which is consistent with the theoretical analysis by using bearing capacity formula suggested by Vesic (1973). Figure 4.3.20 indicates that, at the same settlement, the bearing capacity of unreinforced sand for 152 mm-wide square footing is also greater than that for 152 mm×254 mm rectangular footing. Similarly, in reinforced sand, higher bearing capacity (both at ultimate and the same settlement) was also observed for 152 mm-wide square footing (Figures 4.3.21 through 4.3.24). The comparison of BCRs obtained for these two different shape footings is shown in Figures 4.3.25 and 4.3.26. Figure 4.3.25 clearly shows that the BCRs at ultimate bearing capacity for 152 mm-wide square footing are greater than those obtained for 152 mm×254 mm rectangular footing. The similar trend was identified for the BCRs at settlement ratio less than 12% (Figure 4.3.26). On the other hand, Omar et al. (1993a) reported that the BCRs at ultimate bearing capacity decreased with increasing the $B/L$. It should be pointed out that in their study the model tests were conducted on surface footing condition at which the ultimate bearing capacity of unreinforced sand decreases with increasing $B/L$ according to theoretical analysis.

### 4.3.5 Effect of Tensile Modulus and Type of Reinforcement

Four different types of reinforcement with different tensile modulus were used in the model footing tests on sand. These include BasXgrid11 geogrid, BX6100 geogrid, HP570 geotextile and HP570/BX6100 composite. The properties of these reinforcements were presented earlier in Table 3.6. Figures 4.3.12 and 4.3.27 through 4.3.30 compare the pressure-settlement curves obtained for different types of reinforcements on model tests conducted with multiple reinforcement layers placed at top layer spacing and vertical spacing of 51 mm. As seen in these figures, the performance of BasXgrid11 geogrid and BX6100 geogrid is very similar until ultimate bearing capacity reached, after which the sand reinforced by BasXgrid11 geogrid, which has a higher tensile modulus and smaller aperture size than BX6100 geogrid, performs appreciably better than that reinforced by BX6100 geogrid. This point is more clearly demonstrated in Figures 4.3.31 and 4.3.32. Similar to this observation, the test results presented by Huang and Tatsuoka (1990) for strip footing on reinforced sand indicated that the behavior of reinforcement with different modulus was very similar until the footing settlement reached a certain value. On the other hand, a study conducted by Lee and Manjunath (2000) on reinforced
sand indicated that the geogrid with the highest modulus and smallest aperture size had the best performance.

![Graph showing BCR versus settlement ratio (s/B)](image)

- **(a). Two layers of reinforcement**

- **(b). Three layers of reinforcement**

Figure 4.3.18 BCR versus settlement ratio (s/B)  
(B×L: 152 mm×152mm)
(e). Four layers of reinforcement

Figure 4.3.18 (continued)

Figure 4.3.19 BCR versus type of reinforcement for both embedded footing and surface footing at the ultimate bearing capacity (B×L: 152 mm×152mm)
**Figure 4.3.20** Pressure-settlement curves for model footing tests on unreinforced sand with different shape footing

**Figure 4.3.21** Pressure-settlement curves for model footing tests with four layers of BasXgrid11 geogrid and different shape footing
Figure 4.3.22 Pressure-settlement curves for model footing tests with four layers of BX6100 geogrid and different shape footing

Figure 4.3.23 Pressure-settlement curves for model footing tests with four layers of HP570 geotextile and different shape footing
Figure 4.3.24 Pressure-settlement curves for model footing tests with four layers of Composite and different shape footing

Figure 4.3.25 BCR versus type of reinforcement for two different size footing at the ultimate bearing capacity ($D_f/B = 1.0$)
The variations of BCRs with settlement ratios ($s/B$) for model tests with multiple layers of different types of reinforcement are presented in Figures 4.3.32a through 4.3.32d. It can be seen that the BCR generally increases with the increase of settlement ratio ($s/B$). Before the ultimate bearing capacity is reached, the BCRs of geotextile reinforced sand are 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 is higher compared to that for geogrid reinforced sand. Consequently, at post failure stage, the BCRs of geotextile reinforced sand are much greater than those of geogrid reinforced sand. This point can also be clearly seen in Figure 4.3.31. Furthermore, as shown in Figures C.5 and C.11, the bearing capacity of geotextile reinforced sand at low settlement level ($s/B < 2\%$ for embedded footing and $s/B < 1.5\%$ for surface footing) is even less than that of unreinforced sand. Based on model tests of square footings on sand, Guido et al. (1985) reported that for $s/B < 1.7\%$ the response of unreinforced sand was stiffer than that of geotextile reinforced sand. As discussed in section 4.2.5 for model tests on silty clay, this behavior is due to the slack of woven geotextile. It is also interested to note that the ultimate bearing capacity of geotextile reinforced sand is somewhat higher than that of geogrid reinforced sand for embedded footing, while it is obviously lower for surface footing (Table 3.2 and 3.3). Figures 4.3.10 and 4.3.11 show that the sand reinforced by HP570/BX6100
composite performed better than that reinforced by geogrid or geotextile alone. This better performance of HP570/BX6100 composite becomes pronounced at post failure stage. Again, because of a serviceability requirement, geogrid reinforcement is generally considered to perform better for sand 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.

The settlement reduction factors (SRF) at different footing pressure (q) for the model tests with multiple layers of different types of reinforcement are presented in Figures 4.3.12a through 4.3.12d. It is obvious that the inclusion of the reinforcement would reduce the settlement except for geotextile at low to medium footing pressure (q < 2000 kPa). 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 this study of 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. HP570/BX6100 composite provides the best effect on reducing the footing settlement.

**4.3.6 Stress Distribution in Sand**

Several laboratory model tests were conducted to evaluate the stress distribution in sand with and without reinforcement inclusion. The measured stress distributions along the center line of the footing at the depth of 254 mm (1.67B) below the footing for both embedded and surface footing with different number of layers of geogrid (BX6100 for embedded footing and BasXgrid11 for surface footing) are shown in Figures 4.3.34 and 4.3.35, respectively. Figures 4.3.36 and 4.3.37 present the stress distributions at the depth of 254 mm (1.67B) below the footing for 152 mm×152 mm and 152 mm×254 mm footing at an embedment depth of 152 mm with four layers of different types of reinforcement, respectively. Here, only limited cases of stress distributions are presented; the complete cases of stress distributions are presented in Figure B.1 through B.6 on Appendix B. Figures 4.3.38 through 4.3.41 depict the variation of the stress influence factor (I) under the center of the footing with applied footing pressures.

As can be seen from these Figures, the reinforcement results in redistribution of the applied load to a wider area, thus avoiding stress concentration and achieving improved stress distribution. The induced maximum stresses beneath the center of the footing in reinforced sand are appreciably reduced compared to those in unreinforced sand, especially for surface footing condition. For embedded 152 mm wide square footing with four layers of different types of
Figure 4.3.27 Pressure-settlement curves for model footing tests with one layer of different types of reinforcements (B×L: 152 mm×152mm; Df/B = 1.0)

Figure 4.3.28 Pressure-settlement curves for model footing tests with two layers of different types of reinforcements (B×L: 152 mm×152mm; Df/B = 1.0)
Figure 4.3.29 Pressure-settlement curves for model footing tests with three layers of different types of reinforcements (B×L: 152 mm×152mm; \(D_f/B = 1.0\))

Figure 4.3.30 Pressure-settlement curves for model footing tests with four layers of different types of reinforcements (B×L: 152 mm×152mm; \(D_f/B = 1.0\))
Figure 4.3.31 BCR versus type of reinforcement for sand
(B×L: 152 mm×152mm; D_f/B = 1.0)
reinforcement, the reduction in maximum stress ranges from 14% to 28% and from 12% to 19% at a footing pressure of 766 kPa and 2681 kPa, respectively; while for surface footing with four layers of different types 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. The redistribution of load to a wider area below the reinforced zone usually results in reducing the consolidation settlement of underlying weak clayey soil, which is directly related to the induced stress.

Generally, for the same applied footing pressure, the vertical stresses under the center decrease with increasing number of layers (Figures 4.3.34 and 4.3.35). As shown in Figures 4.3.38 through 4.3.39, under the same footing pressure, the stress influence factor (I) decreases with the increase of the number of reinforcement layers.

Among the geogrids used, the geogrid (BasXgrid11) with higher modulus resulted in a better reduction of the center stresses than geogrid (BX6100) with lower modulus. HP570 geotextile, which has higher tensile modulus than the geogrids used in this study, showed better attenuation of the stresses under the center of footing than geogrids. However, HP570/BX6100 composite provided the best attenuation of the center stresses among four types of reinforcement used in the present study for sand. It is interesting to mention here that the improved performance of reinforced sand is also not always compatible with the improved stress distribution, similar to the observation on reinforced silty clay soil. As shown earlier, before the
Figure 4.3.32 BCR versus settlement ratio (s/B) (B×L: 152 mm×152mm; D_f/B = 1.0)
Figure 4.3.32 (continued)
Figure 4.3.33 SRF versus applied footing pressure (q)
(B×L: 152 mm×152 mm; Df/B = 1.0)
Figure 4.3.33 (continued)
ultimate bearing capacity reached, geogrid reinforced sand generally performs better than geotextile reinforced sand, but the induced stresses under the center of the footing in geogrid reinforced sand are higher than those in geotextile reinforced sand. As indicated in the previous discussion of model tests on silty clay, it seems that improvement of stress distribution in reinforced sand is closely related to the tensile modulus of reinforcement and that better tension membrane effect provides better improved in stress distribution.

Negative stresses were measured in unreinforced sand for surface footing at approximately 2.5B from the center of footing. This result indicates that the sand is pushed upward at a distance of around 2.5B from the center of footing. It confirms again that the inclusion of reinforcement could develop a “surcharge effect”.

Figures 4.3.17 through 4.3.20 show that the stress influence factor (I) in sand is a load dependent value instead of a constant value and it increases with the increase of the footing pressures. This result is in agreement with work by Gabor et al. (1998). They attributed this behavior to the non-linear stress-strain characteristics of sand and load dependent sand modulus. It is also indicated in Figures 4.3.17 and 4.3.18 that the stress influence factors (I) for embedded footing are smaller than those for surface footing. 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.

4.3.7 Strain Distribution along the Reinforcement

One laboratory model test using 152 mm-wide square footing at and embedment depth of 152 mm was conducted to evaluate the strain distribution along the reinforcement. Four layers of BX6100 geogrid placed at a spacing of 51 mm were used in the test. The geogrids with strain gauges 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 Figure 4.3.42. The tensile strain is the largest at the point beneath the center of the footing and becomes almost negligible at about 3.0B from the center of footing. It also 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. This observation is the same as that observed for reinforced silty clay foundation. The corresponding tension developed in geogrid is shown in Figures B.7 on Appendix B.
Compressive strains were measured in the geogrid located beyond 0.85B and 1.15B from the center of footing for geogrid placed at a depth of 51 mm and 203 mm below the footing, respectively. As stated before in reinforced silty clay, this means that the geogrid past this length cannot restrain lateral soil shear flow and works as an anchorage unit to prevent geogrid from failing by pull out. Similar to this finding, Huang and Tatsuoka (1990) reported that compressive forces occurred at about 1.25B from the center of footing on reinforced sand. A study conducted by James and Raymond (2002) indicated compressive strain occurred at about 0.6B from the center of footing on reinforced aggregate. The compressive strain measured in the reinforcement beyond a certain length may be due to the reason that the direction of reinforcement past this length is coincident with the direction of compressive strains in the soil (Huang and Tatsuoka, 1990). This point was clearly described by Michalowski (2004) through limit analysis.

4.4 Small-Scale Laboratory Tests on Reinforced Kentucky Crushed Limestone

The main objective of this study was to investigate the potential benefits of using the RSFs to improve the bearing capacity and reduce the settlement of shallow foundations on crushed limestone. For this purpose, extensive laboratory model tests were conducted on reinforced crushed limestone. The parameters investigated in the model tests include the number of reinforcement layers \( N \), and the tensile modulus of reinforcement.

Five types of geogrids: BX1100, BX1200, BX1500, BasXgrid 11 and MS330, one type of steel wire mesh, and one type of steel bar mesh were used as reinforcement in the tests. The physical and mechanical properties of these reinforcements were presented earlier in Table 3.6.

The measured dry densities for Kentucky crushed limestone test sections with and without reinforcement inclusion varied from 2,243 to 2,333 kg/m\(^3\), with moisture contents ranging from 5.5 to 6%. The corresponding geogauge stiffness moduli were in the range of 70 to 90 MPa.

The results of the laboratory model tests for all crushed limestone test sections are summarized in Table 4.5. In this table the BCRs obtained at settlement ratios, \( (s/B) = 3\%, 5\% \) and 10\%, are presented. The results of the model footing tests are also graphically presented in Figures 4.4.1 through 4.4.3. Figure 4.4.1 depicts the pressure-settlement curves measured for model footing tests on limestone reinforced with a single layer of different types of reinforcements. The measured pressure-settlement curves for model footing tests with two layers of different types of reinforcements are presented in Figure 4.4.2. Figure 4.4.3 depicts the pressure-settlement curves obtained from model footing tests using three layers of different types
Figure 4.3.34 Vertical stress distribution along the center line of footing at a depth of 254 mm below the footing with multi-layer of BX6100 geogrid (B×L: 152 mm×152mm, \(D_f/B = 1.0\))
### Table 4.3.35 Vertical stress distribution along the center line of footing at a depth of 254 mm below the footing with multi-layer of BasXgrid11 geogrid (B×L: 152 mm×152 mm, \(D_f/B = 0.0\))

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#### Figure 4.3.35 (a)
Applied footing pressure \(q=94 \text{ kPa}\)

#### Figure 4.3.35 (b)
Applied footing pressure \(q=750 \text{ kPa}\)

Figure 4.3.35 Vertical stress distribution along the center line of footing at a depth of 254 mm below the footing with multi-layer of BasXgrid11 geogrid (B×L: 152 mm×152 mm, \(D_f/B = 0.0\))
Figure 4.3.36 Vertical stress distribution along the center line of footing at a depth of 254 mm below the footing with four layers of different types of reinforcement (B×L: 152 mm×152mm, Df/B = 1.0)

(a). Applied footing pressure q=766 kPa

(b). Applied footing pressure q=2681 kPa

<table>
<thead>
<tr>
<th>Type</th>
<th>u (mm)</th>
<th>h (mm)</th>
<th>N</th>
</tr>
</thead>
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<tr>
<td>Composite</td>
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Figure 4.3.37 Vertical stress distribution along the center line of footing at a depth of 254 mm below the footing with four layers of different types of reinforcement (B×L: 152 mm×254mm, $D_f/B = 1.0$)
Figure 4.3.38 Stress influence factor (I) at a depth of 254 mm (1.67B) underneath the center of footing versus applied footing pressure for multi-layer of BX6100 geogrid (B×L: 152 mm×152mm, Df/B = 1.0)

Figure 4.3.39 Stress influence factor (I) at a depth of 254 mm (1.67B) underneath the center of footing versus applied footing pressure for multi-layer of BasXgrid11 geogrid (B×L: 152 mm×152mm, Df/B = 0.0)
Figure 4.3.40 Stress influence factor (I) at a depth of 254 mm (1.67B) underneath the center of footing versus applied footing pressure for four layers of different types of reinforcement (B×L: 152 mm×152 mm, Df/B = 1.0)

Figure 4.3.41 Stress influence factor (I) at a depth of 254 mm (1.67B) underneath the center of footing versus applied footing pressure for four layers of different types of reinforcement (B×L: 152 mm×254 mm, Df/B = 1.0)
Figure 4.3.42 Strain distribution along the center line of geogrid
(B×L: 152 mm×152mm; Df/B = 1.0)
of reinforcement. Because of the loading capacity limitation of hydraulic jack used in this study, some tests were not possible to be loaded to fail. The bearing capacity is then obtained at different settlement ratios and used to calculate the BCRs.

Table 4.5 Summary of laboratory model tests for Kentucky crushed limestone

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Reinforcement configuration</th>
<th>u mm</th>
<th>h mm</th>
<th>s/B = 3%</th>
<th>s/B = 5%</th>
<th>s/B = 10%</th>
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</thead>
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<tr>
<td></td>
<td></td>
<td></td>
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<td>q, kPa</td>
<td>BCR</td>
<td>q, kPa</td>
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<td>7455</td>
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4.4.1 Effect of Number of Reinforcement Layers

Several laboratory model footing tests were conducted on the crushed limestone reinforced with multiple layers of reinforcement. Seven different types of reinforcement were used: geogrids: BX1100, BX1200, BX1500, BasXgrid11, and MS330, steel: steel wire mesh, SWM, and steel bar mesh, SBM. The reinforcement layers were placed at a spacing of 51 mm (h/B=0.33). Figures 4.4.1 through 4.4.3 show that the performance of crushed limestone foundation was
Figure 4.4.1 Pressure-settlement curves for model footing tests with single layer of different types of reinforcements in Kentucky crushed limestone

Figure 4.4.2 Pressure-settlement curves for model footing tests with two layers of different types of reinforcements in Kentucky crushed limestone
improved noticeably for all types of reinforcement even with one layer of reinforcement (Figure 4.4.1). At settlement ratios of $s/B=3\%$, 5\%, and 10\%, the bearing capacity of crushed limestone reinforced with one layer of different types of reinforcement (Figure 4.4.1) ranges from 3,048 kPa, 4,174 kPa, and 5,711 kPa to 3,710 kPa, 5,407 kPa, and 8,271 kPa. These correspond to BCRs of 1.03, 1.04, and 1.10 to 1.25, 1.34, and 1.60. The effect of number of reinforcement layers on the BCRs is presented in Figure 4.4.4 at settlement ratios of $s/B=3\%$ and 10\% and in Figure C.1 on Appendix C at the other settlement ratios. As shown in the figures, the BCRs increase with increasing number of reinforcement layers. It can be noticed from Figure 4.4.4 that the effect of number of layers is more appreciable at $s/B = 10\%$ than at $s/B = 3\%$. For example, at $s/B=3\%$, the bearing capacity of BX1500 geogrid reinforced crushed limestone increases from 3,286 kPa to 3,659 kPa, which correspond to an increase in BCR from 1.11 to 1.23, as number of reinforcement layers increases from 1 to 3; while the corresponding bearing capacity (BCR) at $s/B=10\%$ increases from 6,929 kPa (1.34) to 9,289 kPa (1.79). It is obvious that the reinforced benefit is directly related to the footing settlement, which can be explained by achieving better mobilizing of the reinforcements. Studies conducted by other researchers have also shown that
increasing the number of reinforcement layers would increase the BCR of reinforced soils (e.g., Binquet and Lee, 1975a; Huang and Tatsuoka, 1990; Omar et al., 1993; Yetimoglu et al., 1994; Adams and Collin, 1997). Similar to this observation, the test results presented by Adams and Collin (1997) for large square footing on reinforced sand indicated that the benefit increases with increasing settlement ratio. On the other hand, a study conducted by Binquet and Lee (1975) for strip footing on reinforced sand indicated that BCR is constant and independent of the settlement ratio (s/B).

The effect of number of reinforcement layers on the settlement reduction factor (SRF) is shown in Figure 4.4.5 at footing pressures of 2000 kPa and 5500 kPa, and in Figure C.2 on Appendix C at the other footing pressures. It is obvious that the inclusion of the reinforcement would reduce the immediate footing settlement. The figure also shows that the SRFs decrease with increasing number of reinforcement layers. With three layers of reinforcement, the immediate footing settlement can even be reduced by about 60% at a footing pressure of 5500 kPa for all types of reinforcement.

4.4.2 Effect of Tensile Modulus and Type of Reinforcement

Seven different types of reinforcements were used to reinforce crushed limestone in the model footing tests. The properties of these reinforcements were presented earlier in Table 3.6. The BX1100, BX1200 and BX1500 geogrids are made of the same material and have similar structure (single layer/extruded). BX1200 geogrid has higher tensile modulus than BX1100 geogrid. As seen in Figures 4.4.1 through 4.4.3, the crushed limestone reinforced by BX1200 geogrid performs better than that reinforced by BX1100 geogrid. BX1500 geogrid, which has the highest tensile modulus among these three geogrids, has the best performance. As compared to BX1200 geogrid, BasXgrid11 geogrid (woven) and MS330 geogrid (multi-layer/extruded) have different structure and smaller aperture sizes, but with almost similar tensile modulus. In the meanwhile, the similar performance of crushed limestone reinforced with BX1200 geogrid, BasXgrid11 geogrid, and MS330 geogrid was observed in the present study. This result suggests that the structure and aperture size of geogrid within the examined range have minimal influence on the performance of the reinforced crushed limestone, which indicates similar degree of geogrid-crushed limestone interlocking. To further study the effect of tensile modulus, two stiff metal grid reinforcements were used in the present study: steel wire mesh (SWM) and steel bar mesh (SBW). SWM has a tensile modulus of about 30 times higher than the geogrids used in the
Figure 4.4.4 BCR versus type of reinforcement

(a). $s/B = 3\%$

(b). $s/B = 10\%$
Figure 4.4.5 SRF versus type of reinforcement

(a). $q = 2000$ kPa

(b). $q = 5500$ kPa
present study, and the tensile modulus of SBM is around 3 times higher than that of SWM. Figures 4.4.1 through 4.4.3 indicates that the crushed limestone reinforced with SWM and SBM performs much better than those reinforced with geogrids. For three layers of reinforcement at settlement ratio of s/B=10%, BCRs of SWM and SBM reinforced crushed limestone are nearly 1.3 and 1.6 times larger than that for BX1500 geogrid reinforced crushed limestone, respectively. As shown in Figures 4.4.4 and C.1, this study clearly demonstrates that the performance of reinforced crushed limestone improves with increasing the tensile modulus of reinforcement. However, the effect of reinforcement tensile modulus at low settlement ratio (e.g. s/B=2%) is not significant when compared to that at a settlement ratio of s/B=10%. For example, at s/B=2%, the BCR of reinforced crushed limestone for three layers of SBM (with the highest tensile modulus) is 29% higher than that for three layers of BX1100 geogrid (with the lowest tensile modulus); while this difference increases to 88% as the settlement ratio increases to s/B=10%. So the effect of reinforcement modulus seems to be a function of footing settlement. Again, this can be explained by achieving better mobilizing of the reinforcement with increasing footing settlement.

Uchimura et al. (2004) reported that the tensile modulus of the reinforcement has negligible effect on reinforced gravel and it is not necessary to use metal reinforcement, but it should be pointed out that in their study the model tests were only loaded to a settlement ratio of 1% or so, which is equivalent to a footing settlement of around 3.6 mm. This amount of settlement is too small to mobilize the effect of reinforcement as indicated earlier. So the result of present study is still in agreement with the work of Uchimura et al. (2004).

The BCRs at different settlement ratios (s/B) for the model tests on crushed limestone section reinforced with multiple layers of different types of reinforcement are presented in Figure 4.4.6. It can be seen that the BCRs increase with the increase of settlement ratio (s/B). At relatively low settlement ratio (s/B), the increase of the bearing capacity of SWM and SBM reinforced sections has marginal difference from geogrid reinforced sections. However, with the increase of settlement ratio (s/B), the BCRs of footings on SWM and SBM reinforced sections increase at faster rate compared to those on geogrid reinforced sections.

Figure 4.4.7 depicts the settlement reduction factors (SRF) as a function of applied footing pressure (q) for the model tests on crushed limestone section reinforced with multiple layers of different types of reinforcement. As shown in Figure 4.4.7, higher modulus geogrids provide better reduction in settlement than lower modulus geogrids, while the settlements of SWM and SBM reinforced sections are much smaller than those of geogrid reinforced sections. In all cases,
the SRFs decrease with increasing the footing pressure, and the rate of decrease of SRFs increases suddenly at footing pressure of about 4500 kPa. This trend may be expected in the light of the fact that the ultimate bearing capacity of unreinforced crushed limestone is close to 4500 kPa.

(a). One layer of reinforcement

(b). Two layers of reinforcement

Figure 4.4.6 BCR versus settlement ratio (s/B)
(c). Three layers of reinforcement

Figure 4.4.6 (continued)

(a). One layer of reinforcement

Figure 4.4.7 SRF versus applied footing pressure ($q$)
Figure 4.4.7 (continued)

(b). Two layers of reinforcement

(c). Three layers of reinforcement
4.5 Large-Scale Field Tests on Reinforced Silty Clay

A review of existing literature revealed that most of experimental studies on geosynthetic reinforced soils were conducted using small-scale laboratory tests. Due to scale effect sometimes, it is not easy to accurately model the full-scale behavior of reinforced soil with small-scale laboratory tests. Six large scale field tests, therefore, were conducted on geosynthetic reinforced silty clay soils to investigate the potential benefits of using the RSFs to improve the bearing capacity and to reduce the settlement of shallow foundations. The parameters investigated in the field tests include the number of reinforcement layers ($N$), the vertical spacing between reinforcement layers ($h$), and the tensile modulus of reinforcement. The experimental study also includes investigating the vertical stress distribution in the silty clay soils and the strain distribution along the reinforcement.

