PERFORMANCE OF A GROUP OF GEOPIER ELEMENTS LOADED IN COMPRESSION COMPARED TO SINGLE GEOPIER ELEMENTS AND UNREINFORCED SOIL

Final Report

by

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ABSTRACT

As the population increases and land development continues at a rapid pace, there is a growing need to construct on marginal or inadequate soils. The use of Geopier soil reinforcement can be a cost-effective alternative to other soil improvement methods. The primary objectives of this study were as follows: (a) Evaluate the effectiveness of existing methods of analysis to predict the settlement and ultimate bearing capacity of a Geopier-supported footing loaded in compression, (b) compare the response and behavior of a group of Geopier elements to a single isolated element, and (c) compare the settlement and ultimate bearing capacity of a footing supported by a group of Geopier elements and a footing of the same size bearing on unreinforced soil to determine the magnitude of improvement provided by the Geopier elements.

These objectives were achieved by conducting an extensive field and laboratory subsurface exploration and testing program, several full-scale field load tests, ancillary field load tests, and reduction and analysis of data and results from all tests. Tests performed to determine the strength and stiffness of the in-situ soil layers included field tests (borehole shear, cone penetration, and dilatometer) and laboratory tests (direct shear, one-dimensional consolidation, and stress path). In addition, laboratory tests were conducted to determine index properties and engineering classification of the in-situ soil layers. Full-scale field tests consisted of testing in compression to failure two full-scale footings, one bearing on unreinforced soil and the other bearing on a group of five Geopier elements. Unit cell footing, single pier, and plate load tests were also performed.
Values of ultimate bearing capacity for the footing on unreinforced soil determined using standard methods for general shear failure substantially overestimated the measured capacity from the load test. Additional analyses and experimental data indicated that the bearing failure likely occurred from squeezing of a soft layer or layers sandwiched between harder layers. Several standard methods were used to estimate the settlement of the footing on unreinforced soil. In all cases these methods overestimated the measured settlement by at least several tens of percent. In contrast, standard methods used to predict the ultimate bearing capacity and settlement of the footing bearing on Geopier-reinforced soil provided reliable estimates. The ultimate and allowable bearing capacity of the Geopier-reinforced soil was about five times the values for the unreinforced soil. Results from the single pier load test provided better prediction of the settlement and ultimate bearing capacity of the full-scale Geopier footing than did the unit cell test. Comparison of results from tests on a single Geopier element and a group of five elements showed that there was no significant group effect within the normal range of working loads.
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CHAPTER 1

INTRODUCTION

As the world’s population continues to grow, there is an increasing need to construct on marginal or inadequate soils. Traditionally, deep foundation methods such as piles and drilled concrete shafts have been used to transfer loads either deeper within these marginal or inadequate soils or to better materials below them. Recently, there has been a trend toward improving the load-carrying capacity of these soils using reinforcement, modification, or stabilization. Soil improvement methods that have been used include overexcavation and replacement, deep dynamic compaction, load-bearing fills, sand and stone columns, grouting, mechanically stabilized earth, and chemical stabilization, as well as many other methods.

In 1984 initial research was conducted to develop a system of short aggregate piers to reduce the settlement of structural foundations (Lawton and Fox 1994). This system, known as Geopier® soil reinforcement, was developed as a potentially cost effective alternative to deep foundations and other methods of soil improvement. Geopier elements are constructed by placing granular fill material in open holes created by drilling or excavation. The fill material is compacted in layers to a high density by impact ramming using a specially designed tamper. This method not only produces piers that are stiffer and stronger than the surrounding matrix soil, it also creates a buildup of lateral and vertical stresses that prestresses and prestrains the adjacent matrix soil. This
method was awarded a U.S. patent (Fox and Lawton 1993) owing to its uniqueness.

During the nearly two decades that have passed since the inception of the Geopier system, numerous research studies have been undertaken to establish methods of analysis and design for Geopier-supported foundations and to provide a better understanding of how a Geopier-reinforced soil system responds to applied foundation loads. The work described in this report is a continuation of these studies.