Three types of geogrids, BX6100, BX6200 and BX15000, were used as reinforcement in the field tests. The physical and mechanical properties of these reinforcements were presented earlier in Table 3.6. The measured dry densities for Kentucky crushed limestone test sections with and without reinforcement inclusion varied from 1,760 to 1,808 kg/m$^3$, with moisture contents ranging from 15.81 to 16.84%.

Based on the test results of laboratory model test, the top layer spacing of geogrid for all field tests were kept constant with the value of $u=0.33B$. For all tests, all geogrids were placed within the depth of $d=1.67B$ with the bottom layer kept at a depth of 1.67B; the vertical spacing between reinforcement layers can then be determined for each test as:

$$h = \frac{d - u}{N - 1}$$  \hspace{1cm} (4.1)

The results of the field tests for unreinforced and reinforced silty clay test sections are summarized in Table 4.6. In this table, the BCRs obtained at settlement ratios of $s/B = 3\%$, $5\%$ and $10\%$ are presented. The results of the model footing tests are also graphically drawn in Figures 4.5.1. Investigating the pressure-settlement curves, we can see that the pressure keeps increasing with an increase in the settlement for both unreinforced and reinforced silty clay test sections. This settlement pattern resembles a typical punching-shear failure. Since the failure point is not well defined, the bearing capacity obtained at different settlement ratios is used to calculate the BCRs.
Table 4.6 Summary of field tests for silty clay embankment soil

<table>
<thead>
<tr>
<th>Reinforcement configuration</th>
<th>u (mm)</th>
<th>h (mm)</th>
<th>s/B = 3% q, kPa</th>
<th>s/B = 3% BCR</th>
<th>s/B = 5% q, kPa</th>
<th>s/B = 5% BCR</th>
<th>s/B = 10% q, kPa</th>
<th>s/B = 10% BCR</th>
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<td>...</td>
<td>638</td>
<td>...</td>
<td>896</td>
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</table>

* Instrumented with pressure cell
# Instrumented with strain gauge

Figure 4.5.1 Pressure-settlement curves for large-scale field model footing tests

4.5.1 Effect of Number and Vertical Spacing of Reinforcement Layers

The effect of number and vertical spacing of reinforcement layers was investigated using multi layers of BX6200 with a top layer spacing of 51 mm (0.33B). The following reinforcement layers/spacing combinations were chosen to study this effect in the present study: three layers placed at 305 mm spacing, four layers placed at 203 mm spacing, and five layers placed at 152 mm spacing. Figure 4.5.2 presents the pressure-settlement curves of these model tests as compared with unreinforced section. At settlement ratios of s/B=3%, 5%, and 10%, Figure 4.5.2
shows that the bearing capacities of silty clay reinforced by BX6200 geogrid increase from 558 kPa, 725 kPa, and 1055 kPa to 709 kPa, 943 kPa, and 1321 kPa respectively as number of reinforcement layers increases from 3 to 5 and vertical spacing ratio decreases from 0.67 to 0.33. Figure 4.5.3 depicts the variations of BCRs obtained at settlement ratios of s/B=3%, 5%, and 10% for different numbers of reinforcement layers (N) and reinforcement spacing ratios (h/B). It is obvious that the BCR values increase with increasing number of reinforcement layers and decreasing vertical spacing of reinforcement layers with maximum BCR at N = 5 and h = 0.33B. Therefore, for all geogrids placed within influence depth, smaller reinforcement spacing (i.e. more reinforcement layers) should always be examined as an alternative of using higher geogrid tensile modulus as will be discussed later provided that its cost is justified.

Investigating the load-settlement curves, we can see that the shapes and slopes of curves of reinforced soil foundations are very similar to those of unreinforced soil foundations when the settlement ratio (s/B) is less than 0.01; and that the reinforcement effect starts being mobilized when the s/B ratio is greater than 0.01. This will be further discussed later in subdivision 4.5.2 with other types of geogrids.

![Figure 4.5.2 Pressure-settlement curves for large-scale field model footing tests with different number of layers of BX6200 placed at different vertical spacing](image)

Figure 4.5.2 Pressure-settlement curves for large-scale field model footing tests with different number of layers of BX6200 placed at different vertical spacing
4.5.2 Effect of Tensile Modulus of Reinforcement

Three different types of geogrid with different tensile modulus were used in the large scale field tests. These include BX6100 geogrid, BX6200 geogrid, and BX1500 geogrid. The properties of these reinforcements were presented earlier in Table 3.6. Figure 4.5.4 compares the pressure-settlement curves obtained for the different types of reinforcements on model tests conducted with four reinforcement layers placed at top layer spacing of 152 mm and vertical spacing of 203 mm. The BX6100 and BX6200 geogrids are made of the same material and have similar aperture size, but BX6200 has higher tensile modulus than BX6100 (Table 3.6). Figure 4.5.4 shows that the silty clay reinforced by BX6200 geogrid performs better than that reinforced by BX6100 geogrid. The Figures also show that BX1500 geogrid, which has the highest tensile modulus and smallest aperture size out of the three types of geogrid used in present study, has the best performance. This effect can be more clearly seen in Figure 4.5.5.

The BCRs at different settlement ratios ($s/B$) for model tests with multiple layers of different types of reinforcement are presented in Figure 4.5.6. It can be seen that the BCR generally increases with the increase of settlement ratio ($s/B$). It is also noted that the bearing capacity ratio ($BCR$) increases significantly only after the $s/B$ ratio becomes greater than 1% at which it is believed that reinforcing effect of geogrid starts to be mobilized. However, BCR only
keeps substantially increasing up to a settlement ratio of $s/B \approx 3\%$; and it remains more or less constant thereafter.

Figure 4.5.7 depicts the variation of the settlement reduction factors ($SRF$) as a function of footing pressure ($q$) for the model tests with multiple layers of different types of reinforcement. It is obvious that the inclusion of the reinforcement would reduce the immediate settlement significantly. With five layers of reinforcement, the settlement can be reduced by 40% at relatively medium footing pressure (500kPa). The geogrid with higher modulus provides better reduction in immediate settlement than geogrid with lower modulus. In all cases, the SRF decreases with increasing footing pressure. It is also noted that the SRF decreases suddenly at a footing pressure of about 300 kPa; and it becomes stabilized at a footing pressure of 700 kPa and higher. This behavior may be expected in the light of the fact that the settlement ratio ($s/B$) is close to 1% at a footing pressure of 300 kPa. As indicated earlier, reinforcing effect of geogrid starts to be mobilized at $s/B = 1\%$.

For the case of all geogrids placed within influence depth, it is necessary to determine whether the design priority should be given for the number of reinforcement layers or for the reinforcement tensile modulus. The average modulus of reinforcement ($E_{avg}$) is introduced to
**Figure 4.5.5** BCR versus type of reinforcement for large-scale field model footing tests

- BX6100: Type, 152 mm, 203 mm, N=4, u=152 mm, h=203 mm
- BX6200: Type, 152 mm, 305 mm, N=3, u=152 mm, h=305 mm
- BX6200: Type, 152 mm, 203 mm, N=4, u=152 mm, h=203 mm
- BX6200: Type, 152 mm, 152 mm, N=5, u=152 mm, h=152 mm
- BX1500: Type, 152 mm, 203 mm, N=4, u=152 mm, h=203 mm

**Figure 4.5.6** BCR versus settlement ratio (s/B) for large-scale field model footing tests

- BX6100: Type, 152 mm, 203 mm, N=4
- BX6200: Type, 152 mm, 305 mm, N=3
- BX6200: Type, 152 mm, 203 mm, N=4
- BX6200: Type, 152 mm, 152 mm, N=5
- BX1500: Type, 152 mm, 203 mm, N=4
Figure 4.5.7 SRF versus applied footing pressure (q) for large-scale field model footing tests help quantify the decision process. The average modulus of reinforcement is defined as the sum of tensile modulus of reinforcement for each layer divided by the total depth of reinforcement. Due to the fact that the tensile modulus of geogrid in machine direction and in cross machine direction is different; the mean value of tensile modulus in two directions is then used to calculate the average modulus of reinforcement.

\[
E_{\text{avg}} = \frac{\sum_{i=1}^{N} (J_{MDi} + J_{CDi})/2}{d} = \frac{N(J_{MD} + J_{CD})/2}{u + (N-1)h}
\]  

(4.2)

where \(E_{\text{avg}}\) is the average modulus of reinforcement over the influence depth, (kPa); \(J_{MDi}\) and \(J_{CDi}\) are tensile modulus in machine direction and cross machine direction for the \(i^{th}\) layer of reinforcement, (kN/m).

The average modulus of reinforcement for the five large-scale field tests on reinforced silty clay is calculated and presented in Table 3.7.

It can be seen from Table 4.7 that the average modulus of the four layers of BX6100 placed at 203 mm spacing is smaller than that of the three layers of BX6200 placed at 305 mm; However, the four layers of BX6100 placed at 203 mm spacing has better performance as compared to the three layers of BX6200 placed at 305 mm. The same observation was also
Table 4.7 Average modulus of reinforcement for large-scale field model footing tests

<table>
<thead>
<tr>
<th>Reinforcement configuration</th>
<th>u mm</th>
<th>h mm</th>
<th>( E_{\text{avg}} ) kN/m(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>N=4, BX6100</td>
<td>152</td>
<td>203</td>
<td>1147</td>
</tr>
<tr>
<td>N=3, BX6200</td>
<td>152</td>
<td>305</td>
<td>1272</td>
</tr>
<tr>
<td>N=4, BX6200</td>
<td>152</td>
<td>203</td>
<td>1696</td>
</tr>
<tr>
<td>N=5, BX6200</td>
<td>152</td>
<td>152</td>
<td>2119</td>
</tr>
<tr>
<td>N=4, BX1500</td>
<td>152</td>
<td>203</td>
<td>2428</td>
</tr>
</tbody>
</table>

obtained for the five layers of BX6200 placed at 152 mm and the four layers of BX1500 placed at 203 mm. Therefore, the design priority should be given for the number of reinforcement layers over the reinforcement tensile modulus when all geogrids are placed within influence depth and its cost is justified.

4.5.3 Stress Distribution in Silty Clay

All field tests were instrumented with pressure cells to evaluate the vertical stress distribution in silty clay soil with and without reinforcement inclusion. Pressure cells were placed at specified locations/depth for this purpose. The measured stress distributions along the center line of the footing at a depth of 762 mm (1.67B) are shown in Figure 4.5.8 for different number of layers of BX6200 geogrid. Figure 4.5.9 presents the stress distributions at a depth of 762 mm (1.67B) for four layers of different types of reinforcement. The measured stress distributions with depth below the center of footing are shown in Figure 4.5.10 for different number of layers of BX6200 geogrid placed at different vertical spacing and Figure 4.5.11 for four layers of different types of reinforcements. Here, only the measured stress distributions at the applied footing pressures of 43 kPa and 468 kPa are presented, the measured stress distributions at the other applied footing pressures are presented in Figure D.1 through D.4 on Appendix D. Figures 4.5.12 through 4.5.14 depict the variation of the stress influence factor (I) computed under the center of the footing with applied footing pressures.

As can be seen from these figures, the reinforcement results in redistribution of the applied load to a wider area, thus avoiding stress concentration and achieving improved stress distribution. The induced maximum stresses beneath the center of the footing in reinforced silty clay are appreciably reduced compared to those in unreinforced silty clay. At a surface pressure of 43 kPa and 468 kPa, the stress can be reduced up to 21% and 26% at a depth of 152 mm (1.0B), 29% and 18% at a depth of 610 mm (1.33B), and 41% and 32% at a depth of 762 mm.
(1.67B) for five layers of BX6200 geogrid. The redistribution of load to a wider area below the reinforced zone usually results in reducing the consolidation settlement of underlying weak clayey soil, which is directly related to the induced stress.

Generally, for the same applied footing pressure, the vertical stresses under the center of footing decrease with increasing number of layers and decreasing vertical spacing of reinforcement layers (Figure 4.5.8). As shown in Figure 4.5.12 through 4.5.14, under the same footing pressure, the stress influence factor \((I)\) decreases with the increase of the number of reinforcement layers and decrease of vertical spacing. For example, under the footing pressure of 937 kPa, the stress influence factor in BX6200 geogrid reinforced silty clay is reduced from 0.24 to 0.19 as the number of reinforcement layers increases from 3 to 5 and vertical spacing ratio \((h/B)\) decreases from 0.67 to 0.33.

Among geogrids with the same material and aperture size, the geogrid with higher tensile modulus (BX6200) results in a better reduction of center stresses than the geogrid with lower tensile modulus (BX6100). BX1500 geogrid, which has the highest tensile modulus and smallest aperture size among three types of geogrid, provides the best attenuation of the stresses below the center of footing.

The four layers of BX6100 placed at 203 mm spacing results in a better reduction of center stresses as compared to the three layers of BX6200 placed at 305 mm. The same observation was also obtained for the five layers of BX6200 placed at 152 mm and the four layers of BX1500 placed at 203 mm. Again, the measured stress distribution shows that the benefit of increasing the number of geogrid layers on improved performance of reinforced soil foundation is larger than that of increasing the tensile modulus of geogrid when all geogrids are placed within influence depth.

Negative stresses were measured in unreinforced silty clay at approximately 3.0B from the center of footing. This result indicates that the soil is pushed upward at a distance of around 3.0B from the center of footing. The similar behavior is observed only in reinforced silty clay with three layers of BX6200 geogrid placed at a spacing of 305 mm, but the values of measured negative stresses are smaller than those in unreinforced silty clay. This again confirms that the inclusion of reinforcement can develop a “surcharge effect” to prevent soil from moving upward, and thus improve the bearing capacity of silty clay. Figures 4.5.12 through 4.5.14, once again, show that the stress influence factor \((I)\) is a load dependent value instead of a constant value and it increases with the increase of the footing pressures.
Figure 4.5.8 Vertical stress distribution along the center line of footing at a depth of 762 mm for different number of layers of BX6200 placed at different vertical spacing
Figure 4.5.9 Vertical stress distribution along the center line of footing at a depth of 762 mm for four layers of different types of reinforcement.
Figure 4.5.10 Profiles of vertical stress with the depth below the center of footing for different number of layers of BX6200 placed at different vertical spacing
Figure 4.5.11 Profiles of vertical stress with the depth below the center of footing for four layers of different types of reinforcement

(a). Applied footing pressure \( q = 43 \) kPa

(b). Applied footing pressure \( q = 468 \) kPa

<table>
<thead>
<tr>
<th>Type</th>
<th>( u ) (mm)</th>
<th>( h ) (mm)</th>
<th>( N )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced</td>
<td>...</td>
<td>...</td>
<td>...</td>
</tr>
<tr>
<td>BX6100</td>
<td>152</td>
<td>203</td>
<td>4</td>
</tr>
<tr>
<td>BX6200</td>
<td>152</td>
<td>203</td>
<td>4</td>
</tr>
<tr>
<td>BX1500</td>
<td>152</td>
<td>203</td>
<td>4</td>
</tr>
</tbody>
</table>
Figure 4.5.12 Stress influence factor (I) at a depth of 762 mm (1.67B) underneath the center of footing versus applied footing pressure.

Figure 4.5.13 Stress influence factor (I) at a depth of 610 mm (1.33B) underneath the center of footing versus applied footing pressure.
Because of a serviceability requirement, design of foundations is generally controlled by settlement criteria. To evaluate the settlement of foundation, the stress distribution in soil due to applied load should be evaluated. Currently, methods based on elastic solutions such as the Boussinesq solution are generally used to evaluate the stress distribution in foundation application. Comparison between the measured values and calculated values from these elastic methods could give us some idea how these methods work for reinforced soil.

Based on the Boussinesq solutions, the vertical stress in soil at a depth \( z \) under the corner of a uniformly loaded rectangular area can be computed from

\[
\Delta \sigma_z = \frac{q}{4\pi} \left[ \frac{2m \cdot n(m^2 + n^2 + 1)^{1/2}}{m^2 + n^2 + m^2 n^2 + 1} \cdot \frac{m^2 + n^2 + 2}{m^2 + n^2 + 1} + \tan^{-1} \frac{2m \cdot n(m^2 + n^2 + 1)^{1/2}}{m^2 + n^2 - m^2 n^2 + 1} \right] = q I_B \quad (4.3)
\]

where \( \Delta \sigma_z \) is the increase in vertical stress due to load, \( q \) is the applied footing pressure, \( m = B/z \), \( n = L/z \), \( B \) is the width of footing, \( L \) is the length of footing, \( z \) is the depth, and \( I_B \) is the stress influence factor for Boussinesq solution.

For soil reinforced by stiff horizontal layers which prevent horizontal deformation, the elastic solution was given by Westergaard (1938). Similar to Boussinesq solution, the vertical stress in Westergaard material at a depth \( z \) under the corner of a rectangular uniformly loaded area can be evaluated from

Figure 4.5.14 Stress influence factor (I) at a depth of 456 mm (1.0B) underneath the center of footing versus applied footing pressure
\[ \Delta \sigma_z = q I_w \] (4.4)

where \( I_w \) is the stress influence factor for the Westergaard solution.

The comparison between measured and calculated stress distribution is presented in Figures 4.5.15 through 4.5.18 for footing pressure of 43 kPa and 468 kPa and Figures D.5 through D.8 on Appendix D for other footing pressures.

For vertical stress distribution along the center line of footing at a depth of 762 mm (Figures 4.5.15 and D.5), Boussinesq solution matches some cases of reinforced soil very well at a footing pressure of 468 kPa and lower while Westergaard solution matches some cases of reinforced soil at a footing pressure of 255 kPa and lower. After that, however, both solutions underestimate the vertical stresses in soil directly below the footing, especially at the point where maximum vertical stress occurred. As compared to unreinforced case, it can be seen from these Figures that Boussinesq solution underestimates the maximum stress at point beneath the center of footing, but gives a more uniform stress distribution.

For vertical stress distribution along the center line of footing at a depth of 610 mm (Figures 4.5.16 and D.6), Boussinesq solution overestimates the measured stress distribution for reinforced soil at a footing pressure of 43 kPa; But it then matches some cases of reinforced soil at a footing pressure of 468 kPa and lower. On the other hand, Westergaard solution matches some cases of reinforced soil at a footing pressure of 255 kPa and lower. Both solutions underestimate the vertical stresses in soil immediately below the footing, especially at the point where the maximum vertical stress occurred at higher footing pressures. For the unreinforced case, the stress was only measured at the point under the center of footing due to the fact that not enough pressure cells were available at that time.

For vertical stress distribution along the center line of footing at a depth of 457 mm (Figures 4.5.17 and D.7), the same observation as made in vertical stress distribution at a depth of 762 mm was obtained for Boussinesq solution. On the other hand, Westergaard solution underestimates the measured vertical stress in soil immediately below the footing at all footing pressure levels. For the unreinforced case, again the stress was only measured at the point under the center of footing.

For vertical stress distribution along the depth at the center of footing (Figures 4.5.18 and D.8), the match between the Boussinesq and measured stress distribution for unreinforced soil at a relatively low footing pressure of 43 kPa is pretty good while Westergaard solution underestimate the measured stress distribution for reinforced soil. At relatively medium footing
pressures of 255 kPa and 468 kPa, Boussinesq solution matches the measured stress distribution for some cases of reinforced soil. At a footing pressure of 723 kPa and higher, both Boussinesq and Westergaard solutions underestimate the measured stress distributions for all cases. For unreinforced silty clay, the underprediction of stresses below the center of footing is about 40% with Boussinesq solution. For reinforced silty clay, the underestimates are 35% and 60% with Boussinesq and Westergaard solution, respectively. This behavior may be expected in the light of the fact that Boussinesq and Westergaard solutions assume elasticity and constant modulus of elasticity of soil. These assumptions may be justified at low footing pressures but at high footing pressures, it cannot stand. Other conditions, which these elastic solutions are based on, such as homogeneous, isotropic, and free of initial stress, would also have significant influence on the stress distributions within soil in practice.

From above analysis, it is noted that elastic solutions have limitations in predicting the stress distribution in silty clay soil. However, under a relatively medium pressure, these elastic solutions may be used to give acceptable estimation of stress distribution in silty clay soils. At a relatively high pressure, a correction coefficient for the underestimate may be needed.

4.5.4 Strain Distribution along the Reinforcement

Three field tests on silty clay reinforced with four layers of geogrids (BX6100, BX6200, BX1500) were instrumented with strain gauges to evaluate the strain distribution along the reinforcement. The geogrids with instrumentations were placed at the top and bottom layers (at a depth of 152 mm (0.33B) and 762 mm (1.67B), respectively). During the installment of the geogrid in the test, considerable care was taken to place the geogrid as flat as possible, but local bending was still likely to happen, especially during subsequent compaction and loading process. In order to cancel/minimize the influence of bending of the geogrid on the interpretation of tensile force, pair of strain gauges system were therefore installed on the geogrids (one attached to the top face of geogrid and the other one attached to the bottom face of geogrid). The average reading from the pair of strain gauges was then used to calculate the strain and eventually the tensile force developed in the geogrid. The variations of strains measured along the centerline of the geogrid for different settlement ratios(s/B) are presented in Figures 4.5.19 through 4.5.24. The measured tensile strain is the largest at the point beneath the center of the footing and becomes almost negligible at about 2.0B from the center of footing. It indicates that the geogrid beyond the effective length (4.0B) results in insignificant mobilized tensile strength, and thus provides negligible effects on the improved performance of reinforced silty clay soil. In regard to
(a). Applied footing pressure $q = 43 \text{ kPa}$

(b). Applied footing pressure $q = 468 \text{ kPa}$

Figure 4.5.15 Measured and calculated vertical stress distribution along the center line of footing at a depth of 762 mm
(a). Applied footing pressure $q = 43$ kPa

(b). Applied footing pressure $q = 468$ kPa

Figure 4.5.16 Measured and calculated vertical stress distribution along the center line of footing at a depth of 610 mm
Figure 4.5.17 Measured and calculated vertical stress distribution along the center line of footing at a depth of 457 mm

(a). Applied footing pressure $q = 43$ kPa

(b). Applied footing pressure $q = 468$ kPa
Figure 4.5.18 Measured and calculated vertical stress distribution with the depth at the center of footing

(a). Applied footing pressure $q = 43$ kPa

(b). Applied footing pressure $q = 468$ kPa
the fact that the maximum strain measured is about 2%, the tensile modulus at the 2% strain was then used to evaluate the tension in geogrid. The calculated tension is shown in Figures D.9 through D.14 on Appendix D.

From Figures 4.5.19 through 4.5.24, it can be seen that in general the strain distribution in machine direction of geogrid is almost the same as that in cross machine direction in spite of the relatively lower tensile modulus in machine direction for geogrids used in the present study. BX6100 geogrid, BX6200 geogrid and BX1500 geogrid, which are made of the same material, have obviously different tensile modulus; however, the developed strain between these geogrids is very similar at the same settlement. This finding suggests that the strain developed along the reinforcement seems to be mainly related to the settlement. At the same settlement, the higher tension, therefore, would be developed in higher modulus geogrid (Figures D.9 through D.14); and the improved performance of reinforced soil foundation is believed to be positively related to the tension developed in the geogrid. It shows again that the performance of reinforced silty clay soil would improve with increasing the geogrid tensile modulus.

Compressive strain was measured in geogrid located beyond 0.75~1.0B from the center of footing. This means that the geogrid past this length cannot restrain lateral soil shear flow and works as an anchorage unit to prevent geogrid from failing by pull out.

4.6 Summary, Conclusions, and Discussions

A total of one hundred seventeen tests, including thirty eight laboratory model tests on silty clay, fifty one laboratory model tests on sand, twenty two laboratory model tests on Kentucky crushed limestone, and six large scale field tests on silty clay, were performed at Louisiana Transportation Research Center to study the behavior of reinforced soil foundation. Three types of soil and eleven types of reinforcement were used in this testing program. The stress distribution in soil and the strain distribution along the reinforcement were also monitored in this study. The test results of different types of soil are summarized in Table 4.8 for comparison.