The primary objectives of this current research study were as follows:

1. Evaluate the effectiveness of existing methods of analysis to predict the settlement and ultimate bearing capacity of a Geopier-supported footing loaded in compression.

2. Determine how the response of a group of closely spaced Geopier elements to a compressive load differs, if at all, from the response of a single isolated Geopier element subjected to a compressive load of the same magnitude per pier as the group.

3. Compare the settlement and ultimate bearing capacity for a footing supported by a group of Geopier elements with the settlement and ultimate bearing capacity of the same sized footing bearing on unreinforced soil to determine the magnitude of improvement provided by the Geopier elements.

4. Evaluate the effectiveness of existing methods of analysis for settlement and ultimate bearing capacity of footings bearing on unreinforced soil at this site.
These objectives were achieved by conducting the following tasks:

1. In-situ tests were conducted, including cone penetration tests (CPTU), seismic cone penetration tests (SCPT), borehole shear tests (BST), and dilatometer tests (DMT). The results from these tests were used to determine relevant engineering properties of near-surface soils.

2. Laboratory tests were performed, including one dimensional consolidation, stress path, soil classification (sieve and hydrometer analyses, liquid limit and plastic limit, and specific gravity), and direct shear (DSTM) tests.

3. Field tests were conducted, including testing to failure one full-scale footing on unreinforced soil (FSFOUS), one full-scale footing on soil reinforced with Geopier elements (FSFORS), a unit cell footing, a single Geopier element, and a plate load test on unreinforced soil.

4. Test data were gathered and analyzed. Measurements were taken and analyzed with respect to load-settlement relationships, contact pressures, changes in horizontal pressures, changes in pore pressures, stress dissipation with depth, and lateral ground movements.

5. Results from field tests were analyzed, including bearing capacity, settlement, group effect, and behavior of the pier elements before, during, and after failure.

The remainder of this report is organized as follows:

- A review of pertinent literature is presented in Chapter 2, including the theory behind settlement and ultimate bearing capacity of footings bearing on unreinforced soil, and the settlement and ultimate bearing capacity of a Geopier-supported footing.
• All tests that were conducted in this study are presented in Chapter 3, including tests on soil samples to estimate engineering properties, primary field tests that were conducted and the results from those tests along with comparisons of experimental and predicted values. In addition, general comparisons between the different types of field tests conducted are discussed.

• A summary, along with conclusions and recommendations for future research are presented in Chapter 4.

• The appendices contain a list of notations, unit conversions, examples and results of tests and data analyses performed.
2.1 DEVELOPMENT OF GEOPIER SOIL REINFORCEMENT

Geopier soil reinforcement was developed in the early 1980s to provide an economical alternative to deep foundations and traditional methods of soil improvement such as overexcavation and replacement. More details on the development can be found in Lawton and Fox (1994) and Fox and Cowell (1998).

2.1.1 Installation Procedure

Fox and Cowell (1998) presented a detailed discussion of the installation procedure for compressive piers (see Fig. 1) that is discussed briefly below:

1. A hole is excavated with the use of a drill rig (see Fig. 2).
2. Stone is placed at the bottom of the hole and compacted in thin lifts (see Fig. 3). Typically the lift thicknesses are 300 mm (12 in.) or less. A relatively high frequency, impact tamper with a specially designed 45-degree beveled head is used to compact the aggregate, which results in increased vertical and horizontal stresses in the adjacent matrix soil during and after impact. During this process, the diameter of the pier is increased by approximately 76 mm (3 in.) beyond the

a. Make cavity.

b. Place stone at bottom of cavity.

c. Make a bottom bulb. Densify and vertically prestresses matrix soils beneath the bottom bulb.

d. Make undulated-sided pier shaft with 305 mm (or less) thick lifts. Build up lateral soil pressures in matrix soil during shaft construction. Use well-graded base course stone in pier shaft above groundwater levels.
FIG. 2. Geopier Installation Equipment: Excavator Fitted with Special Tamper (left) and Drill Rig (right)

FIG. 3. Loader Fitted with Modified Bucket for Placement of Aggregate
nominal dimensions of the hole. In addition, an estimated one-pier diameter increase in length occurs owing to the creation of a bulb at the bottom of the pier. This increase in the size of the pier prestresses and prestrains the adjacent soil.