The test results showed that the inclusion of reinforcement can appreciably improve the soil’s bearing capacity and reduce the footing settlement. The optimum depth to place the first reinforcement layer was estimated to be about 1/3B below footing for all soil tested in this study.

The bearing capacity of reinforced soil increases with increasing number of reinforcement layers. However, the significance of an additional reinforcement layer decreases with the increase in number of layers. The reinforcing effect becomes negligible below the influence
Figure 4.5.19 Strain distribution along the center line of BX6100 geogrid placed at a depth of 152 mm
Figure 4.5.20 Strain distribution along the center line of BX6100 geogrid placed at a depth of 762 mm
Figure 4.5.21 Strain distribution along the center line of BX6200 geogrid placed at a depth of 152 mm
Figure 4.5.22 Strain distribution along the center line of BX6200 geogrid placed at a depth of 762 mm
Figure 4.5.23 Strain distribution along the center line of BX1500 geogrid placed at a depth of 152 mm
Figure 4.5.24 Strain distribution along the center line of BX1500 geogrid placed at a depth of 762 mm
Table 4.8 Comparison of test results for different types of soils

<table>
<thead>
<tr>
<th></th>
<th>Silty Clay</th>
<th>Sand</th>
<th>Crushed Limestone</th>
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<tbody>
<tr>
<td></td>
<td>Test</td>
<td>Recommended</td>
<td>Test</td>
</tr>
<tr>
<td>$(u/B)_{opt}$</td>
<td>1/3</td>
<td>1/3</td>
<td>1/3</td>
</tr>
<tr>
<td>$(d/B)_{cri}$</td>
<td>1.25 ~1.5</td>
<td>1.5</td>
<td>1.25</td>
</tr>
<tr>
<td>$(h/B)_{opt}$</td>
<td>No</td>
<td>1/3</td>
<td>No</td>
</tr>
<tr>
<td>$(l/B)_{eff}$</td>
<td>4 ~ 6</td>
<td>5</td>
<td>6</td>
</tr>
</tbody>
</table>

$J$ BCR increases with increasing tensile modulus of reinforcement

depth. The influence depth of reinforced sand was obtained at approximately 1.25B in this study regardless of the type of reinforcement and footing embedment depth, while the influence depth of geogrid and geotextile reinforced silty clay was obtained as about 1.5B and 1.25B, respectively.

The BCR values decrease with increasing vertical spacing of reinforcement layers. No optimum vertical spacing was observed for the geogrid reinforced silty clay and sand tested. For the tested soil (silty clay and sand) and geogrid reinforcement, one can realize that the smaller the spacing, the higher the BCR. In practice, cost would govern the spacing and require 6 in. ≤ $h$ ≤ 18 in. For design purpose, engineers need to balance between reducing spacing and increasing geogrid tensile modulus. The author believes that a value of $h/B = 1/3$ can be a reasonable value for use in the design of reinforced soil.

In general, the performance of reinforced soil improves with increasing the reinforcement tensile modulus. Because of a serviceability requirement, foundations are designed at a limited settlement level in most cases. From an engineering practice point of view, geogrid reinforcement is generally considered to perform better for soil foundation than geotextile reinforcement. However, it should be noticed that the selection of the type of reinforcement in engineering practice is a project-dependent issue (Guido et al., 1986). For example, some projects require that geosynthetics only function as reinforcement, while on other projects geosynthetics are required to function as both reinforcement and separator or filter in which relatively poor reinforcement is also acceptable.
The reinforcement of soil usually results in redistribution of the applied load to a wider area, thus minimizing stress concentration and achieving improved distribution of induced stress. From test results, it seems that the improvement of stress distribution in reinforced soil is somehow related to the tensile modulus of reinforcement and better tension membrane effect provides better reduction in stress distribution. The redistribution of applied load to a wider area below the reinforced zone results in reducing the consolidation settlement of underlying weak clayey soil, which is directly related to the induced stress. With the appropriate reinforcement configuration, the inclusion of reinforcement can develop “surcharge effect” to prevent soil from moving upward, and thus improve the bearing capacity of soil. From field test results, it can be seen that, at relatively low to medium pressure, elastic solutions may be used to give acceptable estimation of stress distribution in silty clay. At relatively high pressure, a correction coefficient for the underestimate is needed.

Negligible strain measured in geogrid at about 2.0~3.0B from the center of footing indicates geogrid beyond the effective length (4.0~6.0B) results in insignificant mobilized tensile strength, and thus provides negligible reinforcement effect. For geogrids with different tensile modulus, the developed strain along the geogrids is very similar at the same settlement level. This finding indicates that the strain developed along the geogrid is directly related to the settlement, and therefore higher tension would be developed for geogrid with higher modulus under the same footing settlement.

The footing settlements reported in this study were the average of the two dial gauge readings. This average reading can cancel any possible influence of the tilting of the bearing plate during the loading process. An in-isolation calibration test was conducted to examine the accuracy of the strain measured by strain gauge and the influence of environmental protection (coating) (Appendix E). The measured strain data presented in this study were corrected based on this calibration test. The corresponding correction factors are present in Table E.1 on Appendix E. The pressure cells were calibrated by simply placing some dead weight on the cells before each installation. Although this calibration technique is not very accurate, it is simple and did give the author some confidence with the accuracy of the stress measured by pressure cells.

Due to limited time, only one test was conducted for each case in this study. However, two cases for model footing test on sand were selected to check the repeatability of test sections. The pressure-settlement curves corresponding to these two cases are presented in Figure 4.6.1 and 4.6.2. It can be seen from Figure 4.6.1 that the pressure-settlement curves for the three model
tests with two layers of BasXgrid 11 geogrid are very close and located in a very narrow band. This indicates that the repeatability of the model tests is excellent. Similarly, the pressure-settlement curves for the two model tests with three layers of BasXgrid 11 geogrid placed at a spacing of 51 mm were very close (Figure 4.6.2). These results allow the author to have greater confidence with the reproducibility of test sections. However, to more accurately account for the test variation, for each case at least three tests are needed.

Figure 4.6.1 Pressure-settlement curves for repeated model footing tests on sand with two layers of BasXgrid 11 geogrid (B×L: 152 mm×152mm; D/B: 1.0)
### Figure 4.6.2 Pressure-settlement curves for repeated model footing tests on sand with three layers of BasXgrid 11 geogrid (B×L: 152 mm×152mm; $D_f/B$: 1.0)

<table>
<thead>
<tr>
<th>Type</th>
<th>N</th>
<th>u</th>
<th>h</th>
</tr>
</thead>
<tbody>
<tr>
<td>BasXgrid11</td>
<td>3</td>
<td>51</td>
<td>51</td>
</tr>
<tr>
<td>BasXgrid11 (Repeated)</td>
<td>3</td>
<td>51</td>
<td>51</td>
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<table>
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<tr>
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<tr>
<td>0</td>
</tr>
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<td>1000</td>
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<tr>
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</tr>
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<td>6000</td>
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<table>
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<tr>
<th>Relative Settlement ($s/B$)</th>
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<tbody>
<tr>
<td>0.00</td>
</tr>
<tr>
<td>0.04</td>
</tr>
<tr>
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</tr>
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<td>0.12</td>
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<td>0.16</td>
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<table>
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<th>Footing Settlement (mm)</th>
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<tbody>
<tr>
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</tr>
<tr>
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</tr>
<tr>
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</tr>
<tr>
<td>10</td>
</tr>
<tr>
<td>15</td>
</tr>
<tr>
<td>20</td>
</tr>
<tr>
<td>25</td>
</tr>
</tbody>
</table>

The graph shows the pressure-settlement curves for repeated model footing tests on sand with three layers of BasXgrid 11 geogrid. The tests were conducted under varying applied pressures, and the relative settlement ($s/B$) and footing settlement (mm) are plotted against the applied pressure (kPa). The data points are differentiated by the type of geogrid used, with BasXgrid11 represented by square markers and BasXgrid11 (Repeated) represented by diamond markers.
CHAPTER 5 STATISTICAL ANALYSIS OF TESTS RESULTS

5.1 Introduction

Empirical estimation methods are often used in many Geotechnical Engineering applications. This is generally done by estimating the dependent variable (e.g. bearing capacity of reinforced soil) based on the independent variables (e.g. top layer spacing (u) and number of reinforcement layers (N)). Bearing capacity of small-scale model tests on reinforced soil conducted by several researchers indicates numerous factors would influence the performance of RSF. Considering all factors in the design is not practical, and nor is it necessary. Based on the model test results, multiple linear regression analysis of the bearing capacity is conducted in this study. A parametric study was performed to identify the vital significant parameters that influence the performance of the RSF. A regression model is then developed to estimate the bearing capacity of RSF using the identified significant parameters. All statistical analysis is conducted using the Statistical Analysis Software (SAS) package.

5.2 Multiple Linear Regression Analysis

5.2.1 Regression Model

Regression analysis is a statistical methodology used to examine the relationship between a dependent variable and a set of independent variables. In multiple linear regression analysis, it is hypothesized that this relationship is linear and has the following form:

\[ y_i = \beta_0 + \beta_1 x_{i1} + \beta_2 x_{i2} + \cdots + \beta_k x_{ik} + \epsilon_i \]  

(5.1)

where \( i=1, 2, \ldots, n \) and \( n \) is the number of observations; \( y_i \) is dependent variable; \( x_{i1}, x_{i2}, \ldots, x_{ik} \) are independent variables; \( \beta_0, \beta_1, \ldots, \beta_k \) are unknown parameters; \( \epsilon_i \) is random error. It should be noted that this model is called “linear” because of it’s linearity in \( \beta \)’s, not in the \( x \)’s.

Applying matrix notation, multiple regression model can be presented in a compact form:

\[ y = X\beta + \epsilon \]  

(5.2)

where

\[ y = \begin{bmatrix} y_1 \\ y_2 \\ \vdots \\ y_n \end{bmatrix}, \quad X = \begin{bmatrix} 1 & x_{11} & \cdots & x_{1k} \\ 1 & x_{21} & \cdots & x_{2k} \\ \vdots & \vdots & \ddots & \vdots \\ 1 & x_{n1} & \cdots & x_{nk} \end{bmatrix}, \quad \beta = \begin{bmatrix} \beta_0 \\ \beta_1 \\ \vdots \\ \beta_k \end{bmatrix}, \quad \epsilon = \begin{bmatrix} \epsilon_1 \\ \epsilon_2 \\ \vdots \\ \epsilon_n \end{bmatrix} \]
5.2.2 Fitting the Model

Least squares method can be used to fit the model. The least squares estimate of $\beta$ can be obtained by minimizing:

$$S = \sum_{i=1}^{n} \left[ y_i - \left( \beta_0 + \beta_1 x_{i1} + \beta_2 x_{i2} + \cdots + \beta_k x_{ik} \right) \right]^2 = (y - X\hat{\beta})'(y - X\hat{\beta})$$  \hspace{1cm} (5.3)

Minimizing $S$ by setting the derivative of this formula with respect to $\beta$ to zero:

$$\frac{\partial S}{\partial \beta} = -2X'(y - X\hat{\beta}) = 0$$  \hspace{1cm} (5.4)

where $\hat{\beta}$ is the least squares estimate of $\beta$.

Now, $\hat{\beta}$ can be obtained:

$$\hat{\beta} = (XX)^{-1}Xy$$  \hspace{1cm} (5.5)

5.2.3 Significance Test for the Overall Model

Significance test for the overall model is a test to determine the effectiveness of the entire model, i.e. whether the linear relationship exists between the dependent variable and independent variables. This is generally done by testing the null hypothesis: $H_0: \beta_1 = \beta_2 = \cdots = \beta_k = 0$ against the alternative hypothesis $H_1$: at least one of the $\beta_j$ is non-zero. The null hypothesis implies that none of the independent variables are linearly related to the dependent variable in the assumed multiple regression equation. The alternative hypothesis suggests at least one of the independent variables is linearly related to the dependent variable. This hypothesis can be tested by a comparison of Mean Square Regression (MSR) and Mean Square Error (MSE). This test is an $F$ statistic. The best way for this test is to use Analysis of Variance (ANOVA). ANOVA table are generally used for the ANOVA calculations and it has the following general form:

<table>
<thead>
<tr>
<th>Degrees of freedom</th>
<th>Sum of Squares</th>
<th>Mean Square</th>
<th>$F$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression</td>
<td>$k$</td>
<td>SSR</td>
<td>MSR</td>
</tr>
<tr>
<td>Error</td>
<td>$n-k-1$</td>
<td>SSE</td>
<td>MSE</td>
</tr>
<tr>
<td>Total</td>
<td>$n-1$</td>
<td>SST</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.1 ANOVA table for multiple linear regression
The terms displayed in Table 5.11 are defined and computed as follows:

\[
\begin{align*}
SST &= \sum_{i=1}^{n} (y_i - \bar{y})^2 \quad \text{total sum of squares} \\
SSE &= \sum_{i=1}^{n} (y_i - \hat{y}_i)^2 \quad \text{sum of squares due to error} \\
SSR &= \sum_{i=1}^{n} (\hat{y}_i - \bar{y})^2 \quad \text{sum of squares due to regression} \\
MSE &= \frac{SSE}{n - k - 1} \quad \text{mean square due to error} \\
MSR &= \frac{SSR}{k} \quad \text{mean square due to regression}
\end{align*}
\]

Where \(\hat{y}_i\) are the predicted values, \(\bar{y}\) is the mean of dependent variables.

The three sums of squares are related by the formula:

\[
SST = SSR + SSE \tag{5.6}
\]

Rejecting null hypothesis (H_0) if \(F > F_{\alpha,k,n-k-1}\); failing to reject null hypothesis (H_0), if \(F \leq F_{\alpha,k,n-k-1}\). \(\alpha\) is the significance level.

5.2.4 Goodness of Fit of the Model

The quality of the fit can be measured by the sum of the squares of the residuals, which is defined as:

\[
e_i = y_i - \hat{y}_i \tag{5.7}
\]

A good fit should have small residuals. However, this quantity is dependent on the units of \(y_i\). Thus the coefficient of multiple determination, \(R^2\), is generally used to measure the goodness of fit.

\[
R^2 = \frac{SSR}{SST} = 1 - \frac{SSE}{SST} = 1 - \frac{\sum_{i=1}^{n} (y_i - \hat{y}_i)^2}{\sum_{i=1}^{n} (y_i - \bar{y})^2} \tag{5.8}
\]

\(R^2\) ranges from 0 to 1. The closer it is to 1, the better the fit. \(R^2\) equal to 1 means perfect linear relationship exists between the dependent variable and independent variables, while \(R^2\) equal to 0 indicates independent variables have no impact on the dependent variable. \(R^2\) can only
increase by adding more independent variables to a model. This is because SST is always the same for a given set of observations and SSE never increases with the inclusion of an additional independent variable. Since a large value of $R^2$ made by adding more dependent variables means nothing, it is often advisable to use the adjusted coefficient of multiple determination ($R^2_a$) as an alternative measure of fit.

$$R^2_a = 1 - \frac{SSE/(n-k-1)}{SST/(n-1)} = 1 - \frac{MSE}{MST}$$  \hspace{1cm} (5.9)

MSE is the estimate of standard error ($\sigma^2$), i.e. $s^2 = MSE$. It is easy to show when the number of observations $n$ is large, the approximate width of 95% confidence interval for a future observation is $4s$. Therefore, the quality of the fit can also be assessed by $s^2$. The smaller the values of $s^2$ are, the better the fit. This measurement provides an excellent indication of the quality of the fit when the prediction is important for the model. In most cases, both $R^2$ (and $R^2_a$) and $s^2$ needs to be considered to assess the goodness of fit.

### 5.2.5 Significance Tests for Individual Regression Coefficients

If null hypothesis ($H_0$) in significance test for the entire model is rejected, it only indicates at least one of the $\beta_j$ is non-zero. Then the additional tests are needed to determine which these $\beta_j$ are. Significance tests for individual regression coefficients would be useful for this determination. This is generally done by testing the null hypothesis: $H_0: \beta_j = 0$ against the alternative hypothesis $H_1: \beta_j \neq 0$. If null hypothesis ($H_0$) is not rejected, it indicates the independent variable $x_j$ can be removed from the regression model. This test is a $t$ statistic and can be written as

$$t = \frac{\hat{\beta}_j}{SE_{\hat{\beta}_j}} = \frac{\hat{\beta}_j}{\sqrt{c_{jj}MSE}}$$  \hspace{1cm} (5.10)

Where $SE_{\hat{\beta}_j}$ is the standard error of the regression coefficient $\hat{\beta}_j$, and $c_{jj}$ is diagonal element of $(X'X)^{-1}$ corresponding to $\hat{\beta}_j$. Rejecting null hypothesis ($H_0$), if $|t| > t_{\alpha/2,n-k-1}$; failing to reject null hypothesis ($H_0$), if $|t| \leq t_{\alpha/2,n-k-1}$. (1-$\alpha$)% Confidence Interval (CI) for $\beta_j$ can be constructed as following:
\[
(1 - \alpha)\%CI(\beta_j) = \hat{\beta}_j \pm t_{\alpha/2,n-k-1}\sqrt{\text{MSE}}
\]  
(5.11)

Where \( a \) is the significance level

### 5.2.6 Variable Included in the Analysis and Selection Technique

The parameters used in the regression analysis are selected as top layer spacing ratio \((u/B)\), number of layers \((N)\), vertical spacing between layers \((h/B)\), tensile modulus of reinforcement \((J)\), and settlement ratio \((s/B)\). Because all other variables are dimensionless, the tensile modulus \((J)\) is normalized to 100kN/m and then used in the model, i.e. \( \frac{J}{100\text{kN/m}} \) instead of \( J \) is used in the model.

Since each of the dependent variables may be either included or not included in the regression analysis, \( 2^k \) subset regression equations without interaction terms can be formed for a data set with \( k \) independent variables. It is required to determine which regression model is the best. Two variable selection techniques widely used are all possible regressions technique and stepwise procedure technique. Because all possible regressions technique evaluates all possible subsets of regression models, it is generally preferred over stepwise procedure. However, this technique needs a large amount of computations. The availability of powerful computer program (e.g. SAS) makes this technique feasible up to a certain number of independent variables (10~20) from both economical and technical point of views. The model with the largest \( R^2 \) (\( R_a^2 \)) and the smallest \( s \) is generally considered as the best model. Another criterion is often used to determine the best model is Akaike’s Information Criteria (AIC).

\[
AIC = n \cdot \ln \left( \frac{\text{SSE}}{n} \right) + 2(k + 1)
\]  
(5.12)

The first term is a measure of goodness of fit while the second term is a penalty term accounting for additional dependent variables. The lower the AIC value, the better the model.

### 5.3 Regression Analysis of Small-Scale Tests on Silty Clay

Based on the experimental results a multi-regression statistical analysis was conducted to develop a BCR model that can facilitate the estimate of bearing capacity of a geogrid reinforced soil foundation. In developing the BCR model, all the geogrid layers were assumed to have
enough length to fully mobilize its tensile contribution. Both linear and nonlinear curve fittings are performed in this study. The significance level $\alpha$ is set to 0.05 in this study.

### 5.3.1 Linear Model

A linear regression model described in Equation 5.13 is selected to investigate the effects of all variables.

$$BCR = \beta_0 + \beta_1 \left( \frac{s}{B} \right) + \beta_2 \left( \frac{u}{B} \right) + \beta_3 N + \beta_4 \left( \frac{h}{B} \right) + \beta_5 \left( \frac{J}{100kN/m} \right)$$  \hspace{1cm} (5.13)

The values of $R^2$, adjust $R^2$, $s$, and AIC are calculated for all of subset regression model. It is found that the model including $s/B$, $u/B$, $h/B$ and $N$ has the largest value of $R^2$ and $R_{\text{adj}}^2$ and the smallest value of $s$ and AIC (Appendix F). From the statistical point of view, the model including $s/B$, $u/B$, $h/B$ and $N$ is considered to be the best regression model. The multiple regression analysis is then conducted on this best model and the results yielded the following equation:

$$BCR = 0.902 + 1.083 \left( \frac{s}{B} \right) - 0.141 \left( \frac{u}{B} \right) + 0.151 N + 0.066 \left( \frac{h}{B} \right)$$  \hspace{1cm} (5.14)

To determine the effectiveness of the entire model, significance test for the overall model is performed. Table 5.2 presents the results of ANOVA calculations.

<table>
<thead>
<tr>
<th>Degrees of freedom (DF)</th>
<th>Sum of Squares</th>
<th>Mean Square</th>
<th>F</th>
<th>Pr&gt;F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression</td>
<td>4</td>
<td>38.071</td>
<td>9.518</td>
<td>959.37</td>
</tr>
<tr>
<td>Error</td>
<td>570</td>
<td>5.655</td>
<td>0.00992</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>574</td>
<td>43.726</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The null hypothesis is rejected because $\alpha$ is less than 0.0001 for Pr>F. This suggests that at least one of the independent variables is linearly related to the dependent variable. To determine the independent variables with linear relation, significance tests for individual regression coefficients are then conducted by using $t$ statistics. The results of these $t$ statistics are summarized in Table 5.3.

The null hypothesis is rejected because $\alpha$ is less than 0.05 for Pr>$|t|$. This means that all independent variables are linearly related to the dependent variable.
Table 5.3 t statistics for linear model for silty clay

| Variable | Degrees of freedom (DF) | Parameter | Standard Error | t   | Pr>|t| |
|----------|-------------------------|-----------|----------------|-----|-----|
| Intercept| 1                       | \(\beta_0=0.90151\) | 0.01562         | 57.72 | <0.0001 |
| s/B      | 1                       | \(\beta_1=1.08302\)  | 0.05786         | 18.72 | <0.0001 |
| u/B      | 1                       | \(\beta_2=-0.14114\) | 0.01805         | -7.82 | <0.0001 |
| N        | 1                       | \(\beta_3=0.15099\)  | 0.00420         | 35.96 | <0.0001 |
| h/B      | 1                       | \(\beta_4=0.0663\)   | 0.03207         | 2.07  | 0.0391 |

The statistical analysis showed that reinforcement tensile modulus in the examining range of this study has insignificant effect on the BCR for linear regression model.

5.3.2 Nonlinear Model

Based on the plots of BCR vs. N, BCR vs. s/B, BCR vs. u/B, and BCR vs. h/B presented in Section 4.2, a nonlinear model is chosen as follows:

\[
BCR = \beta_0 \left( \frac{s}{B} \right)^{\beta_1} \left( \frac{u}{B} \right)^{\beta_2} \left( \frac{h}{B} \right)^{\beta_3} \left( \frac{J}{100kN/m} \right)^{\beta_4} \tag{5.15}
\]

After performing logarithmic transformation, the above model can be rewritten as:

\[
\ln(BCR) = \ln(\beta_0) + \beta_1 \ln \left( \frac{s}{B} \right) + \beta_2 \ln \left( \frac{u}{B} \right) + \beta_3 \ln \left( \frac{h}{B} \right) + \beta_4 \ln \left( \frac{J}{100kN/m} \right) \tag{5.16}
\]

The values of \(R^2\), adjust \(R^2\), \(s\), and \(AIC\) are calculated for all of subset regression models, and it is found that the best model includes all the variables (Appendix F). The multiple regression analysis is then conducted on this best model and the results yielded the following equation:

\[
\ln(BCR) = 0.237 + 0.054 \ln \left( \frac{s}{B} \right) - 0.102 \ln \left( \frac{u}{B} \right) + 0.316 \ln N - 0.15 \ln \left( \frac{h}{B} \right) + 0.026 \ln \left( \frac{J}{100kN/m} \right) \tag{5.17}
\]

To determine the effectiveness of the entire model, significance test for the overall model is performed. Table 5.4 presents the ANOVA calculations.

It can be seen from Table 5.6 that at least one of the independent variables is linearly related to the dependent variable. To determine the independent variables with linear relation,
Table 5.4 ANOVA table for nonlinear model for silty clay

<table>
<thead>
<tr>
<th></th>
<th>DF</th>
<th>Sum of Squares</th>
<th>Mean Square</th>
<th>F</th>
<th>Pr&gt;F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression</td>
<td>5</td>
<td>21.873</td>
<td>4.37468</td>
<td>1290.83</td>
<td>&lt;0.0001</td>
</tr>
<tr>
<td>Error</td>
<td>569</td>
<td>1.928</td>
<td>0.00339</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>574</td>
<td>23.802</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The results of these \( t \) statistics are summarized in Table 5.5.