3. More lifts are then added until the compacted aggregate reaches an elevation somewhat above the desired final elevation.

4. Due to lack of confinement and therefore a decreased ability to compact the soil at the ground surface, the top of the pier is constructed some distance below the elevation of the surrounding grade. The surrounding soil is then excavated to final grade (i.e., top of pier) after installation of the pier.

2.1.2 Types of Pier Elements

Two types of piers can be constructed – compressive (bearing) and uplift. The installation procedures for the two types are similar, with the following additional step required to construct an uplift pier: After creation of the bottom bulb, an uplift setup is lowered to the bottom of the hole. This setup consists of threaded steel bars attached to a steel plate by nuts above and below the plate (Figs. 4 and 5). The remainder of the pier is then constructed in the manner described previously.

In a foundation system using compressive piers, the structural foundation (footing, embankment, etc.) bears on top of the pier-reinforced soil. When uplift piers are used, the threaded uplift bars are anchored to the structural foundation either by tying into the reinforcing steel of a reinforced concrete footing or using a separate anchorage system.
FIG. 4. Types of Geopier Elements (modified from Fox and Cowell 1998)

FIG. 5. Uplift Setup Used in This Research Project
2.2 COMPARISON WITH OTHER METHODS OF SOIL IMPROVEMENT

Two methods commonly used for soil improvement are stone columns and overexcavation/replacement. Geopier elements appear to be similar to stone columns but are distinct in several important ways, including the following cited by Lawton and Fox (1994):

1. Piers are designed primarily to stiffen the subgrade. Some increased radial drainage and subgrade strengthening are usually secondary considerations but can be important in some applications.

2. Piers are short, typically two to eight times as long as they are wide.

3. Piers are excavated rather than displaced by vertical or horizontal vibration, thus preserving much of the surrounding soil’s natural fabric and cementation.

4. Construction of piers is by relatively high frequency impact rather than the vibratory techniques used to install stone columns.

5. The piers are constructed using thin lifts that prestrain, prestress, and densify the adjacent soils.

Overexcavation and replacement can be an expensive and time-consuming process. Pier construction is by excavated holes that are spaced in accordance to the need of the foundation. In overexcavation and replacement, the entire area is excavated and replaced thus requiring large amounts of aggregate and a long time to remove, replace, and compact the replaced soil.
2.3 FOUNDATION SOIL ANALYSIS

2.3.1 Settlement Analysis of Footings on Unreinforced Soil

Settlement of a footing bearing on unreinforced ground can consist of some or all of the following components:

\[ S_t = S_i + S_c + S_s + S_m \]  \hspace{1cm} (1)

where

- \( S_t \) = total settlement
- \( S_i \) = immediate settlement
- \( S_c \) = settlement from primary consolidation
- \( S_s \) = settlement from secondary compression
- \( S_m \) = settlement induced by changes in moisture

Immediate settlement is typically the largest component in partially saturated soils and in granular soils. Settlement due to primary consolidation occurs quickly in these soils so that it is difficult to differentiate from immediate settlement. Therefore, the two components are often analyzed as one and called immediate settlement. There are several methods commonly used to estimate immediate settlement. Schmertmann’s (1970, 1978) procedure for immediate settlement of rigid footings on granular soils is probably the most popular method. The immediate settlement is determined using the following equation:

\[ S_i = C_i \Delta q \sum_{i=1}^{i=n} \left( \frac{I_z}{E_s} \right)_i \Delta z_i \]  \hspace{1cm} (2)

where

- \( C_i \) = strain relief factor owing to embedment
\[ \Delta q \] = contact stress at the bearing level that causes settlement

\[ I_z \] = strain influence factor

\[ E_s \] = stress-strain (Young’s) modulus of the soil

\[ \Delta z_i \] = thickness of the \( i^{th} \) sub layer

In engineering practice, values of \( E_s \) of each sublayer are typically determined using empirical equations based on either SPT blowcounts (\( N \)) or CPT tip resistance (\( q_c \)). Schmertmann (1970) recommended that values for \( E_s \) be estimated from \( q_c \) rather than \( N \). Schmertmann et al. (1978) proposed the following equations for \( E_s = f(q_c) \) for granular soils:

**Axisymmetric Conditions (Circular or Square Footing)**

\[ E_s = 2.5 \ q_c \]  

(3a)

**Plane-Strain Conditions (Long Footing)**

\[ E_s = 3.5 \ q_c \]  

(3b)

Although not specifically stated by Schmertmann et al., it is apparent from other correlations that Eq. 3 is valid for normally consolidated granular soils since correlations for overconsolidated granular soils typically give larger values for \( E_s \) than indicated in Eq. 3.

Many other equations have been developed or proposed. Bowles (1996, p. 316) has tabulated equations for \( E_s = f(q_c) \) found in the literature for different types of soil, with the values ranging as follows:

\[ E_s = (1 \ to \ 8) \ q_c \]  

(4)
Bowles also indicated that Young’s modulus for an overconsolidated soil \((E_{s,OC})\) can be estimated from the modulus for the same soil in the normally consolidated state \((E_{s,NC})\) as follows:

\[
E_{s,OC} = E_{s,NC} \sqrt{OCR}
\]  

\(\text{(5)}\)

The strain influence factor varies with depth below the bearing level \((z_b)\) and strain conditions as shown in Fig. 6. For axisymmetric conditions, the influence factor is 0.1 at the bearing level and zero at a depth of \(2B\), where \(B\) is the width of the footing. The peak value occurs at a depth of \(B/2\) and is computed using Eq. 6:

\[
I_{zp} = 0.5 + 0.1 \sqrt{\frac{\Delta q}{\sigma'_{v0}}}
\]  

\(\text{(6)}\)

where \(\sigma'_{v0}\) = initial vertical effective stress at a depth of \(B/2\) below the bearing level.

The influence factor for plane strain conditions is 0.2 at the bearing level and zero at a depth of \(4B\). \(I_{zp}\) occurs at a depth of \(B\) and is also computed using Eq. 6.

Bowles’ (1987, 1996) Modified Elastic Theory is valid for \(S_i\) of a flexible rectangular foundation on an elastic half-space. \(S_i\) for strains occurring beneath the center of a foundation is computed using the following equation:

\[
S_i = 4\Delta qB' \frac{1-\mu^2}{E_s} I_s I_F
\]  

\(\text{(7)}\)

\[
I_s = I_1 + \frac{1-2\mu}{1-\mu} I_2
\]  

\(\text{(8)}\)

where \(E_s\) = weighted average Young’s modulus for the settlement influence zone.
FIG. 6. Strain Influence Factor Diagram: (a) Strain Influence Factor Distributions; and (b) Explanation of Pressure Terms in Eq. 6 (modified from Schmertmann et al. 1978, reproduced by permission of ASCE)
Δq = bearing stress that will cause settlement, in units of \(E_s\)

\[ B' = B / 2 \]

\[ I_i = \text{influence factors, which depend on } L'/B', H, \mu, \text{ and } D_f \]

\[ L = \text{length of the footing} \]

\[ L' = L / 2 \]

\[ \mu = \text{Poisson’s Ratio} \]

\[ H = \text{height of settlement influence zone} \]

\[ D_f = \text{depth of embedment} \]

The half-space may be cohesionless soil of any water content or non-saturated cohesive soils (Bowles 1996).

The settlement profile of a flexible foundation with the applied load in the center of the foundation is largest at the center of the foundation and least at the edges. The settlement profile of a rigid foundation with the load applied in the center of the foundation is uniform. This discrepancy in the settlement patterns requires adjustments to values of \(S_i\) calculated using Eq. 7 if the actual foundation is rigid. NAVFAC (1982) gives influence factors \(I\) based on shape and rigidity to determine \(S_i\) for points within the contact area of a load applied to the surface of an elastic half space. \(I\) is 0.82 for the center of a square rigid foundation, while \(I\) is 1.12 for the center of a square flexible foundation. The ratio of these two factors is 0.73 and this value should be multiplied by the value of \(S_i\) estimated obtained from Eq. 7 to account for rigidity.