Table 5.5 \( t \) statistics for nonlinear model for silty clay

| Variable | DF | Parameter | Standard Error | \( t \) | Pr>|t| |
|----------|----|-----------|----------------|--------|-------|
| Intercept| 1  | \( \ln(\beta_0) = 0.23693 \) | 0.01758 | 13.48 | <0.0001 |
| \( \ln(s/B) \) | 1  | \( \beta_1 = 0.0536 \) | 0.00246 | 21.80 | <0.0001 |
| \( u/B \) | 1  | \( \ln(\beta_2) = -0.10218 \) | 0.01085 | -9.42 | <0.0001 |
| \( \ln(N) \) | 1  | \( \beta_3 = 0.31564 \) | 0.00689 | 45.79 | <0.0001 |
| \( h/B \) | 1  | \( \ln(\beta_4) = -0.14989 \) | 0.02216 | -6.76 | <0.0001 |
| \( \ln\left( \frac{J}{100kN/m} \right) \) | 1  | \( \beta_5 = 0.02604 \) | 0.01329 | 1.96  | 0.0506 |

It can be seen from Table 5.7 that for \( \ln\left( \frac{J}{100kN/m} \right) \) \( \alpha \) is equal to 0.506 which is just a little bit larger than the prescribed significance level 0.05. For all other independent variables, \( \alpha \) is less than the prescribed significance level 0.05. It may be reasonable not to reject the null hypothesis for all independent variables. This suggests that all independent variables are linearly related to the dependent variable. After transforming Equation (5.17) back to original nonlinear model, the above model can be rewritten as:

\[
BCR = 1.267 \left( \frac{S}{B} \right)^{0.054} \left( 0.903 \right)^{\ln(B)} N^{0.316} \left( 0.861 \right)^{\ln(h)} \left( \frac{J}{100kN/m} \right)^{0.026} \tag{5.18}
\]

5.3.3 Verification of the Models

For verification of the regression BCR models in Equations 5.14 and 5.18, the results of regression models are compared with twenty four experimental data which are not included in the model development. The detailed variables and comparison are presented in Tables 5.6. The
absolute error in predicting the BCR value is calculated for each case and presented in the table. The absolute errors range from 0.07 ~ 7.97 % and 0.02 ~ 7.75% for linear and nonlinear model, respectively. This suggests that the BCR values predicted by both regression models have good accuracy.

Table 5.6 Verification of regression models for silty clay

<table>
<thead>
<tr>
<th>s/B</th>
<th>u/B</th>
<th>N</th>
<th>h/B</th>
<th>BCR (measured)</th>
<th>BCR (linear)</th>
<th>Abs (Err) (%)</th>
<th>BCR (nonlinear)</th>
<th>Abs (Err) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.23</td>
<td>1.34</td>
<td>7.97</td>
<td>1.33</td>
<td>7.75</td>
</tr>
<tr>
<td>0.02</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.37</td>
<td>1.35</td>
<td>1.09</td>
<td>1.38</td>
<td>1.01</td>
</tr>
<tr>
<td>0.03</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.41</td>
<td>1.36</td>
<td>3.41</td>
<td>1.41</td>
<td>0.02</td>
</tr>
<tr>
<td>0.04</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.42</td>
<td>1.37</td>
<td>2.99</td>
<td>1.43</td>
<td>1.23</td>
</tr>
<tr>
<td>0.05</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.42</td>
<td>1.38</td>
<td>2.38</td>
<td>1.45</td>
<td>2.29</td>
</tr>
<tr>
<td>0.06</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.41</td>
<td>1.39</td>
<td>1.03</td>
<td>1.46</td>
<td>3.92</td>
</tr>
<tr>
<td>0.07</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.41</td>
<td>1.41</td>
<td>0.19</td>
<td>1.48</td>
<td>4.86</td>
</tr>
<tr>
<td>0.08</td>
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<td>3</td>
<td>1/3</td>
<td>1.40</td>
<td>1.42</td>
<td>1.31</td>
<td>1.49</td>
<td>6.38</td>
</tr>
<tr>
<td>0.09</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.40</td>
<td>1.43</td>
<td>2.07</td>
<td>1.50</td>
<td>7.05</td>
</tr>
<tr>
<td>0.10</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.43</td>
<td>1.44</td>
<td>0.88</td>
<td>1.51</td>
<td>5.61</td>
</tr>
<tr>
<td>0.11</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.45</td>
<td>1.45</td>
<td>0.29</td>
<td>1.51</td>
<td>4.14</td>
</tr>
<tr>
<td>0.12</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.48</td>
<td>1.46</td>
<td>1.53</td>
<td>1.52</td>
<td>2.56</td>
</tr>
<tr>
<td>0.13</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.48</td>
<td>1.47</td>
<td>0.68</td>
<td>1.53</td>
<td>3.13</td>
</tr>
<tr>
<td>0.14</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.49</td>
<td>1.48</td>
<td>0.28</td>
<td>1.53</td>
<td>3.20</td>
</tr>
<tr>
<td>0.15</td>
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<td>3</td>
<td>1/3</td>
<td>1.50</td>
<td>1.49</td>
<td>0.74</td>
<td>1.54</td>
<td>2.35</td>
</tr>
<tr>
<td>0.16</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.53</td>
<td>1.50</td>
<td>1.72</td>
<td>1.54</td>
<td>0.96</td>
</tr>
<tr>
<td>0.17</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.55</td>
<td>1.51</td>
<td>2.46</td>
<td>1.55</td>
<td>0.18</td>
</tr>
<tr>
<td>0.18</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.56</td>
<td>1.52</td>
<td>2.38</td>
<td>1.55</td>
<td>0.51</td>
</tr>
<tr>
<td>0.19</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.57</td>
<td>1.54</td>
<td>2.14</td>
<td>1.56</td>
<td>0.67</td>
</tr>
<tr>
<td>0.20</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.57</td>
<td>1.55</td>
<td>1.29</td>
<td>1.56</td>
<td>0.23</td>
</tr>
<tr>
<td>0.21</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.57</td>
<td>1.56</td>
<td>0.56</td>
<td>1.57</td>
<td>0.06</td>
</tr>
<tr>
<td>0.22</td>
<td>1/3</td>
<td>3</td>
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<td>1.57</td>
<td>1.57</td>
<td>0.07</td>
<td>1.57</td>
<td>0.12</td>
</tr>
<tr>
<td>0.23</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
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<td>1.58</td>
<td>0.15</td>
<td>1.57</td>
<td>0.11</td>
</tr>
<tr>
<td>0.24</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.59</td>
<td>1.59</td>
<td>0.24</td>
<td>1.58</td>
<td>0.95</td>
</tr>
</tbody>
</table>

5.4 Regression Analysis of Small-Scale Tests on Sand

5.4.1 Linear Model

A linear regression model described in Equation 5.19 is chosen to include the effects of variables investigated for footings on sand with an embedment depth 152 mm.

$$BCR = \beta_0 + \beta_1 \left( \frac{s}{B} \right) + \beta_2 \left( \frac{u}{B} \right) + \beta_3 N + \beta_4 \left( \frac{h}{B} \right) + \beta_5 \left( \frac{J}{100kN/m} \right)$$  \hspace{1cm} (5.19)
Following the same analysis process as indicated in Section 5.3.1, the following best linear model is yielded for sand:

\[
BCR = 0.894 + 2.823 \left( \frac{s}{B} \right) + 0.053N + 0.456 \left( \frac{h}{B} \right) + 0.0062 \ln \left( \frac{J}{100kN/m} \right)
\] (5.20)

The corresponding results of ANOVA calculations and t statistics are presented in Tables 5.7 and 5.8, respectively.

Table 5.7 ANOVA table for linear model for sand

<table>
<thead>
<tr>
<th></th>
<th>DF</th>
<th>Sum of Squares</th>
<th>Mean Square</th>
<th>F</th>
<th>Pr&gt;F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression</td>
<td>4</td>
<td>5.463</td>
<td>1.366</td>
<td>152.98</td>
<td>&lt;0.0001</td>
</tr>
<tr>
<td>Error</td>
<td>235</td>
<td>2.098</td>
<td>0.00893</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>239</td>
<td>7.561</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5.8 t statistics for linear model for sand

| Variable     | DF | Parameter | Standard Error | t    | Pr>|t| |
|--------------|----|-----------|----------------|------|-------|
| Intercept    | 1  | $\beta_0$= 0.89368 | 0.0232     | 38.47      | <0.0001 |
| s/B          | 1  | $\beta_1$= 2.82311 | 0.218      | 12.95      | <0.0001 |
| N            | 1  | $\beta_3$= 0.05258 | 0.0107     | 4.92       | 0.0001  |
| h/B          | 1  | $\beta_4$= 0.4557 | 0.0693     | 6.58       | <0.0001 |
| $\frac{J}{100kN/m}$ | 1  | $\beta_5$= 0.00623 | 0.00291  | 2.14       | 0.033   |

5.4.2 Nonlinear Model

The following nonlinear model is chosen based on the plots of BCR vs. N, BCR vs. s/B, BCR vs. u/B, and BCR vs. h/B presented in Section 4.3:

\[
BCR = \beta_0 \left( \frac{s}{B} \right)^{\beta_2} \left( \frac{u}{B} \right)^{\beta_3} N^{\beta_4} \left( \frac{J}{100kN/m} \right)^{\beta_5}
\] (5.21)

Using the same procedure as indicated in Section 5.3.2, the best nonlinear model for sand is developed as:

\[
BCR = 1.389 \left( \frac{s}{B} \right)^{0.0688} N^{0.123} (h/B)^{1.259}
\] (5.22)
The corresponding results of ANOVA calculations and \( t \) statistics are presented in Tables 5.9 and 5.10, respectively.

**Table 5.9 ANOVA table for nonlinear model for sand**

<table>
<thead>
<tr>
<th></th>
<th>DF</th>
<th>Sum of Squares</th>
<th>Mean Square</th>
<th>( F )</th>
<th>( Pr&gt;F )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression</td>
<td>3</td>
<td>3.59</td>
<td>1.197</td>
<td>198.26</td>
<td>&lt;0.0001</td>
</tr>
<tr>
<td>Error</td>
<td>236</td>
<td>1.424</td>
<td>0.00604</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>239</td>
<td>5.015</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table 5.10 \( t \) statistics for nonlinear model for sand**

| Variable | DF | Parameter | Standard Error | \( t \) | \( Pr>|t|\) |
|----------|----|-----------|----------------|--------|-------------|
| Intercept| 1  | ln(\( \beta_0 \))= 0.32885 | 0.0199 | 16.5 | <0.0001 |
| ln(s/B)  | 1  | \( \beta_1= 0.06882 \) | 0.00555 | 12.4 | <0.0001 |
| ln(N)    | 1  | \( \beta_3= 0.12339 \) | 0.022 | 5.61 | <0.0001 |
| h/B      | 1  | ln(\( \beta_4 \))= 0.23026 | 0.071 | 3.24 | 0.0013 |

**5.4.3 Verification of the Models**

The regression BCR models in equation 5.20 and 5.22 are verified by comparing the results of regression models with nine experimental data which are not used to develop the model. The detailed variables and comparison are presented in Table 5.7. The absolute errors vary from 0.00 ~ 4.29 % and 0.07 ~ 4.32% for linear and nonlinear model, respectively. A good agreement exits between predicted BCR and measured BCR for both regression models.

**Table 5.11 Verification of regression models for sand**

<table>
<thead>
<tr>
<th>s/B</th>
<th>u/B</th>
<th>N</th>
<th>h/B</th>
<th>BCR (measured)</th>
<th>BCR (linear)</th>
<th>Abs (Err) (%)</th>
<th>BCR (nonlinear)</th>
<th>Abs (Err) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.21</td>
<td>1.26</td>
<td>4.01</td>
<td>1.25</td>
<td>3.74</td>
</tr>
<tr>
<td>0.02</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.27</td>
<td>1.28</td>
<td>1.48</td>
<td>1.31</td>
<td>3.66</td>
</tr>
<tr>
<td>0.03</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.29</td>
<td>1.31</td>
<td>1.49</td>
<td>1.35</td>
<td>4.32</td>
</tr>
<tr>
<td>0.04</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.35</td>
<td>1.34</td>
<td>0.42</td>
<td>1.38</td>
<td>2.20</td>
</tr>
<tr>
<td>0.05</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.40</td>
<td>1.37</td>
<td>2.10</td>
<td>1.40</td>
<td>0.07</td>
</tr>
<tr>
<td>0.06</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.44</td>
<td>1.40</td>
<td>2.89</td>
<td>1.41</td>
<td>1.66</td>
</tr>
<tr>
<td>0.07</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.46</td>
<td>1.43</td>
<td>2.69</td>
<td>1.43</td>
<td>2.38</td>
</tr>
<tr>
<td>0.08</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.45</td>
<td>1.45</td>
<td>0.00</td>
<td>1.44</td>
<td>0.72</td>
</tr>
<tr>
<td>0.09</td>
<td>1/3</td>
<td>3</td>
<td>1/3</td>
<td>1.42</td>
<td>1.48</td>
<td>4.29</td>
<td>1.45</td>
<td>2.40</td>
</tr>
</tbody>
</table>
5.5 Regression Analysis of Small-Scale Tests on Kentucky Crushed Limestone

5.5.1 Linear Model

A linear regression model described in Equation 5.23 is chosen to include the effects of variables investigated in this study for crushed limestone.

\[
BCL = \beta_0 + \beta_1 \left( \frac{s}{B} \right) + \beta_2 N + \beta_3 \left( \frac{J}{100kN/m} \right)
\]  

(5.23)

Following the same analysis process as indicated in Section 5.3.1, the following best linear model is yielded for crushed limestone:

\[
BCL = 0.574 + 6.502 \left( \frac{s}{B} \right) + 0.147 N + 0.00115 \left( \frac{J}{100kN/m} \right)
\]  

(5.24)

The corresponding results of ANOVA calculations and \( t \) statistics are presented in Tables 5.12 and 5.13, respectively.

| Variable       | DF | Parameter | Standard Error | t     | Pr>|t| |
|----------------|----|-----------|----------------|-------|------|
| Intercept      | 1  | \( \beta_0 = 0.57393 \) | 0.0405 | 14.16 | <0.0001 |
| \( \frac{s}{B} \) | 1  | \( \beta_1 = 6.50154 \) | 0.43     | 15.13 | <0.0001 |
| N              | 1  | \( \beta_2 = 0.1472 \)  | 0.0154   | 9.59  | <0.0001 |
| \( \frac{J}{100kN/m} \) | 1  | \( \beta_3 = 0.00115 \) | 0.000072 | 15.93 | <0.0001 |

5.5.2 Nonlinear Model

The following nonlinear model is chosen based on the plots of BCR vs. N and BCR vs. \( s/B \) presented in Section 4.4:
Using the same procedure as indicated in Section 5.3.2, the best nonlinear model for crushed limestone is developed as:

\[
BCR = \beta_0 \left( \frac{s}{B} \right)^{\beta_1} N^{\beta_2} \left( \frac{J}{100kN/m} \right)^{\beta_3}
\]  

(5.25)

The results of corresponding ANOVA calculations and \(t\) statistics are presented in Tables 5.14 and 5.15, respectively.

**Table 5.14 ANOVA table for nonlinear model for Kentucky crushed limestone**

<table>
<thead>
<tr>
<th></th>
<th>DF</th>
<th>Sum of Squares</th>
<th>Mean Square</th>
<th>F</th>
<th>Pr&gt;F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression</td>
<td>3</td>
<td>7.642</td>
<td>2.547</td>
<td>239.44</td>
<td>&lt;0.0001</td>
</tr>
<tr>
<td>Error</td>
<td>196</td>
<td>2.085</td>
<td>0.0106</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>199</td>
<td>9.727</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table 5.15 \(t\) statistics for nonlinear model for Kentucky crushed limestone**

| Variable                  | DF | Parameter        | Standard Error | \(t\) | Pr>|\(t| |
|---------------------------|----|------------------|----------------|-------|-------|
| Intercept \(\ln(\beta_0)=0.54754\) | 1  | \(\beta_0\)      | 0.0357         | 15.34 | <0.0001 |
| \(\ln(s/B)\) \(\beta_1=0.17722\)   | 1  | \(\beta_1\)      | 0.0105         | 16.90 | <0.0001 |
| \(\ln(N)\) \(\beta_2=-0.17283\)   | 1  | \(\beta_2\)      | 0.0162         | 10.67 | <0.0001 |
| \(\ln\left(\frac{J}{100kN/m}\right)\) \(\beta_3=-0.06284\) | 1  | \(\beta_3\)      | 0.0036         | 17.48 | <0.0001 |

### 5.5.3 Verification of the Models

The regression BCR models in equation 5.26 and 5.28 are verified by comparing the results of regression models with ten experimental data which are not used to develop the model. The detailed variables and comparison are presented in Table 5.16. The absolute errors vary from 0.25 ~ 4.22 % and 0.95 ~ 9.96% for linear and nonlinear model, respectively. Both regression models have acceptable prediction accuracy for BCR.
### Tab 3.5.16 Verification of regression models for Kentucky crushed limestone

<table>
<thead>
<tr>
<th>s/B</th>
<th>N</th>
<th>J/100kN/m</th>
<th>BCR (measured)</th>
<th>BCR (linear)</th>
<th>Abs (Err) (%)</th>
<th>BCR (nonlinear)</th>
<th>Abs (Err) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01</td>
<td>3</td>
<td>3.75</td>
<td>1.09</td>
<td>1.08</td>
<td>0.73</td>
<td>1.01</td>
<td>7.89</td>
</tr>
<tr>
<td>0.02</td>
<td>3</td>
<td>3.75</td>
<td>1.13</td>
<td>1.15</td>
<td>2.01</td>
<td>1.14</td>
<td>0.95</td>
</tr>
<tr>
<td>0.03</td>
<td>3</td>
<td>3.75</td>
<td>1.19</td>
<td>1.21</td>
<td>2.00</td>
<td>1.22</td>
<td>2.31</td>
</tr>
<tr>
<td>0.04</td>
<td>3</td>
<td>3.75</td>
<td>1.26</td>
<td>1.28</td>
<td>1.81</td>
<td>1.29</td>
<td>2.33</td>
</tr>
<tr>
<td>0.05</td>
<td>3</td>
<td>3.75</td>
<td>1.31</td>
<td>1.34</td>
<td>2.86</td>
<td>1.34</td>
<td>2.35</td>
</tr>
<tr>
<td>0.06</td>
<td>3</td>
<td>3.75</td>
<td>1.35</td>
<td>1.41</td>
<td>4.22</td>
<td>1.38</td>
<td>2.15</td>
</tr>
<tr>
<td>0.07</td>
<td>3</td>
<td>3.75</td>
<td>1.44</td>
<td>1.47</td>
<td>2.14</td>
<td>1.42</td>
<td>1.65</td>
</tr>
<tr>
<td>0.08</td>
<td>3</td>
<td>3.75</td>
<td>1.52</td>
<td>1.54</td>
<td>1.11</td>
<td>1.45</td>
<td>4.52</td>
</tr>
<tr>
<td>0.09</td>
<td>3</td>
<td>3.75</td>
<td>1.60</td>
<td>1.60</td>
<td>0.25</td>
<td>1.48</td>
<td>7.25</td>
</tr>
<tr>
<td>0.1</td>
<td>3</td>
<td>3.75</td>
<td>1.68</td>
<td>1.67</td>
<td>0.60</td>
<td>1.51</td>
<td>9.96</td>
</tr>
</tbody>
</table>

### 5.6 Summary and Discussions

Based on the statistical analysis on experimental test results of reinforced soil foundation, it can be seen that from statistical point of view the effect of tensile modulus of reinforcement is not significant in the bearing capacity ratio for linear regression model developed for silty clay and nonlinear regression model developed for sand in this study. However, in practice the tensile modulus of reinforcement is an important factor that controls the design of reinforced soil foundation. The experimental study presented in Chapter 4 also has demonstrated this point. Therefore, the models including the tensile modulus, nonlinear regression model for silty clay and linear regression model for sand, should have more practical meaning and are recommended for application in engineering practice. For both linear and nonlinear regression models for sand, the effect of top layer spacing ratio \( u/B \) is not significant.

All regression models are applicable for 152 mm wide square footing and all the geogrid layers were assumed to have enough length to fully mobilize its tensile contribution. In addition, the regression models for silty clay with geogrid are valid for the following conditions:

\[
0.167 \leq u/B \leq 1.333 \\
0.167 \leq h/B \leq 0.667 \\
1 \leq N \leq 5 \\
D_f/B = 0 \\
218.5 \text{ kN/m} \leq J \leq 365 \text{ kN/m}
\]

The regression models for sand with geogrid are valid for the following conditions:

\[
0.167 \leq u/B \leq 1.333 \\
0.167 \leq h/B \leq 0.5
\]
\[ 1 \leq N \leq 4 \]
\[ D_B = 1 \]
\[ 218.5 \text{kN/m} \leq J \leq 800 \text{kN/m} \]

The regression models for crushed limestone are valid for the following conditions:
\[ u/B = h/B = 1/3 \]
\[ 1 \leq N \leq 3 \]
\[ D_B = 0 \]
\[ 267.5 \text{kN/m} \leq J \leq 48480 \text{kN/m} \]

To use above models for estimation of BCR in the field, the scale effect should be considered. Consequently, the discussion on scale effect is followed in next chapter.
CHAPTER 6 FINITE ELEMENT ANALYSIS OF SCALE EFFECT

6.1 Introduction

The model test is easier to operate due to its small size and the corresponding cost is much cheaper as compared to the full-scale prototype test. Therefore, in order to study the behavior of reinforced soil foundation, the small-scale model test is always a good choice. However, this raises a question on the difference in performance between the actual full scale reinforced soil foundations and the model footing tests. This problem has been addressed by geotechnical engineers in many applications and is known as the “scale effect”. To properly address this question, it is necessary to study the scale effect on the results of model footing tests. To implement this objective, Finite Element Analysis (FEA) was conducted on footings with different sizes on reinforced and unreinforced soils using commercially available FEA program ABAQUS.

The footings used in this study were square footings. So, to accurately study their behaviors, three-dimensional modeling of soil foundation is usually needed from theoretical point of view. However, 3-D modeling of footings is time consuming and not practical to run multiple cases. It is common to treat square footings and circular footings with the same area as being equivalent in bearing capacity calculations (Skempton, 1951). Although, there is no theoretical justification for this assumption, it has been successfully used by different researchers (e.g. Lawton, 1995; Osman and Bolton, 2005). A numerical analysis procedure with reasonable approximation is then chosen in this study: the square footings were first converted to equivalent circular footings with the same area; Axisymmetric finite element analysis was then performed on these equivalent circular footings. The diameter of equivalent circular footings can be calculated as:

\[ D = \frac{2B}{\sqrt{\pi}} \]

(6.1)

where \( D \) is the diameter of the equivalent circular footing; \( B \) is the width of the square footing.

6.2 Finite Element Modeling of Reinforced Soil Foundation

6.2.1 Finite Element Mesh

4-node bilinear axisymmetric quadrilateral solid elements (CAX4R) are used to discretize the soil, while 2-node linear axisymmetric membrane elements (MAX1) are used to discretize the
reinforcement. Number of finite element meshes with different degrees of refinement was tried first in order to obtain appropriate mesh for the analysis of square footing on reinforced soil that converges to a unique solution. The finally adopted finite element model for the 457 mm (1.5 ft) wide square footing is illustrated in Figure 6.2.1, which has a radius of 4.0D and total depth of 4.0D (D: equivalent diameter of circular footing). It includes 899 elements for soil and 29 elements for each layer of reinforcement. The size of soil and reinforcement elements is kept the same for all size footings, i.e. the ratio of the number of soil/reinforcement elements to footing size is kept the same for different size footings.

![Figure 6.2.1 Finite element model of the circular footing sitting on reinforced soil](image)

**6.2.2 Loading and Boundary Conditions**

A rigid, perfectly rough footing is assumed in this study. The uniform vertical displacement is applied at nodal points immediately underneath the footing to model the rigid footing condition, while the corresponding horizontal displacement of these points is restrained to zero to simulate the perfect roughness of the soil-footing interface. The loading process is implemented by
applying footing displacement in an increment continuously until the prescribed displacement is reached.

The boundary conditions are specified to simulate those of laboratory tests, i.e. no normal displacement is allowed on the two sides of model; no normal and tangential displacement are allowed at the bottom surface of the model; while the top surface of the model is free of restrain.

6.3 Constitutive Models and Material Parameters

6.3.1 Soil Model

The soil is simulated as an isotropic elasto-perfectly plastic continuum. The yield criterion is described by the extended Drucker-Prager model with a linear form. This model is a simple modification of the Von Mises model by adding the influence of the hydrostatic pressure on the yielding of material.

The linear Drucker-Prager model available in ABAQUS/Standard can match the different yield values in triaxial tension and compression, and it can be written as (Hibbitt, Karlsson & Sorensen, Inc., 2002):

\[ f = t - p \tan \beta - d = 0 \]  
(6.2)

Where, \( t = \frac{1}{2} \left[ q + \frac{1}{K} \left( 1 - \frac{1}{K} \right)^{3/2} \right], p = \frac{1}{3} \text{trace} \sigma, q = \sqrt{\frac{3}{2} S : S}, S \) is the deviatoric stress = \( \sigma - pI \), \( r = \left( \frac{9}{2} S : S \right)^{1/3} \), \( K \) is the flow stress ratio, the ratio of the yield stress in triaxial tension to the yield stress in triaxial compression; \( \beta \) is the slope of the yield surface in the \( p-t \) stress plane, \( d \) is the cohesion intercept of material in the \( p-t \) stress plane. The yield surface of this model in the \( p-t \) stress plane is illustrated in Figure 6.3.1.

Generally, the experimental data are only available for Mohr-Coulomb model with the friction angle and cohesion. To use the linear Drucker-Prager model, it is necessary to match the shear strength parameters of the Mohr-Coulomb model to the Drucker-Prager model. The matching procedure is described in detail in Abaqus/Standard manual and the final relationships for matching these two models can be determined as follows (Hibbitt, Karlsson & Sorensen, Inc., 2002):

\[ \beta = \arctan \left( \frac{6 \sin \phi}{3 - \sin \phi} \right) \]  
(6.3)
Where $\phi$ is the friction angle of soil in the $\tau$-$\sigma$ stress plane, $c$ is the cohesion intercept of soil in the $\tau$-$\sigma$ stress plane.

To keep the yield surface convex, the friction angle of soil must be less than 22°. For materials having a friction angle a little bit higher than 22°, the same method can be used to calculate $\beta$, and $d$. For material having a friction angle significantly higher than 22°, the above equations may provide a poor triaxial match of the Mohr-Coulomb parameters, and $\beta$ can then be approximately set equal to $\phi$ value (Leng, 2000)

6.3.2 Goesynthetics Model

The reinforcement is simulated as a membrane, which transmits in-plane force only and has no bending stiffness. A membrane element maybe the most appropriate element for the simulation of the geosynthetics (Perkins, 2001) and was used by many researchers (e.g. Dondi, 1994; Leng, 2002). The stress-strain behavior of reinforcement is modeled by a linear elastic model.

6.3.3 Soil-Geosynthetics Interface Model

The soil-reinforcement interface properties are one of the basic factors influencing the performance of reinforced soil foundation. The surface-based contact interaction available in Abaqus/Standard is used in this study to model the soil-reinforcement interface. Surface-based
contact simulations between two deformable bodies generally need to define mechanical contact property models in two directions: normal direction and tangential direction.

For silty clay-reinforcement interface, the “hard contact” is assumed in normal direction and no separation of surfaces is allowed once surfaces contact. These contact properties in normal direction can minimize the penetration of slave nodes into the master surface and don’t allow the transfer of tensile stress across the interface. The contact pressures transmitted across the interface usually are shear and normal forces. The relationship between these two force components is described in terms of the Coulomb friction model as shown in Figure 6.3.2. The general form of the Coulomb friction model is expressed as:

$$\tau_{\text{crit}} = \mu \sigma$$  \hspace{1cm} (6.5)

where $\tau_{\text{crit}}$ is the critical shear stress along the interface; $\sigma$ is the normal stress along the interface; $\mu$ is the interface friction angle.

With extension, an additional limit on the allowable elastic slip ($\gamma_{\text{crit}}$) can be included in the Coulomb friction model. The elastic slip is related to the interface shear stress with the relation:

$$\tau = \kappa \gamma$$  \hspace{1cm} (6.6)

where $\tau$ is the shear stress along the interface; $\gamma$ is the elastic slip along the interface; $\kappa$ is the elastic shear stiffness $= \tau_{\text{crit}} / \gamma_{\text{crit}}$. The $\gamma_{\text{crit}}$ describes the interface shear stiffness, and is the limit of the relative shear displacement before the allowable interface shear stress is reached.

![Figure 6.3.2 Basic Coulomb friction model](after Hibbitt, Karlsson & Sorensen, Inc., 2002)
For crushed limestone-reinforcement interface, an assumption of full interlocking between the geogrid and the crushed limestone surrounding it is made, i.e. crushed limestone and geogrid are tied together at interface so that there is no relative motion between them. This type of contact interactions can be achieved by tied contact available in ABAQUS/standard.

6.3.4 Properties of Materials and Interface

Two types of materials, soil and geogrid, are involved in modeling the reinforced soil foundation. Silty clay and crushed limestone, which were used in model tests, are studied here. The reinforcement used in modeling the reinforced silty clay is BX6200 geogrid, while BX1200 geogrid is used in modeling the reinforced crushed limestone.

The parameters used for modeling the reinforced silty clay are summarized in Table 6.1. Table 6.2 presents the parameters used for modeling the reinforced crushed limestone. It is well-known that the soil-reinforcement interface properties are one of the basic factors influencing the performance of reinforced soil foundation. Friction coefficients are selected to make sure that the interface shear strength is the same as that determined from direct shear tests. An elastic slip ($\gamma_{\text{slip}}$) of 1 mm was selected to prescribe the allowable relative displacement along the interface of silty clay and geogrid.
Table 6.1 Material and interface properties for silty clay

<table>
<thead>
<tr>
<th>Materials</th>
<th>Model</th>
<th>Mechanical Properties</th>
<th>Elastic Modulus</th>
<th>Poisson ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silty clay</td>
<td>linear Drucker-Prager</td>
<td>c = 13 kPa</td>
<td>15 MPa</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>φ = 25°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reinforcement (BX6200 geogrid)</td>
<td>Linear Elastic Model</td>
<td>N/A</td>
<td>254 MPa</td>
<td>0.3</td>
</tr>
<tr>
<td>Soil-Reinforcement interface</td>
<td>Coulomb friction model</td>
<td>μ = 0.6</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>γ_slip = 0.001 m</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 6.2 Material and interface properties for Kentucky crushed limestone

<table>
<thead>
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6.4 Numerical Results and Analysis

6.4.1 Finite Element Model Verification

In order to verify the suitability of the adopted models for the soil, geogrids, and geogrid-soil interaction, finite element analyses were first checked against the results from laboratory model tests for a square footing on the reinforced soil. Figures 6.4.1 show the comparison between the finite element analyses and the laboratory model tests for unreinforced and four-layer geogrid reinforced silty clay soil. A comparison of the finite element analysis with the laboratory model tests for unreinforced and three-layer geogrid reinforced crushed limestone is presented in Figure 6.4.2. As can be seen from these figures, the finite element analyses have a reasonable agreement with model test results, although there are some discrepancies between them.
Figure 6.4.1 Verification of the model for silty clay

Figure 6.4.2 Verification of the model for Kentucky crushed limestone
6.4.2 Finite Element Analysis Results

Finite element analysis is first conducted on a 152 mm (0.5 ft) wide square footing, which is in the same size as that used in the laboratory tests. In the following series of finite element analysis, the size of footing is increased to 3 times (B = 457 mm (1.5 ft)), 6 times (B = 914 mm (3.0 ft)), 9 times (B = 1372 mm (4.5 ft)), and 12 times (B = 1829 mm (6.0 ft)) of 152 mm wide square footing. Correspondingly, the size of soil block and reinforcement are increased by the same scale factors. The properties of soil and reinforcement remain the same in all models.

Three series of finite element analyses with different reinforcement layout are conducted to examine the scale effect of the model tests:

Case 1: Keeping the total depth ratio of reinforcement \((d/B)\), the vertical spacing ratio \((h/B)\) of reinforcement layers, and the number of reinforcement layers \((N)\) constant. BX6200 geogrid and BX1200 geogrid are used for all size footings on silty clay and crushed limestone, respectively.

Case 2: Keeping the total depth ratio \((d/B)\) of reinforcement and the vertical spacing of reinforcement layers \((h)\) constant, i.e increasing the number of reinforcement layers \((N)\) by the same scale factors as footing size. BX6200 geogrid and BX1200 geogrid are used for all size footings on silty clay and crushed limestone, respectively.

Case 3: Keeping the total depth ratio of reinforcement \((d/B)\), the vertical spacing ratio \((h/B)\) of reinforcement layers, and the number of reinforcement layers \((N)\) constant; Increasing the tensile modulus of reinforcement \((J)\) by the same scale factors as footing size. BX6200 geogrid and BX1200 geogrid are used for 914 mm (3.0 ft) wide square footing on silty clay and crushed limestone, respectively; while the tensile modulus of reinforcement \((J)\) is increased by the same scale factors as footing size for 1372 mm (4.5 ft) and 1829 mm (6.0 ft) wide square footings.

The settlement of footing in all cases is expressed in the nondimensional form of \(s/B\). The corresponding load settlement curves for silty clay with and without reinforcement are plotted in \(q \sim s/B\) plane as shown in Figure 6.4.3 for case one, Figure 6.4.4 for case two, and Figure 6.4.5 for case three. Figures 6.4.6, 6.4.7, and 6.4.8 depict the load settlement curves of crushed limestone with and without reinforcement for case one, case two and case three, respectively.

6.4.3 Analysis of Numerical Results

It can be seen from Figures 6.4.3 through 6.4.8 that the load-settlement curves of unreinforced soil for different size footings follow the same shape. This result indicates that the unreinforced
Figure 6.4.3 Pressure-settlement curves of silty clay for case one study

Figure 6.4.4 Pressure-settlement curves of silty clay for case two study
Figure 6.4.5 Pressure-settlement curves of silty clay for case three study

Figure 6.4.6 Pressure-settlement curves of Kentucky crushed limestone for case one study
Figure 6.4.7 pressure-settlement curves of Kentucky crushed limestone for case two study

Figure 6.4.8 pressure-settlement curves of Kentucky crushed limestone for case three study
soil (silty clay and crushed limestone) foundation has no scale effect in this study, if the settlement is expressed in a nondimensional relative settlement of $s/B$. This numerical result is in agreement with static loading test results of Ismael (1985), Briaud and Gibbens (1994), and Fellenius and Altacee (1994).

Reinforced ratio ($R_r$) is introduced to assist the analysis of scale effect on reinforced soil foundation. The reinforced ratio ($R_r$) is defined as:

$$R_r = \frac{E_R A_R}{E_s A_s}$$  \hspace{1cm} (6.7)

where $E_R$ is the elastic modulus of the reinforcement $= J/t_R$; $J$ is the tensile modulus of reinforcement; $A_R$ is the area of reinforcement per unit width $= N t_R \times l$; $t_R$ is the thickness of the reinforcement; $N$ is the number of reinforcement layers; $E_s$ is the modulus of elasticity of soil; $A_s$ is the area of reinforced soil per unit width $= d \times l$; $d$ is the total depth of reinforcement $= u + (N-1)h$.

In practice, the top layer spacing $u$ is equal to vertical spacing $h$ in most cases. By substituting $u$ by $h$, the reinforced ratio ($R_r$) in the reinforced zone can be written as

$$R_r = \frac{J}{E_s d} = \frac{J}{E_s h}$$  \hspace{1cm} (6.8)

It can be seen from equation (6.8) that the reinforced ratio is proportional to the tensile modulus of reinforcement and inversely proportional to the vertical spacing of reinforcement ($h$) if the same soil is used.

For case one, the total depth ratio of reinforcement ($d/B$) is the same for all size footings and the vertical spacing of reinforcement ($h$) increases with increasing the footing size. BX6200 geogrid and BX1200 geogrid are used for all size footings on silty clay and crushed limestone, respectively. This suggests the reinforced ratio ($R_r$) decreases with the increase of footing size. Figure 6.4.3 shows that the bearing capacity of reinforced silty clay at the same settlement ratio ($s/B$) for case one decreases with increasing the footing size. The variations of BCRs obtained at settlement ratios of $s/B=5\%, 8\%$, and $10\%$ for different footing size ($B$) and reinforced ratio ($R_r$) are shown in Figure 6.4.9. It can be seen from this figure that the BCRs decrease with increasing footing size ($B$) and decreasing reinforced ratio ($R_r$), and appear to become almost constant after $B = 1372$ mm (4.5 ft) and $R_r = 0.047$. The similar behavior was also obtained for the finite element analysis of crushed limestone as can be seen in Figures 6.4.6 and 6.4.10.
For case two, both the total depth ratio of reinforcement \((d/B)\) and the vertical spacing of reinforcement \((h)\) are kept constant. BX6200 geogrid and BX1200 geogrid are used for all size footings on silty clay and crushed limestone, respectively. The reinforced ratio \((R_r)\) of the reinforced zone is the same for all size footings. The results shown in Figures 6.4.4 and 6.4.7 indicate that load-settlement curves of reinforced soil (silty clay and crushed limestone) for different size footing are similar for case two. The difference in bearing capacity of reinforced soil at the same settlement ratio \((s/B)\) is no more than 3.5%. This result suggests that the load-settlement response of reinforced soil is not sensitive to the scale effect for case two if the settlement is expressed in a nondimensional relative settlement of \(s/B\).

For case three, the total depth ratio of reinforcement \((d/B)\), the vertical spacing ratio \((h/B)\) of reinforcement layers, and the number of reinforcement layers \((N)\) are kept constant. BX6200 geogrid and BX1200 geogrid are used for 914 mm (3 ft) wide square footing on silty clay and crushed limestone, respectively; while the tensile modulus of reinforcement \((J)\) is increased by the same scale factors as footing size for 1372 mm (4.5 ft) and 1829 mm (6 ft) wide square footings. The reinforced ratio \((R_r)\) of the reinforced zone is kept constant for all size footings. Figures 6.4.5 and 6.4.8 show that load-settlement curves of reinforced soil (silty clay and crushed limestone) for different size footing are similar. The difference in bearing capacity of reinforced soil at the same settlement ratio \((s/B)\) is less than 1%. This result suggests that the scale effect has negligible effect on the load-settlement response of reinforced soil for case three if the settlement is expressed in a nondimensional relative settlement of \(s/B\).

These results indicate that the scale effect is mainly related to the reinforced ratio \((R_r)\) of the reinforced zone if the total depth ratio of reinforcement \((d/B)\) keeps constant.

### 6.5 Summary and Discussions

Based on the finite element analysis of square footing of different sizes on reinforced soil, it can be seen that the load-settlement curves of unreinforced soil are the same if the settlement is expressed in a nondimensional settlement ratio of \(s/B\). The bearing capacity of reinforced soil decreases with increasing footing size if the total depth ratio of reinforcement \((d/B)\), the vertical spacing ratio \((h/B)\) of reinforcement layers, and hence the number of reinforcement layers \((N)\) are kept constant. However, the difference in the bearing capacity is negligible if the total depth ratio \((d/B)\) of reinforcement and the reinforced ratio \((R_r)\) remain constant for all sizes of footing.
The FEM analysis in this study indicates that the scale effect is mainly related to the reinforced ratio ($R_r$) of the reinforced zone. In laboratory model tests, if we can keep the total

![Reinforced Ratio ($R_r$) vs. Width of Footing (B, mm) for Reinforced Silty Clay](image)

Figure 6.4.9 BCR vs. width of footing ($B$) and reinforced ratio ($R_r$) for reinforced silty clay

![Reinforced Ratio ($R_r$) vs. Width of Footing (B, mm) for Reinforced Crushed Limestone](image)

Figure 6.4.10 BCR vs. width of footing ($B$) and reinforced ratio ($R_r$) for reinforced crushed limestone
depth ratio of reinforcement \((d/B)\) and the reinforced ratio \((R_r)\) the same as those used in actual full scale reinforced soil foundations, the model test results can be extrapolated to the performance of actual full scale reinforced soil foundations. In general, to keep the reinforced ratio \((R_r)\) in laboratory tests the same as that in actual full scale reinforced soil foundations, we can keep the vertical spacing ratio of reinforcement \((h/B)\) and the ratio of reinforcement tensile modulus to the footing size in laboratory tests the same as those in actual full scale reinforced soil foundations.

Future experimental study is recommended to substantiate and improve on these findings.

It should be mentioned here that the geogrid is simulated as membrane in this study. This means that the geogrid is treated as a continuous sheet of reinforcement, which does not allow the flow of soil particles through the fabric. But the real geogrid is in the form of grid structure with aperture geometry, which allows soil particles flowing from one side of the geogrid to the other. This difference in soil-reinforcement interaction may cause difference in behavior of reinforced soil foundation and needs to be further investigated.
CHAPTER 7 DESIGN OF REINFORCED SOIL FOUNDATION

7.1 Introduction

The benefits of using reinforcements to increase the bearing capacity and reduce the settlement of soil foundation have been widely recognized. The experimental results of this study clearly substantiated this point. It is therefore necessary to develop a new stability analysis technique for reinforced soil foundation to account for this positive effect from reinforcement. During the past thirty years, many hypotheses have been postulated to describe the reinforcing mechanism and determine the possible failure modes of RSF, but as compared to RSF’s engineering application, the development of its design method and theory is relatively slow. The Stability analysis of reinforced soil foundation in this chapter that includes the effect of reinforcement is an attempt to examine existing methods and/or develop reasonable design methods for different soil types.

7.2 Stability Analysis of Reinforced Soil Foundation

Based on the literature review and experimental test results of present study, five different failure modes can be identified: failure above reinforcement (Binquet and Lee, 1975), failure between reinforcement (Wayne et al., 1998), failure like footing on a two layer soil system (strong soil layer over weak soil layer) (Wayne et al., 1998), failure in reinforced zone, and partial punching-shear failure in reinforced zone. The first two failure modes can be avoided by keeping the top layer spacing \(u\) and vertical spacing between reinforcement layers \(h\) small enough. Based on the experimental results of the present study, the top layer spacing \(u\) and vertical spacing \(h\) are recommended to be less than 0.5\(B\) to prevent these two failure modes from occurring. This requirement should not be difficult to fulfill in engineering practice, therefore, the discussion here is only focused on the latter three failure modes. As mentioned in literature review, the reinforcement can restrain lateral deformation or potential tensile strain of the soil (confinement effect) and the deformed reinforcement can also develop an upward force (tension membrane effect). All these effects lead to an increase in bearing capacity. So, the contribution of reinforcements to bearing capacity needs to be included in the bearing capacity calculation.

7.2.1 Failure like Footings on Two Layer Soil System (Strong Soil Layer Over Weak Soil Layer)

If the strength of the reinforced zone is much larger than that of the underlying unreinforced zone and the reinforcement depth ratio \(d/B\) is relatively small, a punching shear failure will occur in
the reinforced zone followed by a general shear failure in the unreinforced zone as shown in Figure 7.2.1. This failure mode was first suggested by Meyerhof and Hanna (1978) for stronger soil underlying by weaker soil. With some modification, Meyerhof and Hanna’s solution can be used to calculate the bearing capacity of reinforced soil foundation (Wayne et al, 1998).

The determination of the exact shape of reinforcement at the ultimate load is not easy. Two different reinforcing mechanisms are therefore discussed here: horizontal confinement effect of reinforcement and vertical reinforcement tension along the punching failure surfaces $aa'$ and $bb'$ as shown in Figure 7.2.1(tension membrane effect). The actual reinforcing effect should be the combination of these two reinforcing mechanisms.

![Figure 7.2.1 Failure like footing on a two layer soil system](image)

### 7.2.1.1. Horizontal Confinement Effect of Reinforcement

Considering the strip footing case as shown in Figure 7.2.2, The forces on the vertical punching failure surface in the upper soil layer include the total passive earth pressure $P_p$, inclined at an average angle $\delta$, and adhesive force $C_a = c_a d$ acting upwards. With the inclusion of reinforcement, there is an upward shear force induced by the tension of reinforcement on the vertical failure surface.

The ultimate bearing capacity can be given as follow for strip footing on a reinforced soil foundation with horizontal reinforcement:

$$q_{u(R)} = q_b + \frac{2(C_a + P_p \sin \delta)}{B} - \gamma_d + \Delta q_f$$  \hspace{1cm} (7.1)

where $q_{u(R)}$ is the ultimate bearing capacity of reinforced soil foundation; $q_b$ is the ultimate bearing capacity of the underlying unreinforced soil; $C_a$ is the adhesive force along two sides $= c_a d$; $c_a$ is the unit adhesion of soil along two sides; $d$ is the thickness of reinforced zone; $P_p$ is the...
Figure 7.2.2 Failure like footing on a two layer soil system with horizontal reinforcement

\[
\Delta q_T = \sum_{i=1}^{N} T_i \sin \delta
\]

\[
B
\]

\[
q_n = c_c N_c + q N_q + 0.5 \gamma_b B N_r
\]

(7.2)

Where \(c_c\) is the cohesion of soil in unreinforced zone; \(q\) is the surcharge, \(= \gamma \cdot (D_f + d)\); \(\gamma_b\) is the unit weight of soil in the unreinforced zone; \(N_c, N_q,\) and \(N_r\) are the bearing capacity factors, which are dependent on the friction angle of soil in the unreinforced zone, \(\phi_b\) and take the following form.

\[
N_q = \tan^2 \left( \frac{\pi}{4} + \frac{\phi}{2} \right) e^{\frac{\gamma \tan \phi}{2}}
\]

\[
N_c = \frac{N_q - 1}{\tan \phi}
\]

\[
N_r = 2(N_q + 1) \tan \phi
\]

(7.3)

(7.4)

where \(D_f\) is the embedment depth of the footing; \(K_{pH}\) is the horizontal component of the passive earth pressure coefficient.

\[
\Delta q_T = \frac{2 \sum_{i=1}^{N} T_i \tan \delta}{B}
\]

(7.5)

where \(T_i\) is the tensile force in the \(i^{th}\) layer of reinforcement; \(N\) is the number of reinforcement layers.
Substituting Equations (7.4) and (7.5) into Equation (7.1)

\[ q_{w(R)} = q_b + \frac{2c_a d}{B} + \gamma_i d_2 \left( 1 + \frac{2D_f}{d} \right) K_{pil} \tan \delta + \frac{2 \sum_{i=1}^{N} T_i \tan \delta}{B} - \gamma_i d \]  

(7.6)

Let

\[ K_{pil} \tan \delta = K_s \tan \phi_i \]  

(7.7)

then,

\[ q_{w(R)} = q_b + \frac{2c_a d}{B} + \gamma_i d_2 \left( 1 + \frac{2D_f}{d} \right) K_s \tan \phi_i + \frac{2 \sum_{i=1}^{N} T_i \tan \delta}{B} - \gamma_i d \]  

(7.8)

where \( K_s \) is the punching shear coefficient, which depends on the friction angle of soil in the reinforced zone and the ultimate bearing capacity of soil in both the reinforced zone and the underlying unreinforced zone; \( \phi_i \) is the friction angle of soil in the reinforced zone.

The determination of \( \delta \) and \( c_a \) is not necessarily simple. It varies along the depth of the vertical punching failure surface (Meyerhof and Hanna, 1978). An average value of \( \delta \) and \( c_a \) is generally selected for analysis. These values are dependent on the embedment depth of footing, the thickness of the upper layer, and the relative strengths of the upper and lower layers of soil (Valsangkar and Meyerhof, 1979). For preliminary bearing capacity estimates, an average value of \( \delta = \phi/2 \) was suggested by Meyerhof and Hanna (1978) for footings under vertical load on layered sand or on a sand layer overlying a clay layer. The punching shear coefficient, \( K_s \), can then be determined by using the passive earth pressure coefficient charts proposed by Caquot and Kerisel (1949). On the other hand, Meyerhof and Hanna (1978) also suggested that an average value of \( c_a = 0.75 c_t \) may be used for footings on layered clay with vertical load; \( c_t \) is the cohesion of soil in reinforced zone. A better estimate of the punching shear coefficient, \( K_s \), and unit adhesion of soil, \( c_a \), can be obtained from Figure 7.2.3 and 7.2.4, respectively.