Primary consolidation settlement is usually the major contributor to total settlement for saturated clays. Primary consolidation settlement for one-dimensional (1D) strain conditions is commonly determined using the following equations:
\[ S_c = \sum_{i=1}^{i=n} \frac{C_{ri}}{1 + e_{oi}} \log \left( \frac{\sigma'_{v0i}}{\sigma'_{v0i}} \right) H_{oi} \quad \text{(for } \sigma'_{v0} < \sigma'_{vi} \leq \sigma'_{vp}) \quad (9) \]

\[ S_c = \sum_{i=1}^{i=n} \frac{C_{ri}}{1 + e_{oi}} \log \left( \frac{\sigma'_{v0i}}{\sigma'_{v0i}} \right) H_{oi} + \frac{C_{ci}}{1 + e_{pi}} \log \left( \frac{\sigma'_{v0i}}{\sigma'_{v0i}} \right) H_{oi} \quad \text{(for } \sigma'_{v0} < \sigma'_{vp} \leq \sigma'_{vi}) \quad (10) \]

\[ S_c = \sum_{i=1}^{i=n} \frac{C_{ri}}{1 + e_{oi}} \log \left( \frac{\sigma'_{v0i}}{\sigma'_{v0i}} \right) H_{oi} \quad \text{(for } \sigma'_{v0} = \sigma'_{vp} \leq \sigma'_{vi}) \quad (11) \]

where  
\[ C_{ri} = \text{recompression index of the } i^{th} \text{ sublayer} \]
\[ C_{ci} = \text{compression index of the } i^{th} \text{ sublayer} \]
\[ e_{oi} = \text{initial void ratio of the } i^{th} \text{ sublayer} \]
\[ e_{pi} = \text{void ratio at the effective preconsolidation stress of the } i^{th} \text{ sublayer} \]
\[ \sigma'_{vpi} = \text{effective preconsolidation stress of the } i^{th} \text{ sublayer} \]
\[ \sigma'_{v0i} = \text{initial effective vertical stress of the } i^{th} \text{ sublayer} \]
\[ \sigma'_{v1i} = \text{effective vertical stress of the } i^{th} \text{ sublayer after equilibrium has been achieved} \]
\[ H_{0i} = \text{initial height of the } i^{th} \text{ sublayer} \]
\[ H_{pi} = \text{height of the } i^{th} \text{ sublayer when the effective stress is equal to the effective preconsolidation stress} \]

The compressible layer should be divided into sublayers of relatively small thickness to increase accuracy. Eqs. 9-11 are valid for a condition of 1D strain. This condition is approximately satisfied in the field when the width of a uniformly loaded area is much greater than the thickness of the compressible layer or when the bearing zone is confined laterally. For a true condition of 1D strain, the induced excess pore water pressure is equal to the applied stress at the instant the load occurs. This relationship between
induced excess pore water pressure and applied stress is not valid when the strain condition is either two or three-dimensional (2D or 3D). Skempton and Bjerrum (1957) proposed that the following equations be used to correct for 2D or 3D strain effects when calculating $S_c$:

$$S_c(2D \text{ or } 3D) = K \cdot S_c(1D)$$

(12)

$$K = A + \alpha (1 - A)$$

(13)

where $A =$ Skempton’s pore pressure parameter

$$\alpha = f(\text{shape of bearing area, geometry of soil profile, } \mu)$$

$\mu =$ Poisson’s ratio

Using this method, $S_c(2D)$ or $S_c(3D)$ will always be less than or equal to $S_c(1D)$. An average value of Skempton’s pore pressure parameter $A$ for the bearing zone is usually determined from triaxial tests or estimated from Table 1 or the engineer’s experience. Appropriate values for $\alpha$ can be estimated from the data given in Table 2.

Leonards (1976) developed an alternate method to estimate $K = f(OCR, B/H)$ as illustrated in Fig. 7. If the width of the loaded area at the top of the compressible stratum ($B$) is more than four times the thickness of the compressible stratum ($H$), or if the depth from the bearing level to the top of the clay stratum is more $2B$, then $K$ is commonly assumed to be 1.0 (Holtz 1991).