Similar to Equation (7.8), the ultimate bearing capacity formula for square footings on a reinforced soil foundation can be given as:

\[ q_{w(R)} = q_b + \frac{4c_a s_a d}{B} + 2\gamma_i d_2 \left( 1 + \frac{2D_f}{d} \right) K_s s_{s_a} \tan \phi_i + \frac{4 \sum_{i=1}^{N} T_i s_{s_T} \tan \delta}{B} - \gamma_i d \]  

(7.9)

Where \( s_{s_a}, s_s \) and \( s_{s_T} \) are shape factors for punching shear resistance.
$q_p = 1.3c
\gamma_c N_v + q N_q + 0.4\gamma_b BN_v$  \hspace{1cm} (7.10)

Valsangkar and Meyerhof (1979) showed that for circular footing a shape factor may be taken as unity for the punching failure of footings in both sand and clay layers. For square footings, this approximation can still be applied.

This failure mechanism most likely controls the performance of reinforced clayey soil.

**7.2.1.2 Vertical Reinforcement Tension along the Punching Failure Surfaces aa’ and bb’**

For the reinforcement turning vertically along the punching failure surfaces at the ultimate load as shown in Figure 7.2.5, the solution proposed by Wayne et al (1998) can be used to calculate the ultimate bearing capacity of reinforced soil foundation.

![Figure 7.2.3 Coefficients of punching shear resistance under vertical load (after Meyerhof and Hanna, 1978)](image-url)
Figure 7.2.4 Variation of punching shearing parameter $c_a$ under vertical load (after Meyerhof and Hanna, 1978)

For strip footing on a reinforced soil foundation:

$$q_{u(s)} = q_b + \frac{2c_a d}{B} + \gamma_i d^2 \left(1 + \frac{2D_f}{d}\right) K_s \tan \phi_i + \frac{2}{B} \sum_{i=1}^{N} T_i - \gamma_i d$$  \hspace{1cm} (7.11)

For square footing on a reinforced soil foundation:
This kind of failure mechanism most likely occurs in the reinforced clayey soil with large deformation.

7.2.2 Failure in Reinforced Zone

If the strength of the reinforced zone is slightly larger than that of the underlying unreinforced zone or if the reinforcement depth ratio \((d/B)\) is relatively large, the failure will occur in the reinforced zone as shown in Figure 7.2.6.

![Figure 7.2.6 Failure in reinforced zone](image)

Again, two different reinforcing mechanisms are discussed here: horizontal reinforcement (confinement effect) and reinforcement along the \(ac\) and \(bc\) faces of the soil wedge \(abc\) as shown in Figure 7.2.6 (tension membrane effect).

7.2.2.1. Horizontal Confinement Effect of Reinforcement

The classic bearing capacity formula, also known as the “triple \(N\) formula”, includes three items which account for the contributions of surcharge \(q\), cohesion \(c\), and weight of soil \(\gamma\). Superposition is applied to add these three items together. The general form of the bearing capacity formula for strip footing is given by:

\[
q_{u(R)} = q_u + \frac{4c_h d}{B} + 2\gamma d^2 \left( 1 + \frac{2D_f}{d} \right) K_i \tan \phi_i + \frac{4 \sum T_i}{B} - \gamma d
\]  

(7.12)

where \(q_u\) is the ultimate bearing capacity of unreinforced soil foundation; \(c\) is the cohesion of soil; \(q\) is the surcharge load; \(\gamma\) is the weight of soil; \(B\) is the width of footing; and \(N_c\), \(N_q\), and \(N_\gamma\) are bearing capacity factors, which are dependent on the friction angle of soil \(\phi\).
To include the contribution of reinforcement, the method of superposition can be used and an additional item $\Delta q_T$ is then added in terms of tensile force $T$. The bearing capacity formula now takes the following form:

$$q_{u(r)} = cN_c + qN_q + 0.5\gamma BN_{\gamma} + \Delta q_T$$  \hspace{1cm} (7.14)

where $q_{u(r)}$ is the ultimate bearing capacity of reinforced soil foundation; $\Delta q_T$ is the increased bearing capacity due to the tensile force of the reinforcement.

First, consider the strip footing case and the single layer of reinforcement. The failure surface in soil for the strip footing at the ultimate load is shown Figure 7.2.7. The reinforcement is located at a depth of $u$.

![Figure 7.2.7 Failure in reinforced zone with horizontal reinforcement](image)

Considering the soil wedge $abc$, the passive force $P_p$ acting on the faces $ac$ and $bc$ includes four components as shown in Figure 7.2.8 and can be written as:

$$P_p = P_{pc} + P_{pq} + P_{py} + P_{pT}$$  \hspace{1cm} (7.15)

Where $P_{pc}$, $P_{pq}$, $P_{py}$, and $P_{pT}$ are the passive force due to surcharge $q$, cohesion $c$, weight of soil $\gamma$, and the tensile force of reinforcement $T$.

![Figure 7.2.8 Passive forces on the triangular wedge abc](image)
Derivation of \( P_{pc} \), \( P_{pq} \), and \( P_{pT} \) can be found in many foundation engineering books (e.g. Das, 1999). Therefore, the discussion will only focus on the derivation of \( P_{pT} \) here.

Considering the free body diagram of the soil wedge \( bcdg \) shown in Figure 7.2.9, the forces per unit length of the wedge \( bcdg \) due to the tensile force of reinforcement \( T \) include \( P_{pT} \) tensile force of reinforcement, \( T_L \) and \( T_R \), and the resisting force along the log spiral \( cd \), \( F \) as shown in Figure 7.2.9.

![Figure 7.2.9 Free body diagram of the soil wedge bcdg](image)

The log spiral is described by the equation \( r = r_0 e^{\theta \tan \phi} \). This means that the radial line at any point makes an angle \( \phi \) with the normal direction of the log spiral. The resisting force \( F \) also makes an angle \( \phi \) with the normal direction of the log spiral. Taking the moment about center of the log spiral, \( b \)

\[
P_{pT} \cos \phi \frac{B/4}{\cos(\pi/4 + \phi/2)} = (T_L - T_R) \mu \times 1 \tag{7.16}
\]

\[
P_{pT} = \frac{4(T_L - T_R) \mu \cos(\pi/4 + \phi/2) \times 1}{B \cos \phi} \tag{7.17}
\]

Considering the equilibrium of the soil wedge \( abc \) shown in Figure 7.2.10.

\[
\Delta q_T B \times 1 = 2P_{pT} \sin(\pi/4 + \phi/2) \tag{7.18}
\]

\[
\Delta q_T = \frac{4(T_L - T_R) \mu}{B^2} \tag{7.19}
\]
The distance that the tensile force $T_R$ is applied from the center of footing $x_{r_s}$ is the function of the friction angle of soil $\phi$. The variation of $x_{r_s} / B$ with the soil friction angle $\phi$ is given in Figure 7.2.11. From this figure, it can be seen that the distance that the tensile force $T_R$ is applied from the center of footing is greater than $2B$ when soil friction angle $\phi$ is greater than $25^\circ$. Based on the measured strain distribution along the reinforcement in this study, the tensile force in the reinforcement at this distance is negligible, so the tensile force $T_R$ can be taken as zero and equation (7.19) can then be simplified as:

$$
\Delta q_T = \frac{4T_i u}{B^2} = \frac{4T u}{B^2}
$$

(7.20)

For two or more layers of reinforcement, the increased bearing capacity $\Delta q_T$ can be easily shown to be:

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

(7.21)

where $T_i$ is the tensile force in the $i^{th}$ reinforcement layer; $N$ is the number of reinforcement layers; $u$ is the top layer spacing of reinforcement; $h$ is the vertical spacing between reinforcement layers. It should be noted that all reinforcement layers must be placed above the failure zone, i.e. above the point $f$ as shown in Figure 7.2.7, to contribute to improving the performance of the soil foundation. The ultimate bearing capacity of the strip footing on a soil with horizontal reinforcement can now be given as:

$$
q_{u(R)} = cN_c + qN_q + 0.5\gamma BN_f + \sum_{i=1}^{N} \frac{4T_i [u + (i-1)h]}{B^2}
$$

(7.22)
For square footings, the increased bearing capacity $\Delta q_T$ can be simply calculated as:

$$\Delta q_T = \sum_{i=1}^{N} \frac{12T_i[u + (i-1)h]r_T}{B^2}$$

where:

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

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

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

The ultimate bearing capacity of the square footing on a reinforced soil foundation with horizontal reinforcement can now be given as:

$$q_{u(R)} = 1.3cN_c + qN_q + 0.4\gamma B N_y + \sum_{i=1}^{N} \frac{12T_i[u + (i-1)h]r_T}{B^2}$$

This type of failure mechanism most likely controls the performance of the reinforced sandy soil.
7.2.2.2. Reinforcement Tension along the Faces \( ab \) and \( bc \) of Soil Wedge \( abc \)

The strip footing with single layer of reinforcement is first discussed here. The failure surface in soil for the strip footing and the shape of reinforcement at the ultimate load are of the form as shown in Figure 7.2.12.

![Figure 7.2.12 Failure in reinforced zone with reinforcement tension along the faces \( ac \) and \( bc \)](image)

The increased bearing capacity due to the tensile force of the single layer of reinforcement \( \Delta q_T \) can be evaluated by considering the equilibrium of the soil wedge \( abc \) as shown in Figure 7.2.13.

\[
\Delta q_T \times B = 2T \sin \left( \frac{\pi}{4} + \phi/2 \right) \tag{7.27}
\]

\[
\Delta q_T = \frac{2T \sin \left( \frac{\pi}{4} + \phi/2 \right)}{B} \tag{7.28}
\]

![Figure 7.2.13 Free body diagram of the soil wedge \( abc \)](image)

For two or more layers of reinforcement, the increased bearing capacity \( \Delta q_T \) can be easily shown to be:
\[
\Delta q_T = \sum_{i=1}^{N} \frac{2T_i \sin\left(\frac{\pi}{4} + \frac{\phi}{2}\right)}{B} \tag{7.29}
\]

It should be noted that all reinforcement layers must be placed above the triangle wedge \(abc\), i.e., above the point \(c\) as shown in Figure 7.2.12, to contribute to improving the performance of the soil foundation for this case. The ultimate bearing capacity of strip footing on a reinforced soil foundation with the inclusion of reinforcement can now be given as:

\[
q_{u(R)} = cN_c + qN_q + 0.5\gamma BN_T + \sum_{i=1}^{N} \frac{2T_i \sin\left(\frac{\pi}{4} + \frac{\phi}{2}\right)}{B} \tag{7.30}
\]

For square footings, the increased bearing capacity \(\Delta q_T\) can be calculated as:

\[
\Delta q_T = \sum_{i=1}^{N} \frac{4T_i \sin\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \left[ B - 2[u + (i-1)h] \tan\left(\frac{\pi}{4} - \frac{\phi}{2}\right) \right]}{B^2} \tag{7.31}
\]

The ultimate bearing capacity of the square footing on a reinforced soil foundation can now be given as:

\[
q_{u(R)} = 1.3cN_c + qN_q + 0.4\gamma BN_T + \sum_{i=1}^{N} \frac{4T_i \sin\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \left[ B - 2[u + (i-1)h] \tan\left(\frac{\pi}{4} - \frac{\phi}{2}\right) \right]}{B^2} \tag{7.32}
\]

This kind of failure mechanism most likely occurs in the reinforced soil with large particle size.

### 7.2.3 Partial Punching-Shear Failure

If the strength of the reinforced zone is moderately larger than that of the underlying unreinforced zone (i.e., between aforementioned two cases), a punching shear failure may occur partially in the reinforced zone followed by a general shear failure as shown in Figure 7.2.14.

Again, two different reinforcing mechanisms are discussed here: horizontal reinforcement (confinement effect) and reinforcement along faces \(aa'c\) and \(bb'c\) of the soil wedge \(abb'ca'\) (tension membrane effect) as shown in Figure 7.2.14.

#### 7.2.3.1 Horizontal Confinement Effect of Reinforcement

For strip footing with horizontal confinement effect of reinforcement as shown in Figure 7.2.15, the ultimate bearing capacity of reinforced soil foundation can be given as:

\[
q_{u(R)} = cN_c + qN_q + 0.5\gamma BN_T + \sum_{i=1}^{N} \frac{2T_i \sin\left(\frac{\pi}{4} + \frac{\phi}{2}\right)}{B} \tag{7.29}
\]
Figure 7.2.14 Partial punching shear failure

\[ q_{u(s)} = q_b + \frac{2(C_a + P_p \sin \delta)}{B} - \gamma_t D_p + \Delta q_T \]  

(7.33)

Where \( D_p \) is the depth of the punching shear failure in the reinforced zone. The increased bearing capacity, \( \Delta q_T \), can be calculated by combining equations (7.5) and (7.21) as follows:

\[ \Delta q_T = 2 \sum_{i=1}^{N_p} \frac{T_i B \tan \delta + \sum_{i=N_p+1}^{N} \frac{4T_i [u + (i-1)h - D_p]}{B^2}}{N} \]  

(7.34)

where \( N_p \) is the number of reinforcement layers located in the punching shear failure zone;

Substituting Equations (7.4), (7.7) and (7.34) into Equation (7.33) and replacing \( d \) in Equation (7.4) with \( D_p \).

\[ q_{u(s)} = q_b + \frac{2C_a D_p}{B} + \gamma_t D_p \left(1 + \frac{2D_p}{D_p} \right) \frac{K_s \tan \phi_i}{B} - \gamma_t D_p + 2 \sum_{i=1}^{N_p} \frac{T_i B \tan \delta + \sum_{i=N_p+1}^{N} \frac{4T_i [u + (i-1)h - D_p]}{B^2}}{N} \]  

(7.35)

Figure 7.2.15 Partial punching shear failure with horizontal reinforcement

Similarly, the ultimate bearing capacity of the square footing on a reinforced soil foundation with horizontal reinforcement can be evaluated as:
This first failure mechanism most likely occurs in the reinforced clayey soil with low reinforced ratio \( (R_r) \).

### 7.2.3.2 Reinforcement Tension along the Faces aa’c and bb’c of Soil Wedge abb’ca’

For strip footing with reinforcement along the shear failure surface aa’c and bb’c as shown in Figure 7.2.16, the increased bearing capacity \( \Delta q_T \) can be shown to be:

\[
\Delta q_T = \sum_{i=1}^{N_r} \frac{2T_i \sin \left( \frac{\pi}{4} + \frac{\phi}{2} \right)}{B} \tag{7.37}
\]

Substituting Equations (7.4), (7.7) and (7.35) into Equation (7.33), the ultimate bearing capacity of the strip footing on a reinforced soil foundation can be written as:

\[
q_{u(R)} = q_b + \frac{4c_a D_p}{B} + 2\gamma_i d^2 \left( 1 + \frac{2D_f}{D_p} \right) \frac{K_i \tan \phi_i}{B} - \gamma_i D_p + 2 \sum_{i=1}^{N_r} \frac{T_i}{B} + 2 \sum_{i=N_r+1}^{N} \frac{2T_i \sin \left( \frac{\pi}{4} + \frac{\phi}{2} \right)}{B} \tag{7.38}
\]

Similarly, the ultimate bearing capacity of the square footing on a reinforced soil foundation can be given as:

\[
q_{u(R)} = q_b + \frac{4c_a D_p}{B} + 2\gamma_i d^2 \left( 1 + \frac{2D_f}{D_p} \right) \frac{K_i \tan \phi_i}{B} - \gamma_i D_p + \Delta q_T \tag{7.39}
\]
where

\[ \Delta q_i = 4 \sum_{i=1}^{N_i} \frac{T_i}{B} \tan \delta + \sum_{i=N_r+1}^{N} \frac{4T_i \sin \left( \frac{\pi}{4} + \frac{\phi}{2} \right) B - 2u \tan \left( \frac{\pi}{4} - \frac{\phi}{2} \right)}{B^2} \]  

(7.40)

This first failure mechanism most likely controls the performance of the reinforced soil with large particle size such as crushed limestone.

### 7.3 Tensile Force in Reinforcement

In the experimental work of this study, the strain distribution along the reinforcement was measured by strain gauges. The tensile force developed along the reinforcement can be evaluated based on this measured strain. In real world design, the mobilized tensile force in reinforcement is unknown and has to be estimated. The following analysis is made to obtain a reasonable estimation on the tensile force along the reinforcement for foundation on reinforced sand.

Experimental test results of this study showed that the strain developed along the reinforcement is directly related to the settlement. At the same footing settlement, the vertical settlement distribution in reinforced soil is assumed to be the same as that in unreinforced soil. At a certain settlement level, the shape of deformed reinforcement should be compatible with vertical settlement distribution.

In the absence of a rigorous solution for the vertical settlement distribution at a certain depth, it may be assumed that the shape of reinforcement at that certain depth is of the form as shown in Figure 7.3.1 for sand. The reinforcement beneath the footing is assumed to move downward uniformly (lines $bc$). The reinforcement located outside of a certain boundary (lines $a-a'$ and $d-d'$) is considered to have negligible strain. Based on the measured strain distribution along the reinforcement in the present study, the slope of the boundary lines $a-a'$ and $d-d'$ can be taken as about 2:1 (vertical : horizontal), which is the same as the simplified 2:1 stress distribution lines.

Since the distribution of vertical settlement is now known, next step is to determine the amount of settlement at a certain depth beneath the footing ($S_0$).

Schmertmann (1970) and Schmertmann et al. (1978) conducted extensive research on the prediction of settlement over sand. Based on sand model tests and finite element method study, Schmertmann and Harman (1978) suggested a practical distribution of vertical strain along the
depth below the footings in terms of strain influence factor $I_{\varepsilon}$ as shown in Figure 7.3.2. The peak value of the strain influence factor $I_{\varepsilon p}$ is evaluated by the following equation:

$$I_{\varepsilon p} = 0.5 + 0.1 \sqrt{\frac{q - \gamma D_f}{\sigma_{\varepsilon p}'}} \quad (7.41)$$

$$\sigma_{\varepsilon p}' = \gamma(D_f + B/2) \quad (\text{square footing}) \quad (7.42)$$

$$\sigma_{\varepsilon p}' = \gamma(D_f + B) \quad (\text{strip footing}) \quad (7.43)$$

where $q$ is bearing pressure of footing; $\gamma$ is the unit weight of sand; $D_f$ is the embedment depth of footing; $B$ is the width of footing.

Using this simplified strain influence factor distribution diagram, the elastic settlement $S_e$ in sand can then be calculated as:

$$S_e = C_1 C_2 C_3 (q - \gamma D_f) \sum \frac{I_{\varepsilon} \Delta z}{E_s} \quad (7.44)$$

$$C_1 = 1 - 0.5 \frac{\gamma D_f}{q - \gamma D_f} \quad (7.45)$$

$$C_2 = 1 + 0.2 \log \left( \frac{t}{0.1} \right) \quad (7.46)$$

$$C_3 = 1.03 - 0.03 L/B \geq 0.73 \quad (7.47)$$

where $C_1$ is a correction factor for the depth of embedment; $C_2$ is a correction factor for secondary creep in sand; $C_3$ is a correction factor for the footing shape; $E_s$ is the elastic modulus of sand; $t$ is the time since application of load (yr) ($t \geq 0.1 \text{yr}$); $L$ is the length of footing; $B$ is the width of the footing.

Based on the above assumptions and analysis, the average strain in reinforcement at a certain footing settlement can now be calculated as:
Where $S_e$ is the settlement at a depth of $z$ beneath the center of footing; $z$ is the depth of reinforcement $= u+(i-1)h$. The average tensile force, $T_{avg}$, developed in reinforcement can then be evaluated by the following equation:

$$T_{avg} = J\varepsilon_{avg}$$  \hspace{1cm} (7.51)

Where $J$ is the tensile modulus of reinforcement.

The measured strain (Chapter 3) showed that the strain distribution along the reinforcement is not uniform. The tensile strain is the largest at the point beneath the center of the footing and decreases with the distance away from the center of footing. A triangle distribution as shown in Figure 7.3.3 is assumed here to approximately describe the real strain distribution along the reinforcement. The maximum strain in this triangle distribution can be calculated as:
\[ \varepsilon_{\text{max}} = 2 \varepsilon_{\text{avg}} \]  

(7.52)

Figure 7.3.3 Simplified strain distribution along the reinforcement

For crushed limestone, due to its relatively larger particle size, the reinforcement is believed to move together with the soil wedge abc or abb’ca’ as shown in Figures 7.2.12 and 7.2.16. It may then be assumed that the shape of reinforcement at the certain depth is of the form as shown in Figure 7.3.4. The reinforcement in the soil wedge beneath the footing is assumed to move down uniformly (lines cd or c’d’). The reinforcement outside of the wedge is taken as horizontal. The strain of the reinforcement beyond a certain boundary (lines a-a’-a’’ and f-f’-f’’) is considered to be insignificant. Without measuring strain data, the boundary lines a-a’-a’’ and f-f’-f’’ are assumed to have a slope of 2 which is the same as that for sand. The amount of settlement at a certain depth beneath the footing \( S_s \) can be approximately evaluated by Schmertmann’s method.

\[ \varepsilon_{\text{avg}} = \frac{L_{ab} + L_{bc} + L_{cd} + L_{de} + L_{ef} - L_{af}}{L_{af}} \]  

(7.53)

Figure 7.3.4 Simplified shape of reinforcement in crushed limestone

The average strain in reinforcement at a certain footing settlement can now be calculated as:
\[ L_{ab} = L_{ef} = z/2 \]  
(7.54)

\[ L_{bc} = L_{de} = S_c \sqrt{\sin \left( \frac{\pi + \phi}{4} \right)} \]  
(7.55)

\[ L_{cd} = B - 2S_e \cotg \left( \frac{\pi + \phi}{4} \right) \]  
(7.56)

The average tensile force \( T_{av} \) developed in reinforcement can then be evaluated by Equation (7.51). A triangle distribution as shown in Figure 7.3.3 is again assumed here to approximately describe the real strain distribution along the reinforcement. The maximum strain in this triangle distribution can be calculated by Equation (7.52).

For silty clay foundation with all geosynthetics reinforcement placed in the influence depth, it is recommended for design purpose that the tensile strain takes the value of 1.5–2% and 0.5–0.8% at the point beneath the center of footing for top and bottom layer geosynthetics, respectively. The corresponding strain for geosynthetics located between the top and bottom layers can be approximately linearly interpolated. A triangular distribution as shown in Figure 7.3.3 is again assumed here to approximately describe the real strain distribution along the reinforcement.

### 7.4 Verification of Analytical Model

To verify the analytical model, the test results obtained by Adams and Collin (1997) are compared with the calculated bearing capacities. A comparison is also made between the field test data of this study and the analytical results. Due to the flowability of sand, the failure of reinforced sand most likely occurs in the reinforced zone with the reinforcement close to the horizontal direction. Due to the cohesive property of silty clay, “deep footing” effect is likely to develop in the reinforced silty clay with the proper reinforcement configuration. The first failure mechanism (failure like footings on a two-layer soil system) would then control the performance of reinforced silty clay. It should be noted that the concept of “deep footing” is different from the traditional concept of “deep foundation”. Traditionally, “deep foundation” refers to piles and drilled shafts. Here, the “deep footing” effect suggests that the performance of reinforced soil is very similar to that of unreinforced soil with a rigid footing having an additional embedment depth equal to the depth of the reinforced zone.

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7.4.1 Comparison with Adams and Collin’s Test Results

A series of large scale model tests on reinforced sand has been reported by Adams and Collin (1997). 0.3 m, 0.61m, and 0.91 m wide square footings were used in their study. The geogrid used in model tests has tensile strengths of 20 kN/m and 25 kN/m at 5% strain in the machine direction and cross-machine direction, respectively. The ultimate bearing capacities were obtained at a settlement ratio $s/B = 10\%$. As the friction angle of soil was provided in the work of Adams and Collin (1997), it was back-calculated from the model test results for the unreinforced case by using the bearing capacity formula suggested by Vesic (1973). The elastic modulus of sand was also back-calculated from the model test results on unreinforced case by substituting settlement $s = 0.1B$ in equation (7.44). The failure of reinforced sand is believed to occur in the reinforced zone with the reinforcement close to the horizontal direction. To illustrate the analytical model, an example calculation for a case adopted from model tests presented by Adams and Collin (1997) is presented in the Appendix G.

Table 7.1 presents a comparison of the measured and estimated bearing capacities for the large scale model tests conducted by Adams and Collin (1997). The predicted values by using the analytical solution of this study are in good agreement with the test results of TL286, TL2861 and TL386.

The soil properties of sand in the test TL3861 were the same as those in the test TL386. The same type and size of geogrid were also used in both tests. The only difference was the number of reinforcement layers. Test TL3861 used two layers of reinforcement, while test TL386 used only one layer of reinforcement. The ultimate bearing capacity of the test TL 3861 should be higher than that of the test TL386, but the measured data was opposite. This may be the reason that the predicted value for TL3861 is much higher as compared to measured data.

The predicted values for the tests TL166 and TL169 are also relatively higher as compared to measured data. This is also due to the test variation. The density of the tests TL166 and TL169 was higher than the density in the tests TL286 and TL2861, but the ultimate bearing capacity of unreinforced sand corresponding to the tests TL166 and TL169 was lower than that of unreinforced sand corresponding to the tests TL286 and TL2861. This is not usual because a footing is expected to have a higher ultimate bearing capacity on denser sand if the other conditions keep the same.
### Table 7.1 Measured and estimated bearing capacities for the experiments conducted by Adams and Collin (1997)

<table>
<thead>
<tr>
<th>Test ID</th>
<th>$q_u$ (kPa)</th>
<th>B (m)</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>u/B</th>
<th>h/B</th>
<th>N</th>
<th>$q_{u(R)}$ (kPa) (measured)</th>
<th>$q_{u(R)}$ (kPa) (calculated)</th>
<th>Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>TL286</td>
<td>270</td>
<td>0.61</td>
<td>14.5</td>
<td>0.25</td>
<td></td>
<td>1</td>
<td>370</td>
<td>335</td>
<td>9.2%</td>
</tr>
<tr>
<td>TL2861</td>
<td>270</td>
<td>0.61</td>
<td>14.5</td>
<td>0.25</td>
<td>0.25</td>
<td>2</td>
<td>370</td>
<td>377</td>
<td>2.2%</td>
</tr>
<tr>
<td>TL386</td>
<td>138</td>
<td>0.61</td>
<td>14.2</td>
<td>0.25</td>
<td></td>
<td>1</td>
<td>203</td>
<td>203</td>
<td>0.0%</td>
</tr>
<tr>
<td>TL3861</td>
<td>138</td>
<td>0.61</td>
<td>14.2</td>
<td>0.25</td>
<td>0.25</td>
<td>2</td>
<td>185</td>
<td>243</td>
<td>31.4%</td>
</tr>
<tr>
<td>TL166</td>
<td>240</td>
<td>0.61</td>
<td>14.7</td>
<td>0.25</td>
<td></td>
<td>1</td>
<td>360</td>
<td>305</td>
<td>15.3%</td>
</tr>
<tr>
<td>TL169</td>
<td>240</td>
<td>0.61</td>
<td>14.7</td>
<td>0.375</td>
<td></td>
<td>1</td>
<td>398</td>
<td>301</td>
<td>24.4%</td>
</tr>
</tbody>
</table>

#### 7.4.2 Comparison with Large-Scale Field Test Results of this Study

A comparison between the measured and calculated bearing capacities for all five field tests is presented in Table 7.2. An example calculation for five layers of BX6200 geogrid placed at 152 mm spacing is presented in the Appendix G.

It can be seen from Table 7.2 that the predicted values by using the analytical solution with the first failure mechanism (failure like footings on a two layer soil system) are in good agreement with the test results of four layers of BX1500 geogrid placed at 203 mm spacing or five layers of BX6200 geogrid placed at 152 mm spacing. This suggest that the reinforced zone with four layers of BX1500 geogrid placed at 203 mm spacing or five layers of BX6200 geogrid placed at 152 mm spacing is strong enough to develop “deep footing” effect. The reinforced ratios ($R_r$) are $2.43 \frac{MPa}{E_s}$ and $2.12 \frac{MPa}{E_s}$ for these two test sections. For four layers of BX6100 and BX6200 geogrid placed at 203 mm spacing, the reinforced ratios ($R_r$) are 1.15 $\frac{MPa}{E_s}$ and 1.7 $\frac{MPa}{E_s}$, respectively. The predicted values are relatively higher as compared to measured data for these two test sections. The reinforced zone in these two sections may not be strong enough to form perfect “deep footing” effect. The back calculation by applying partial punching shear failure mechanism indicates that the depth of the punching shear failure ($D_p$) is equal to $3d/5$. The reinforced ratio ($R_r$) is recommended to be greater than 2.0 $\frac{MPa}{E_s}$ to achieve the best reinforcement effect for silty clay tested in the present study. For three layers of BX6200 geogrid placed at 305 mm spacing, the vertical spacing ratio ($h/B$) is equal to 2/3. The failure of reinforced silty clay most likely occurred between the top two layers of geogrid. This failure mode should be avoided in engineering practice. As mentioned before, this failure mode can be prevented from occurring by keeping vertical spacing ($h$) less than 0.5B.
Table 7.2 Measured and estimated bearing capacities for the field tests

<table>
<thead>
<tr>
<th>Geogrid Type</th>
<th>B (m)</th>
<th>u/B</th>
<th>h/B</th>
<th>N</th>
<th>Rᵣ (MPa/Eₘᵣ)</th>
<th>qᵤ(R) (kPa) (measured)</th>
<th>qᵤ(R) (kPa) (calculated)</th>
<th>Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>BX6100</td>
<td>0.457</td>
<td>1/3</td>
<td>4/9</td>
<td>4</td>
<td>1.15</td>
<td>1108</td>
<td>1375</td>
<td>24.1%</td>
</tr>
<tr>
<td>BX6200</td>
<td>0.457</td>
<td>1/3</td>
<td>2/3</td>
<td>3</td>
<td>1.27</td>
<td>1055</td>
<td>1376</td>
<td>30.4%</td>
</tr>
<tr>
<td>BX6200</td>
<td>0.457</td>
<td>1/3</td>
<td>4/9</td>
<td>4</td>
<td>1.7</td>
<td>1186</td>
<td>1385</td>
<td>16.8%</td>
</tr>
<tr>
<td>BX6200</td>
<td>0.457</td>
<td>1/3</td>
<td>1/3</td>
<td>5</td>
<td>2.12</td>
<td>1321</td>
<td>1393</td>
<td>5.5%</td>
</tr>
<tr>
<td>BX1500</td>
<td>0.457</td>
<td>1/3</td>
<td>4/9</td>
<td>4</td>
<td>2.43</td>
<td>1302</td>
<td>1394</td>
<td>7.1%</td>
</tr>
</tbody>
</table>

7.5 Comparison of Analytical Solutions with Laboratory Model Test Results

A large number of model tests presented in Chapter 4 provide experimental data to compare the analytical solution described herein. For obtaining the predicted ultimate bearing capacity ratio of reinforced soil, the ultimate bearing capacity ratios were calculated based on aforementioned failure modes. Based on the observation during the model tests, the failure of reinforced sand most likely occurred in the reinforced zone with reinforcement close to the horizontal direction (failure mode 1); the reinforced silty clay in lab tests behaved like footing on a two layer soil system (failure mode 2); and the partial punching failure in the reinforced zone is most likely to happen in geosynthetics reinforced crushed limestone in this study. The design methods from Huang and Tatsuoka (1990), Huang and Menq (1997), Kumar and Saran (2003), and Wayne et al. (1998) are also compared with proposed analytical solution.

7.5.1 Laboratory Model Test Series for Silty Clay

Only the design methods of Huang and Menq (1997) and Wayne et al. (1998) can be applied for cohesive soil. Compared with laboratory test results, Huang and Menq’s method underestimates the bearing capacity of reinforced silty clay. This may be expected in the light of the fact that the regression model for the estimate of the load-spreading angle (wide slab effect) developed in their study was based on model test results for sand. On the other hand, Wayne et al.’s method overpredicts the bearing capacity of reinforced clay. In their method, Wayne et al (1998) assumed that the reinforcement sheet turns vertically at the punching failure surface due to the punching failure. This assumption requires a large deformation to be developed. The visual inspection of the reinforcement after the tests confirmed that this amount of deformation wasn’t reached in this study. The proposed design method of this study assumed that the reinforcement remains horizontal at the ultimate bearing capacity. It can be seen from Figures 7.5.1 through 7.5.4 that the assumption of the horizontal reinforcement gives better prediction of bearing capacity as compared to the assumption of vertical reinforcement along the punching failure.
surface. This suggests that taking the reinforcement as horizontal is more appropriate for this study. The actual shape of the reinforcement at the ultimate bearing capacity should be between these two cases. The relatively poor match between measured and predicted BCR for geotextile reinforced silty clay as show in Figure 7.5.4 may be due to the slack of woven geotextile.

![Figure 7.5.1 BCR vs. number of layers (N) for reinforced silty clay with BasXgrid11 geogrid](image1)

![Figure 7.5.2 BCR vs. number of layers (N) for reinforced silty clay with BX6100 geogrid](image2)
Figure 7.5.3 BCR vs. number of layers (N) for reinforced silty clay with BX6200 geogrid

Figure 7.5.4 BCR vs. number of layers (N) for reinforced silty clay with HP570 geotextile

7.5.2 Laboratory Model Test Series for Sand
All other methods overestimated the performance of reinforced sand. The proposed design method provides a better prediction. The “Deep footing” effect is explicitly or implicitly implied in the design methods of Huang and Tatsuoka (1990), Huang and Menq (1997), and Wayne et al. (1998). This effect results in an almost linear increase of the bearing capacity ratio with increasing the number of reinforcement layers or total depth of the reinforcement because of relatively high friction angle of sand. It seems that using geosynthetics to reinforce uniform sand can’t form this effect due to the flowability of sand. The design method of Kumar and Saran (2003) assumes that all reinforcement layers either fail by tension rupture or by pull out of reinforcement. This assumption leads to the result that the tensile force developed in reinforcement increases with increasing the depth of reinforcement (because normal load increases) which is obviously opposite to the measured data of this study.

Figure 7.5.5 BCR vs. number of layers (N) for reinforced sand with BasXgrid11 geogrid
(Dv/B = 0.0)
Figure 7.5.6 BCR vs. number of layers (N) for reinforced sand with HP570 geotextile (Dφ/B = 0.0)

Figure 7.5.7 BCR vs. number of layers (N) for reinforced sand with BasXgrid11 geogrid (Dφ/B = 1.0)
Figure 7.5.8 BCR vs. number of layers (N) for reinforced sand with BX6100 geogrid (Df/B = 1.0)

Figure 7.5.9 BCR vs. number of layers (N) for reinforced sand with HP570 geotextile (Df/B = 1.0)
7.5.3 Laboratory Model Test Series for Kentucky Crushed Limestone

The partial punching shear failure in the reinforced zone most likely occurs in the geosynthetics reinforced crushed limestone. The depth of the punching shear failure \((D_p)\) is taken as one fourth of the total depth of reinforcement \((d)\), i.e. \(D_p = d/4\). Again, as indicated in the sand, all other methods overestimate the performance of crushed limestone reinforced by geosynthetics. The proposed design method provides a better prediction. However, the design method of Huang and Menq (1997) gives a good prediction of BCR for crushed limestone reinforced by steel wire mesh and steel bar mesh. It seems that if the reinforcement has much higher stiffness as compared to the crushed limestone, the reinforced mass would act as a rigid block and the “deep footing” effect can then be formed. In this case, Huang and Menq’s method is recommended for use.

![Figure 7.5.10 BCR vs. number of layers (N) for reinforced crushed limestone with BX1100 geogrid](image)
Figure 7.5.11 BCR vs. number of layers (N) for reinforced crushed limestone with BX1200 geogrid

Figure 7.5.12 BCR vs. number of layers (N) for reinforced crushed limestone with BX1500 geogrid
Figure 7.5.13 BCR vs. number of layers (N) for reinforced crushed limestone with BasXgrid11 geogrid

Figure 7.5.14 BCR vs. number of layers (N) for reinforced crushed limestone with MS330 geogrid
Figure 7.5.15 BCR vs. number of layers (N) for reinforced crushed limestone with steel wire mesh

Figure 7.5.16 BCR vs. number of layers (N) for reinforced crushed limestone with steel bar mesh
7.6 Procedure for Reinforced Soil Foundation Design

The following step by step procedure is recommended for the design of reinforced soil foundation.
1. Assume the footing width, \( B \).
2. Calculate the ultimate bearing capacity of unreinforced soil foundation, \( q_u \).
3. Determine the bearing pressure along the bottom of a shallow foundation, \( q \).
4. Select the geogrid with specific tensile modulus (\( J \)) and the proper reinforcement layout.
   Based on the experimental test results of this study, typical design parameters for reinforcement layout are recommended in Table 7.3.

<table>
<thead>
<tr>
<th>Typical value</th>
<th>Recommended</th>
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<tr>
<td>( u/B )</td>
<td>0.2 ~ 0.5</td>
</tr>
<tr>
<td>( h/B )</td>
<td>0.2 ~ 0.5</td>
</tr>
<tr>
<td>( d/B )</td>
<td>1.3 ~ 1.7</td>
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<tr>
<td>( l/B )</td>
<td>4 ~ 6</td>
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</table>

5. Determine the possible failure mode of reinforced soil foundation
6. Determine the tensile force, \( T \), developed in the reinforcement using the method suggested in Chapter 7.3.
7. Calculate the increased bearing capacity due to the contribution of the reinforcement.
8. Calculate the ultimate bearing capacity of reinforced soil foundation, \( q_u(R) \).
9. Calculate the allowable bearing capacity of reinforced soil foundation, \( q_a(R) \) as

\[
q_{a(R)} = \frac{q_{u(R)}}{F_s}
\]

(7.57)

where \( F_s \) is the factor of safety
10. If the allowable bearing capacity of reinforced soil foundation, \( q_a(R) \), is lower than the bearing pressure, \( q \), repeat Steps 1 through 9.

7.7 Summary and Conclusions

The failure mechanisms of reinforced soil foundation were proposed based on the literature review and experimental test results of the present study. Stability analysis including the effect of reinforcement has been conducted based on these proposed failure mechanisms. The new bearing capacity formula with the contribution of reinforcements to an increase in bearing capacity was
then developed for reinforced soil foundation. In the light of the fact that the mobilized tensile force in reinforcement needs to be known to quantify the benefit of reinforcement, a reasonable estimation on the tensile force along the reinforcement was proposed. The design method proposed in this study has been verified by the large scale field test results obtained by Adams and Collin (1997) for reinforced sand and the author for reinforced silty clay. The predicted values matched well with the test results.
CHAPTER 8 SUMMARY, CONCLUSIONS, AND SUGGESTIONS FOR FUTURE RESEARCH

8.1 Summary

The benefits of using geosynthetics to reinforce soils have been widely recognized. Past research works available in the literature demonstrated that the use of reinforcements can significantly increase the bearing capacity of the soil foundation and reduce the settlement of the footing.

This research is undertaken to investigate the potential benefits of using reinforcement to improve the bearing capacity and reduce the settlement of shallow foundations on soils. For this purpose, four series of tests were conducted, small-scale laboratory tests on silty clay soil, small-scale laboratory tests on sandy soil, small-scale laboratory tests on Kentucky crushed limestone, and large-scale field tests on silty clay embankment soil. The influences of different variables and parameters contributing to the improved performance of reinforced soil foundation (RSF) were examined in these tests. The investigated parameters include top layer spacing (\(u\)), number of reinforcement layers (\(N\)), vertical spacing between reinforcement layers (\(h\)), ensile modulus and type of reinforcement, embedment of the footing (\(D_f\)), shape of footing, and type of soil. In the mean time, 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. Statistical analyses were conducted on the model test results to develop regression models to predict the bearing capacity ratio of reinforced soil. An axisymmetric finite element analysis with three series of reinforcement layout strategy was performed to study the scale effect on the results of model footing tests. The results of model tests were used to examine the existing analytical solutions proposed by other researchers. Based on the literature review and experimental test results of present study, possible failure modes were identified for reinforced soil foundation; and used to develop new methods to calculate the bearing capacity of RSF for different soil types. Based on laboratory and analytical studies, a step-by-step procedure was recommended for the design of reinforced soil foundation.

8.2 Conclusions

Based on the results of the present study, the following conclusions can be drawn:

a. The inclusion of reinforcement generally resulted in increasing the ultimate bearing capacity of soils and reducing the footing settlement.
b. The optimum depth to first reinforcement layer was estimated to be at about 0.33B below the footing for all soil types tested in this study.

c. The bearing capacity of reinforced soil increases with increasing number of reinforcement layers (at same vertical spacing). However, the significance of an additional reinforcement layer decreases with the increase in number of layers. The reinforcing effect becomes negligible below the influence depth. The influence depth of reinforced sand was obtained at approximately 1.25B in this study regardless of the type of reinforcement and footing embedment depth; while the influence depth of geogrid and geotextile reinforced silty clay was obtained at about 1.5B and 1.25B, respectively.

d. The bearing capacity ratio (BCR) decreases with increasing vertical spacing of reinforcement layers. No optimum vertical spacing was observed for the geogrid reinforced silty clay and sand tested soils. For the tested soil (silty clay and sand) and geogrid reinforcement, one can realize that the smaller the spacing, the higher the BCR. In practice, cost would govern the spacing and require 152 mm (6 in.) ≤ h ≤ 457 mm (18 in.) For design purposes, engineers need to balance between reducing spacing and increasing geogrid tensile modulus. The author believes that a value of $h/B = 0.33$ can be a reasonable value for use in the design of reinforced soil.

e. Geogrid beyond the effective length (4.0~6.0B) results in insignificant mobilized tensile strength, and thus provides negligible reinforcement benefit.

f. In general, the performance of reinforced soil improves with increasing the reinforcement tensile modulus. For a project controlled by settlement criteria, geogrid reinforcement is generally considered to perform better for soil foundation than geotextile.

g. The inclusion of reinforcement will redistribute the applied load to a wider area, thus minimizing stress concentration and achieving a more uniform stress distribution. The redistribution of stresses below the reinforced zone will result in reducing the consolidation settlement of the underlying weak clayey soil which is directly related to the induced stress. With the appropriate reinforcement configuration, the inclusion of reinforcement can develop “surcharge effect” to prevent soil from moving upward, and thus improve the bearing capacity of soil.

h. The strain developed along the reinforcement is directly related to the settlement, and therefore higher tension would be developed for geogrid with higher modulus under the same footing settlement.
i. Finite element analysis in this study indicates that the scale effect is mainly related to reinforced ratio \((R_r)\) of the reinforced zone. In laboratory model tests, if we can keep the total depth ratio of reinforcement \((d/B)\) and the reinforced ratio \((R_r)\) the same as those used in actual full scale reinforced soil foundations, the model test results can be extrapolated to the performance of actual full scale reinforced soil foundations.

j. The failure mechanisms of reinforced soil foundations were proposed for different soil types based on the literature review and the model test results of present study. Stability analyses were then conducted on the proposed failure mechanisms of RSFs of tested soil types to evaluate the contribution of reinforcement. New bearing capacity formulas that include the benefit of reinforcement to the increase in bearing capacity were developed for the RSFs of three soil types. In the light of the fact that the mobilized tensile force in reinforcement needs to be known to quantify the benefit of reinforcement, a reasonable estimation on the tensile force along the reinforcement was proposed. The predicted bearing capacities of reinforced soil foundation by using the methods of this study are generally in good agreement with the field test results of previous research for reinforced sand and this study for reinforced silty clay. The proposed methods also provide good predictions of laboratory model test results of this study.

8.3 Suggestions for Future Research

This work presents a detailed study toward understanding the behavior of reinforced soil foundations. However, the performance of reinforced soil foundation is influenced by numerous factors. Due to limited time, this study cannot address all these factors. The future research is recommended to address the following:

1. The results of direct shear tests conducted at the LTRC indicated that the reinforcement efficiency is sensitive to the moisture content of soil. Seasonal variation of soil moisture content is a common phenomenon in poorly drained embankment cohesive soils. Future research work is recommended to determine how the performance of reinforced soil foundation is affected by the variation of soil’s moisture content.

2. Finite element analysis in this study indicated that the scale effect is mainly related to the reinforced ratio \((R_r)\) of the reinforced zone. Future experimental study is recommended to substantiate and improve on this finding. It is also recommended that this finding is evaluated in actual full scale reinforced soil foundations.
3. Most previous experimental studies were focused on short-term behavior of reinforced soil foundations. The tendency of geosynthetics to creep under sustained loading and aging by hydrolysis or abrasion poses a potential risk to the performance of reinforced soil foundation. The future work is recommended to investigate the long-term performance of reinforced soil foundation. In addition, for footings on clay, consolidation settlement and secondary compression, both time dependent settlements, are more critical issues. This study has demonstrated that the inclusion of reinforcement can redistribute the applied load to a wider area below the reinforced zone, thus achieving a more uniform stress distribution and reducing the consolidation settlement of underlying weak clayey soil which is directly related to the stress. The investigation of long-term performance of reinforced soil foundation would provide additional insight into the degree or level of reduction of the consolidation settlement due to the inclusion of reinforcement.
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Http://www.mirafi.com/products/product_hp_index2.html


APPENDIX A

VERTICAL STRESS DISTRIBUTION IN SOIL WITH AND WITHOUT REINFORCEMENT AND TENSION DISTRIBUTION ALONG THE REINFORCEMENT FOR SMALL-SCALE LABORATORY MODEL TESTS ON SILTY CLAY
Figure A.1 Vertical stress distribution along the center line of footing at a depth of 254 mm for multi-layer of BX6200 geogrid reinforced section (B×L: 152 mm×152mm)
Relative Distance From the Center of Footing (x/B)

(c). Applied footing pressure q=937 kPa

Figure A.1 (continued)
Figure A.2 Vertical stress distribution along the center line of footing at a depth of 254 mm for multi-layer of BX6100 geogrid reinforced section (B×L: 152 mm×254 mm)
(c). Applied footing pressure q=703 kPa

(d). Applied footing pressure q=843 kPa

Figure A.2 (continued)
(e). Applied footing pressure $q=984$ kPa

(f). Applied footing pressure $q=1096$ kPa

Figure A.2 (continued)
Figure A.3 Vertical stress distribution along the center line of footing at a depth of 152 mm for three layers of BX6200 geogrid at different vertical spacing (B×L: 152 mm×152 mm)
(c). Applied footing pressure $q=937$ kPa

Figure A.3 (continued)
Figure A.4 Vertical stress distribution along the center line of footing at a depth of 254 mm for three layers of BX6200 geogrid at different vertical spacing (B×L: 152 mm×152 mm)
### Figure A.4 (continued)

Relative Distance From the Center of Footing ($x/B$)

(c). Applied footing pressure $q=937$ kPa

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Figure A.4 (continued)
Figure A.5 Vertical stress distribution along the center line of footing at a depth of 254 mm for five layers of different types of reinforcement (B×L: 152 mm×152 mm)
(c). Applied footing pressure $q=937$ kPa

Figure A.5 (continued)
Figure A.6 Vertical stress distribution along the center line of footing at a depth of 254 mm for five layers of different types of reinforcement (B×L: 152 mm×254mm)
(c). Applied footing pressure $q=703$ kPa

Figure A.6 (continued)
(e). Applied footing pressure $q=984 \text{ kPa}$

(f). Applied footing pressure $q=1096 \text{ kPa}$
Figure A.7 Profiles of vertical stress with the depth below the center of footing
(B×L: 152 mm×152mm)
(c). Applied footing pressure $q=937$ kPa

Figure A.7 (continued)
Figure A.8 Tension distribution along the center line of BX6100 geogrid
(B×L: 152 mm×152 mm)
Figure A.9 Tension distribution along the center line of BX6100 geogrid in the width direction of footing (B×L: 152 mm×254mm)
Figure A.10 Tension distribution along the center line of BX6100 geogrid in the length direction of footing (B×L: 152 mm×254 mm)
APPENDIX B

VERTICAL STRESS DISTRIBUTION IN SOIL WITH AND WITHOUT REINFORCEMENT AND TENSION DISTRIBUTION ALONG THE REINFORCEMENT FOR SMALL-SCALE LABORATORY MODEL TESTS ON SAND
Figure B.1 Vertical stress distribution along the center line of footing at a depth of 254 mm below the footing with multi-layer of BX6100 geogrid (B×L: 152 mm×152mm; D/B: 1.0)
Figure B.1 (continued)

(c). Applied footing pressure $q=1532$ kPa

(d). Applied footing pressure $q=1915$ kPa
Relative Distance From the Center of Footing (x/B)

Figure B.1 (continued)

(e) Applied footing pressure $q = 2298$ kPa
Figure B.2 Vertical stress distribution along the center line of footing at a depth of 254 mm below the footing with multi-layer of HP570 geotextile (B×L: 152 mm×152mm; D/B: 1.0)
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### Applied Footing Pressure

- **(c)** Applied footing pressure $q=1149$ kPa
- **(d)** Applied footing pressure $q=1532$ kPa

**Figure B.2 (continued)**
(e). Applied footing pressure $q=1915$ kPa

(f). Applied footing pressure $q=2298$ kPa

Figure B.2 (continued)
(g). Applied footing pressure $q=2298$ kPa

Figure B.2 (continued)
Figure B.3 Vertical stress distribution along the center line of footing at a depth of 254 mm below the footing with multi-layer of BasXgrid11 geogrid (B×L: 152 mm×152 mm; D/B: 0.0)
### (c). Applied footing pressure $q=375$ kPa

![Graph showing stress distribution for different types of footings.]