To use Eqs. 9-11 for input into Eq. 12, the induced vertical stress at the midheight of each sublayer needs to be estimated. Methods appropriate for a foundation bearing on the ground surface include equations based on the Boussinesq and Westergaard theories and the 2:1 method. If the foundation is embedded, methods that account for the embed-
TABLE 1. Typical Values of Skempton’s Pore Pressure Parameter $A$ for the Working Range of Stress Below a Foundation (from Skempton & Berrum 1957)

<table>
<thead>
<tr>
<th>Types of Clay</th>
<th>$A$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very sensitive soft clays</td>
<td>$&gt;1.0$</td>
</tr>
<tr>
<td>NC clays</td>
<td>0.50-1.0</td>
</tr>
<tr>
<td>OC clays</td>
<td>0.25-0.50</td>
</tr>
<tr>
<td>Heavily OC sandy clays</td>
<td>0-0.25</td>
</tr>
</tbody>
</table>

TABLE 2. Theoretical Values of Skempton and Bjerrum’s (1957) Modification Factor $\alpha$ for Circular and Strip Flexible Bearing Areas ($\mu = 0.5$)

<table>
<thead>
<tr>
<th>$H/B$</th>
<th>$\alpha_{\text{cir}}^a$</th>
<th>$\alpha_{\text{str}}^b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>0.25</td>
<td>0.67</td>
<td>0.74</td>
</tr>
<tr>
<td>0.5</td>
<td>0.50</td>
<td>0.53</td>
</tr>
<tr>
<td>1</td>
<td>0.38</td>
<td>0.37</td>
</tr>
<tr>
<td>2</td>
<td>0.30</td>
<td>0.26</td>
</tr>
<tr>
<td>4</td>
<td>0.28</td>
<td>0.20</td>
</tr>
<tr>
<td>10</td>
<td>0.26</td>
<td>0.14</td>
</tr>
<tr>
<td>$\infty$</td>
<td>0.25</td>
<td>0.00</td>
</tr>
</tbody>
</table>

$^a$For circular or square footings

$^b$For strip footings

ment should be used, such as the equations developed by Skopek (1961) for an embedded rectangular load and Nishida (1966) for a circular embedded load.

2.3.2 Settlement Analysis of Footings on Soil Reinforced with Geopier Elements

Estimation of settlement of bearing soils reinforced with Geopier elements is more complicated than for untreated soil. Fox and Cowell (1998) describe in detail the methods used in current practice to estimate the settlement of footings bearing on soils reinforced with piers. The total settlement of the reinforced footing is described by the following equation:

\[ S_i = S_{i, LZ} + S_{i, UZ} + S_{c, LZ} + S_{s, LZ} + S_{m, LZ} \]  

(14)

where

- \( S_{i, LZ} \) = immediate settlement of the lower zone
- \( S_{i, UZ} \) = immediate settlement of the upper zone
- \( S_{c, LZ} \) = settlement from primary consolidation of the lower zone
- \( S_{s, LZ} \) = settlement from secondary consolidation of the lower zone
- \( S_{m, LZ} \) = settlement from changes in moisture within the lower zone

Fig. 8 shows the relative locations of the upper and lower zones within the soil system. The height of the upper zone is the length of the pier plus one diameter. The addition of the one diameter is meant to compensate for the bulb at the bottom of the pier (Fox and Cowell 1998).

The settlement of the upper zone is based on the composite stiffness of the pier and the densified matrix soil surrounding the pier. The settlement of the upper zone is a function of the modulus of the pier and the stresses on the pier material, and the modulus
of the matrix soil and the stresses on the matrix soil. Fig. 9 shows a spring analogy used as an aid to understand the development of stress concentration on the Geopier elements along the bearing interface of a footing founded on pier-reinforced soil. The stiffer spring represents the stiffer pier material in Fig. 9 and the softer springs represent the matrix soil. For a rigid footing and a centric load $P$, all springs will deflect the same amount ($\delta$). From fundamental physics it is known that the resisting force generated within a spring ($P$) acts in a direction opposite to the direction of $\delta$, and the magnitude of $P$ is directly proportional to the magnitude of $\delta$, as indicated by the following equation:

$$P = -K \delta$$

(15)

where $K$ = spring constant

Therefore, the resisting force generated by the stiffer springs will be greater than the force generated by the softer springs. In the pier-reinforced system, the bearing stresses generated by the footing on the tops of the piers ($q_p$) are much greater than the stresses generated on the matrix soil ($q_m$). The spring constants are analogous to the subgrade moduli of the piers ($k_p$) and the matrix soil ($k_m$). Subgrade modulus is defined as the induced bearing stress divided by the corresponding settlement.

Summing forces in the vertical direction for the upper part of Fig. 9 gives the following equation:

$$\Sigma F_v = 0 = q_0 A - q_p A_p - q_m A_m$$

(16)

where $F_v$ = vertical force

$q_0$ = average bearing stress
FIG. 8. Upper Zone and Lower Zone Beneath an Isolated Footing (from Fox and Cowell 1998)
FIG. 9. Spring Analogy for Stress Concentration Along Interface of Footing and Geopier-Reinforced Bearing Soil
(modified from Fox and Cowell 1998)
\[ A = \text{total footing area} \]

\[ q_p = \text{induced bearing stress on top of pier} \]

\[ A_p = \text{area of pier} \]

\[ q_m = \text{induced bearing stress on matrix soil} \]

\[ A_m = \text{area of matrix soil} \]

Solving Eq. 16 for \( q_p \) and \( q_m \) gives

\[ q_p = q_0 \frac{R_s}{R_s - 1} \]

\[ q_m = q_0 \frac{1}{R_s - 1} \]

where

\[ R_s = \frac{q_p}{q_m} \]

\[ R_s = \frac{A_p}{A} \]

\( R_s \) and \( R_a \) are termed the stress concentration and areal replacement ratios, respectively.

Typical values for the area replacement ratio used in practice range from about 10-40 percent. It is recommended to use the nominal area of the pier and not to include the increase in size of the pier due to the compaction process. Measured values of \( R_s \) for Geopier elements have ranged from 8 to 40.

If the footing is assumed to be rigid and a subgrade modulus approach is used, the immediate settlement of the upper zone can be approximated by Eq. 21 (Lawton and Fox 1994):

\[ S_{UZ} = \frac{q_p}{k_p} - \frac{q_m}{k_m} \]
This approach has been used on many occasions and is generally conservative. Fox and Cowell (1998) provide a table that can be used to estimate $k_p$ based on the Standard Penetration Test (SPT) blow counts, which can be used for an initial design.

Typically the value of $k_p$ is initially assumed then verified by performing a static plate-load test in which a steel plate is used with a diameter equal to the nominal size of the pier. The value of $k_p$ may be assumed to be linear in the initial part of the $k_p$ vs. $q_p$ curve from the plate load test. However, as the applied load becomes larger the relationship between $k_p$ and $q_p$ becomes non-linear. The results from the static load test are then used to verify the value of $S_{uz}$ obtained using the preliminary value of $k_p$ estimated from SPT blow counts. If the relationship between $q_p$ and settlement is non-linear then an iterative process is implemented. The first steps of the iterative process are to assume a value of $R_s$ and estimate $k_p$ from the $q_p$–settlement curve using the preliminary value of $S_{uz}$. $k_p$ and $q_p$ are then substituted into Eq. 21 to predict $S_{uz}$. If the predicted value of $S_{uz}$ is not equal to the preliminary value, a new value of $S_{uz}$ is assumed and the process repeated until the assumed and calculated values are the same.