### (d). Applied footing pressure $q=468$ kPa

![Graph showing stress distribution for different types of footings.]

**Figure B.3 (continued)**
(e). Applied footing pressure $q=562$ kPa

(f). Applied footing pressure $q=656$ kPa

Figure B.3 (continued)
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(g). Applied footing pressure $q=843$ kPa

Figure B.3 (continued)
(a). Applied footing pressure $q=94$ kPa

(b). Applied footing pressure $q=187$ kPa

Figure B.4 Vertical stress distribution along the center line of footing at a depth of 254 mm below the footing with multi-layer of HP570 geotextile ($B \times L$: 152 mm × 152 mm; $D_f/B$: 0.0)
Relative Distance From the Center of Footing (x/B)

(c). Applied footing pressure q=281 kPa

(d). Applied footing pressure q=375 kPa

Figure B.4 (continued)
Relative Distance From the Center of Footing (x/B)

(e). Applied footing pressure $q=468$ kPa

(f). Applied footing pressure $q=562$ kPa

Figure B.4 (continued)
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</table>

(g). Applied footing pressure $q=656$ kPa

(h). Applied footing pressure $q=750$ kPa

Figure B.4 (continued)
Figure B.4 (continued)

(i). Applied footing pressure $q=843$ kPa
Figure B.5 Vertical stress distribution along the center line of footing at a depth of 254 mm below the footing with four layers of different types of reinforcement
(B×L: 152 mm×152mm; Df/B: 1.0)
(c). Applied footing pressure $q=1532$ kPa

(d). Applied footing pressure $q=1915$ kPa

Figure B.5 (continued)
Figure B.5 (continued)

(e). Applied footing pressure $q=2298$ kPa
Figure B.6 Vertical stress distribution along the center line of footing at a depth of 254 mm below the footing with four layers of different types of reinforcement (B×L: 152 mm×254 mm, Df/B = 1.0)
Figure B.6 (continued)
(e). Applied footing pressure $q=1379$ kPa

(f). Applied footing pressure $q=1609$ kPa

Figure B.6 (continued)
(a). at a depth of 51 mm below the footing

(b). at a depth of 203 mm below the footing

Figure B.7 Tension distribution along the center line of geogrid
(B×L: 152 mm×152 mm; D/B: 0.0)
APPENDIX C

BEARING CAPACITY RATIO AND SETTLEMENT REDUCTION FACTOR VERSUS TYPE OF REINFORCEMENT FOR SMALL-SCALE LABORATORY MODEL TESTS ON KENTUCKY CRUSHED LIMESTONE
Figure C.1 BCR versus type of reinforcement

(a). $s/B = 1\%$

(b). $s/B = 2\%$
Figure C.1 (continued)
Figure C.1 (continued)
Figure C.1 (continued)
Figure C.2 SRF versus type of reinforcement

(a) $q = 2500$ kPa

(b) $q = 3000$ kPa
Figure C.2 (continued)
Figure C.2 (continued)
APPENDIX D

VERTICAL STRESS DISTRIBUTION IN SOIL WITH AND WITHOUT REINFORCEMENT AND TENSION DISTRIBUTION ALONG THE REINFORCEMENT FOR LARGE-SCALE FIELD MODEL TESTS ON SILTY CLAY
Figure D.1 Vertical stress distribution along the center line of footing at a depth of 762 mm for different number of layers of BX6200 placed at different vertical spacing.
Figure D.1 (continued)

(c). Applied footing pressure $q = 936 \text{ kPa}$

(d). Applied footing pressure $q = 1064 \text{ kPa}$
Figure D.2 Vertical stress distribution along the center line of footing at a depth of 762 mm for four layers of different types of reinforcement

(a). Applied footing pressure $q = 255$ kPa

(b). Applied footing pressure $q = 723$ kPa
(c). Applied footing pressure $q = 936$ kPa

(d). Applied footing pressure $q = 1064$ kPa

Figure D.2 (continued)
Figure D.3 Profiles of vertical stress with the depth below the center of footing for different number of layers of BX6200 placed at different vertical spacing
(c). Applied footing pressure q = 936 kPa

(d). Applied footing pressure q = 1064 kPa

Figure D.3 (continued)
Figure D.4 Profiles of vertical stress with the depth below the center of footing for four layers of different types of reinforcement.

(a). Applied footing pressure \( q = 255 \) kPa

(b). Applied footing pressure \( q = 723 \) kPa
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(c). Applied footing pressure $q = 936$ kPa

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(d). Applied footing pressure $q = 1064$ kPa

Figure D.4 (continued)
Figure D.5 Measured and calculated vertical stress distribution along the center line of footing at a depth of 762 mm
(c). Applied footing pressure \( q = 936 \text{ kPa} \)

(d). Applied footing pressure \( q = 1064 \text{ kPa} \)

Figure D.5 (continued)
Figure D.6 Measured and calculated vertical stress distribution along the center line of footing at a depth of 610 mm
Relative Distance From the Center of Footing (x/B)

(c). Applied footing pressure $q = 936$ kPa

(d). Applied footing pressure $q = 1064$ kPa

Figure D.6 (continued)
Figure D.7 Measured and calculated vertical stress distribution along the center line of footing at a depth of 457 mm

(a). Applied footing pressure $q = 255$ kPa

(b). Applied footing pressure $q = 723$ kPa
(c). Applied footing pressure $q = 936$ kPa

(d). Applied footing pressure $q = 1064$ kPa

Figure D.7 (continued)
Figure D.8 Measured and calculated vertical stress distribution along the depth at the center of footing
(c). Applied footing pressure $q = 936$ kPa

(d). Applied footing pressure $q = 1064$ kPa

Figure D.8 (continued)
Figure D.9 Tension distribution along the center line of BX6100 geogrid placed at a depth of 152 mm
Figure D.10 Tension distribution along the center line of BX6100 geogrid placed at a depth of 762 mm
Figure D.11 Tension distribution along the center line of BX6200 geogrid placed at a depth of 152 mm
Figure D.12 Tension distribution along the center line of BX6200 geogrid placed at a depth of 762 mm
Figure D.13 Tension distribution along the center line of BX1500 geogrid placed at a depth of 152 mm

(a). Machine direction of geogrid

(b). Cross machine direction of geogrid
Figure D.14 Tension distribution along the center line of BX1500 geogrid placed at a depth of 762 mm.
APPENDIX E

CALIBRATION OF STRAIN GAUGES

To fully understand the behavior of geosynthetic reinforced soil foundation, it is necessary to assess the strain developed in the geosynthetic. In this study, strain gauges were attached to the geosynthetics to measure the strain distribution along the geosynthetic. Of concern, is the reliability of the strain data. An in-isolation calibration test was conducted to examine the accuracy of the strain measured by strain gauge and the influence of environmental protection (coating).

E.1 In-isolation calibration test

The apparatus used is United Mechanical Testing Machine SFM-30E manufactured by United Calibration Corporation. The test follows the procedure in ASTM D6637-01. The specimen is a piece of BX6100 geogrid with dimensions of 16 in. long and 8 in. wide. Each end of the geogrid sample was placed in stainless steel clamps. Figure E.1 shows the test setup. Total of four strain gauges were attached to the geogrid in this test. Two strain gauges (gauges 1 and 3 as shown in Figure E.2) were attached to the geogrid with DOW CORNING 3140 MIL-A-46146 RTV COATING (for environmental protection). The other two (gauges 2 and 4 as shown in Figure E.2) were attached to the geogrid without coating.

Video camera was used for noncontact measurement of strain in geogrid in this test. The white points (a, a’, b and b’) were painted on the ribs of geogrid at the locations shown in Figure E.2. Video camera recorded the locations of these white points continuously during loading process. The strains were then calculated based on the relative displacement of points a and a’/b and b’. The calculated strain was then compared to the strain recorded by the strain gauge.

E.2 In-isolation calibration test results

Figure E.3 shows the results for the strain measurements taken with both the strain gauges with/without coating and video camera. Depending on the loading level, the strain measured by strain gauges with coating is about 5~15% less than that measured by strain gauges without coating, while the strain recorded by video camera is 0~5% less than that recorded by strain gauge without coating. These results may be expected in the light of the fact that the attached strain gauge and coating would provide some stiffness to the rib of geogrid and can redistribute the strain around this area.
The test geogrid (BX6100) has the lowest stiffness among geogrids used in this study. For geogrid with higher stiffness, the additional stiffness provided by strain gauge and coating should have less effect on the measured strain as compared to the geogrid with lower stiffness.

A correction factor of 1.12 is then applied for BX6100 geogrid (in machine direction) with coating. Correction factors for other geogrids can be approximately calculated by:

$$S_e = \frac{0.12E_{BX6100-CD} + E}{E}$$

(E.1)

The calculated correction factors for geogrids used in this study are summarized in Table E.1

Figure E.1 In-isolation calibration test set-up
Figure E.2. Locations of strain gauges and strain monitoring points
Figure E.3. Strain gauge calibration test results

Table E.1 Strain correction factors

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APPENDIX F
ALL POSSIBLE REGRESSION MODELS
Table F.1 Possible linear models for silty clay

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### Table F.2 Possible nonlinear models for silty clay

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<th>Variables in model</th>
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Table F.3 Possible linear models for sand

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Table F.4 Possible nonlinear models for sand

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<tr>
<td>0.6757</td>
<td>0.6811</td>
<td>-1192.6658</td>
<td>0.08249</td>
<td>s/B, u/B, h/B, J/(100 kN/m)</td>
</tr>
<tr>
<td>0.6755</td>
<td>0.6796</td>
<td>-1193.5704</td>
<td>0.08251</td>
<td>s/B, h/B, J/(100 kN/m)</td>
</tr>
<tr>
<td>0.6753</td>
<td>0.6780</td>
<td>-1194.3627</td>
<td>0.08254</td>
<td>s/B, h/B</td>
</tr>
<tr>
<td>0.5274</td>
<td>0.5333</td>
<td>-1103.2996</td>
<td>0.09958</td>
<td>h/B, N, J/(100 kN/m)</td>
</tr>
<tr>
<td>0.5268</td>
<td>0.5308</td>
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<td>0.09964</td>
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</tr>
<tr>
<td>0.5262</td>
<td>0.5342</td>
<td>-1101.7268</td>
<td>0.09970</td>
<td>u/B, h/B, N, J/(100 kN/m)</td>
</tr>
<tr>
<td>0.5262</td>
<td>0.5321</td>
<td>-1102.6957</td>
<td>0.09970</td>
<td>u/B, h/B, N</td>
</tr>
<tr>
<td>0.5161</td>
<td>0.5181</td>
<td>-1099.6118</td>
<td>0.10076</td>
<td>N</td>
</tr>
<tr>
<td>0.5159</td>
<td>0.5200</td>
<td>-1098.5315</td>
<td>0.10078</td>
<td>N, J/(100 kN/m)</td>
</tr>
<tr>
<td>0.5159</td>
<td>0.5199</td>
<td>-1098.4999</td>
<td>0.10079</td>
<td>u/B, N</td>
</tr>
<tr>
<td>0.5152</td>
<td>0.5213</td>
<td>-1097.1719</td>
<td>0.10086</td>
<td>u/B, N, J/(100 kN/m)</td>
</tr>
<tr>
<td>0.4907</td>
<td>0.4929</td>
<td>-1087.3504</td>
<td>0.10337</td>
<td>h/B</td>
</tr>
<tr>
<td>0.4906</td>
<td>0.4948</td>
<td>-1086.2765</td>
<td>0.10339</td>
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</tr>
<tr>
<td>0.4902</td>
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</tr>
<tr>
<td>0.4895</td>
<td>0.4959</td>
<td>-1084.8101</td>
<td>0.10349</td>
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</tr>
<tr>
<td>0.2358</td>
<td>0.2453</td>
<td>-987.9505</td>
<td>0.12663</td>
<td>s/B, u/B, J/(100 kN/m)</td>
</tr>
<tr>
<td>0.2127</td>
<td>0.2193</td>
<td>-981.7989</td>
<td>0.12853</td>
<td>s/B, u/B</td>
</tr>
<tr>
<td>0.1986</td>
<td>0.2053</td>
<td>-977.5344</td>
<td>0.12967</td>
<td>s/B, J/(100 kN/m)</td>
</tr>
<tr>
<td>0.1817</td>
<td>0.1851</td>
<td>-973.5253</td>
<td>0.13103</td>
<td>s/B</td>
</tr>
<tr>
<td>0.0523</td>
<td>0.0602</td>
<td>-937.2969</td>
<td>0.14101</td>
<td>u/B, J/(100 kN/m)</td>
</tr>
<tr>
<td>0.0301</td>
<td>0.0341</td>
<td>-932.7294</td>
<td>0.14265</td>
<td>u/B</td>
</tr>
<tr>
<td>0.0160</td>
<td>0.0201</td>
<td>-929.2764</td>
<td>0.14368</td>
<td>J/(100 kN/m)</td>
</tr>
</tbody>
</table>
### Table F.5 Possible linear models for Kentucky crushed limestone

<table>
<thead>
<tr>
<th>$R_{adj}^2$</th>
<th>$R^2$</th>
<th>AIC</th>
<th>$s^2$</th>
<th>Variables in model</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.7455</td>
<td>0.7493</td>
<td>-694.3089</td>
<td>0.17453</td>
<td>s/B, N, J/(100 kN/m)</td>
</tr>
<tr>
<td>0.6279</td>
<td>0.6316</td>
<td>-619.3359</td>
<td>0.21102</td>
<td>s/B, J/(100 kN/m)</td>
</tr>
<tr>
<td>0.4509</td>
<td>0.4565</td>
<td>-541.5261</td>
<td>0.25634</td>
<td>N, J/(100 kN/m)</td>
</tr>
<tr>
<td>0.4190</td>
<td>0.4248</td>
<td>-530.2182</td>
<td>0.26369</td>
<td>s/B, N</td>
</tr>
<tr>
<td>0.3354</td>
<td>0.3388</td>
<td>-504.3311</td>
<td>0.28201</td>
<td>J/(100 kN/m)</td>
</tr>
<tr>
<td>0.2893</td>
<td>0.2929</td>
<td>-490.8999</td>
<td>0.29164</td>
<td>s/B</td>
</tr>
<tr>
<td>0.1276</td>
<td>0.1320</td>
<td>-449.9053</td>
<td>0.32312</td>
<td>N</td>
</tr>
</tbody>
</table>

### Table F.6 Possible nonlinear models for Kentucky crushed limestone

<table>
<thead>
<tr>
<th>$R_{adj}^2$</th>
<th>$R^2$</th>
<th>AIC</th>
<th>$s^2$</th>
<th>Variables in model</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.7824</td>
<td>0.7856</td>
<td>-904.6942</td>
<td>0.10314</td>
<td>s/B, N, J/(100 kN/m)</td>
</tr>
<tr>
<td>0.6578</td>
<td>0.6612</td>
<td>-815.1557</td>
<td>0.12934</td>
<td>s/B, J/(100 kN/m)</td>
</tr>
<tr>
<td>0.4680</td>
<td>0.4734</td>
<td>-726.9279</td>
<td>0.16126</td>
<td>s/B, N</td>
</tr>
<tr>
<td>0.4460</td>
<td>0.4516</td>
<td>-718.8271</td>
<td>0.16455</td>
<td>N, J/(100 kN/m)</td>
</tr>
<tr>
<td>0.3456</td>
<td>0.3489</td>
<td>-686.5101</td>
<td>0.17884</td>
<td>s/B</td>
</tr>
<tr>
<td>0.3088</td>
<td>0.3123</td>
<td>-675.5533</td>
<td>0.18381</td>
<td>J/(100 kN/m)</td>
</tr>
<tr>
<td>0.1350</td>
<td>0.1393</td>
<td>-630.6861</td>
<td>0.20563</td>
<td>N</td>
</tr>
</tbody>
</table>
APPENDIX G
EXAMPLE CALCULATIONS FOR ANALYTICAL SOLUTIONS

G.1 Example Calculations for Reinforced Sand

To illustrate the analytical model, example calculations are presented for a case adopted from model tests presented by Adams and Collin (1997).

The following data are given:

\[ B = 0.61 \text{ m}, \quad D_f = 0.0 \text{ m}, \quad \gamma = 14.5 \text{ kN/m}^3, \quad N = 2, \quad u/B = 0.25, \quad h/B = 0.25, \quad q_u = 270 \text{ kPa} \]

(unreinforced, at \( s/B=10\% \)), \( J = 450 \text{ kN/m} \) (average value in machine and cross-machine direction).

The following data are back-calculated:

\[ \phi = 37.9^\circ, \quad E_s = 3525 \text{ kPa} \]

**Step 1:** Calculating the settlement at the first and second layers of reinforcement:

\( C_1 = 1, \quad C_2 = 1, \quad C_3 = 1 \)

First Layer (at a depth of \( z_1 = u \)):

<table>
<thead>
<tr>
<th>( \Delta z ) (mm)</th>
<th>( E_s ) (kPa)</th>
<th>( z ) (mm)</th>
<th>( I_\varepsilon )</th>
<th>( I_\varepsilon \Delta z/E_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>152.5</td>
<td>3525</td>
<td>228.75</td>
<td>0.986</td>
<td>0.043</td>
</tr>
<tr>
<td>152.5</td>
<td>3525</td>
<td>381.25</td>
<td>1.175</td>
<td>0.051</td>
</tr>
<tr>
<td>152.5</td>
<td>3525</td>
<td>533.75</td>
<td>0.961</td>
<td>0.042</td>
</tr>
<tr>
<td>152.5</td>
<td>3525</td>
<td>686.25</td>
<td>0.747</td>
<td>0.032</td>
</tr>
<tr>
<td>152.5</td>
<td>3525</td>
<td>838.75</td>
<td>0.534</td>
<td>0.023</td>
</tr>
<tr>
<td>152.5</td>
<td>3525</td>
<td>991.25</td>
<td>0.320</td>
<td>0.014</td>
</tr>
<tr>
<td>152.5</td>
<td>3525</td>
<td>1143.75</td>
<td>0.107</td>
<td>0.005</td>
</tr>
<tr>
<td>( \Sigma )</td>
<td></td>
<td></td>
<td>0.209</td>
<td></td>
</tr>
</tbody>
</table>

\[ S_{v1} = C_1 C_2 C_3 (q - \gamma D_f) \sum \frac{I_\varepsilon \Delta z}{E_s} \]

\( = 1)(1)(1)(270 - 0)(0.209) \]

\( = 56.419 \text{ mm} \)

Second Layer (at a depth of \( z_2 = u+h \)): 340
\[
S_{z2} = C_1 C_2 C_3 (g - \gamma D) \sum I \Delta z / E_s
\]

\[
= (1)(1)(1)(270 - 0)(0.166)
\]

\[
= 44.902 \text{ mm}
\]

**Step 2:** Calculating the tensile forces in the first and second layers of reinforcement:

First Layer:

\[
L_{ab} = L_{cd} = \sqrt{s_{a1}^2 + (z_1/2)^2} = \sqrt{56.419^2 + (152.5/2)^2} = 94.854 \text{ mm}
\]

\[
L_{bc} = B = 610 \text{ mm}
\]

\[
L_{ad} = B + z_b = 610 + 152.5 = 762.5 \text{ mm}
\]

Average strain:

\[
\varepsilon_{avg} = \frac{L_{ab} + L_{bc} + L_{cd} - L_{ad}}{L_{ad}} = \frac{94.854 + 610 + 94.854 - 762.5}{762.5} = 4.88\%
\]

\[
\varepsilon_{max} = 2\varepsilon_{avg} = 2 \times 4.88\% = 9.76\%
\]

Strain at the triangle soil wedge faces \(ac\) and \(bc\) as shown in Figure 7.2.7:

\[
\varepsilon = \frac{u}{B + u} \left(2 + \tan(\pi/4 + \phi/2) \right) \varepsilon_{max} = 3.86\%
\]

\[
T_1 = J\varepsilon = 450 \cdot 3.86\% = 17.4 \text{ kN/m}
\]

Second Layer:

\[
L_{ab} = L_{cd} = \sqrt{s_{a1}^2 + (z_2/2)^2} = \sqrt{44.902^2 + (305/2)^2} = 158.973 \text{ mm}
\]

\[
L_{bc} = B = 610 \text{ mm}
\]
\[ L_{\text{ad}} = B + z_y = 610 + 305 = 915 \text{ mm} \]

\[ \varepsilon_{\text{avg}} = \frac{L_{\text{ad}} + L_{bc} + L_{cd} - L_{\text{ad}}}{L_{\text{ad}}} = \frac{158.973 + 610 + 158.973 - 915}{915} = 1.415\% \]

\[ \varepsilon_{\text{max}} = 2\varepsilon_{\text{avg}} = 2 \times 1.415\% = 2.83\% \]

\[ \varepsilon = \frac{u + h}{B + u + h} + \frac{u + h}{2} \varepsilon_{\text{max}} = 1.87\% \]

\[ T_2 = J\varepsilon = 450 \times 1.87\% = 8.4 \text{ kN/m} \]

**Step 3:** Calculating the increased bearing capacity \( \Delta q_T \):

\[ \Delta q_T = \frac{12T_1 \left[ u + (i-1)h \right] \left[ 1 - 2u + (i-1)h \right] \tan \left( \frac{\pi}{4} - \frac{\phi}{2} \right) - B^2}{\varepsilon} = 107 \text{kPa} \]

**Step 4:** Calculating the ultimate bearing capacity of reinforced sand:

\[ q_u + q_T = 1.3cN_v + qN_q + 0.4\gamma BN_r + \Delta q_T = 270 + 107 = 377 \text{ kPa} \]

**G.2 Example Calculations for Reinforced Silty Clay**

To illustrate the analytical model, example calculations are presented for five layers of BX6200 geogrid placed at 152 mm spacing.

The following data are given:

- \( B = 0.457 \text{ m}, D_f = 0.0 \text{ m}, \gamma = 17.3 \text{ kN/m}^3, N = 5, u/B = 1/3, h/B = 1/3, q_u = 896 \text{ kPa} \) (unreinforced, at \( s/B=10\% \)), \( J = 323 \text{ kN/m} \) (average value in machine and cross-machine direction).

The following data are back-calculated:

- \( c = 25 \text{ kPa}, \phi = 280 \)

**Step 1:** The tension developed in the reinforcement at different levels (based on measuring strain):

\( T_1 = 2.65 \text{ kN/m}, T_2 = 2.24 \text{ kN/m}, T_3 = 1.83 \text{ kN/m}, T_4 = 1.42 \text{ kN/m}, T_5 = 1.01 \text{ kN/m} \)

**Step 2:** Calculating the ultimate bearing capacity of the underlying unreinforced silty clay:

\[ N_q = 14.72, \quad N_c = 25.8, \quad N_r = 16.72, \quad d = 0.762 \text{m} \]

\[ q_u = 1.3cN_v + \gamma \left( d + D_f \right) N_q + 0.4\gamma BN_r = 1086 \text{kPa} \]

**Step 3:** Calculating the ultimate bearing capacity of the reinforced silty clay:
\[ K_s = 4.796, \ c_a = 25 \text{ kPa}, \ \delta = 28^o \]

\[ q_{u(b)} = q_b + \frac{4c_a d}{B} + 2\gamma_d \left( 1 + \frac{2D_f}{d} \right) \frac{K_s \tan \phi_i}{B} - \gamma_d d + \frac{4 \sum_{i=1}^{N} T_i \tan \delta}{B} = 1393 \text{ kPa} \]
VITA

Qiming Chen was born on October 20th, 1975, in Hunan Province, People’s Republic of China. He received his bachelor’s degree in civil engineering from Nanjing Architecture and Civil Engineering Institute (currently renamed to Nanjing University of Technology), Nanjing, China, in 1997 and his master’s degree in civil engineering from Tongji University, Shanghai, China, in 2000. Then he worked as a civil engineer in Shanghai, China, for about four years. He came to the United States to pursue a doctoral degree in civil engineering at Louisiana State University in January 2004. The degree of Doctor of Philosophy in civil engineering will be conferred to him in August 2007.