### 2.3.3 Modulus of Subgrade Reaction

The modulus of subgrade reaction of the matrix soil or a pier can be estimated from the results of a static plate load test (ASTM D1194, see Fig. 10). Values of subgrade modulus are dependent on: (a) characteristics of the soil, (b) size of the loaded area, and (c) shape of the loaded area. Values from a plate load test should not be used directly to determine $k_m$ for a full-scale footing. Engesser (1893) pointed out that as $B$ increases, $k_m$ decreases.
Terzaghi (1955) gave scaling relationships for two cases. “If the deformation characteristics are more or less independent of depth, like those of stiff clay, it can be assumed that the settlement increases in simple proportion to the depth of the bulb of pressure” (Terzaghi 1955, p. 302). Therefore, the subgrade modulus of full-scale footings on stiff clays or other soils whose stiffness is more or less independent of depth, can be related to the subgrade modulus from the plate load test as follows:

\[
    k_m = k_{mp} \frac{B_p}{B} 
\]  

(22)

where \( k_{mp} \) = subgrade modulus of a square plate load test

\( k_m \) = subgrade modulus of a full-scale footing

\( B_p \) = width of square static load plate

\( B \) = width of full-size footing
“If the beams rest on clean sand, the settlement assumes almost instantaneously its ultimate values, but the deformation characteristics of the sand are a function of depth, because the modulus of elasticity of sand increases with depth” (Terzaghi 1955, p. 302). Terzaghi purposed the subgrade modulus of full-scale footing on sand or other soils whose modulus increases with depth can be estimated from the results of a 0.3 m (1 ft) square plate load test using the following empirical equation:

\[ k_m = k_{m1} \left( \frac{B + B_i}{2B} \right)^2 \] (23a)

where \( B_i \) is the width of the square plate in the same units as \( B \) (0.30 m or 1.0 ft) and \( k_{m1} \) is the subgrade modulus for the plate-load test at the same level of \( q_m \) for which \( k_m \) is desired. If the plate-load test is conducted with a plate width other than 0.30 m (1.0 ft), then the following equation is readily derived from Eq. 23a:

\[ k_m = k_{mp} \left( \frac{B_p \cdot B + B_i}{B \cdot B_p + B_i} \right)^2 \] (23b)

Since the stress-strain behavior of soil is nonlinear, \( k_m \) decreases with increasing load or settlement. Thus, \( k_{mp} \) used to determine \( k_m \) should come from an appropriate value of contact stress for the footing.

2.3.4 Ultimate Bearing Capacity Analysis of Footings on Unreinforced Soil

Most footing design is controlled by settlement; however, there are some cases where ultimate bearing capacity (\( q_{ult} \)) may control. In addition, settlement analyses are applicable only for stresses below \( q_{ult} \). There are many methods to estimate \( q_{ult} \), most of which apply to homogeneous soils. A simple method to estimate \( q_{ult} \) for layered soils
involves using weighted average values of friction angle ($\phi_{av}$), cohesion intercept ($c_{av}$), and unit weight of the soil ($\gamma_{av}$) as given by the following equations (Bowles 1996):

$$\phi_{av} = \tan^{-1}\left( \frac{\sum_{i=1}^{n} H_i \tan \phi_i}{\sum_{i=1}^{n} H_i} \right)$$  \hspace{1cm} (24)$$

$$c_{av} = \left( \frac{\sum_{i=1}^{n} H_i c_i}{\sum_{i=1}^{n} H_i} \right)$$  \hspace{1cm} (25)$$

$$\gamma_{av} = \left( \frac{\sum_{i=1}^{n} H_i \gamma_i}{\sum_{i=1}^{n} H_i} \right)$$  \hspace{1cm} (26)$$

where \( H_i = \) height of the \( i^{th} \) soil layer

\( \phi_i = \) friction angle of the \( i^{th} \) soil layer

\( c_i = \) cohesion intercept of the \( i^{th} \) layer

\( \gamma = \) unit weight of the \( i^{th} \) sublayer

\( n = \) total number of soil layers of composite depth

Bowles (1996) stated that the composite depth below the bearing level \( (d_{comp}) \) for computing average values should be limited to the approximate height of the failure wedge, calculated as follows:

$$d_{comp} = 0.5B \tan \left( 45^\circ + \frac{\phi_{av}}{2} \right)$$  \hspace{1cm} (27)$$
An iterative process is needed to determine appropriate values of $\phi_{av}$, $c_{av}$ and $\gamma_{av}$.

The elevation of the ground water table (GWT) can affect the value of $\gamma$ used to calculate $q_{ult}$. Bowles (1996) discussed the effects of the different possible locations of the GWT. If the GWT is at or above the bearing elevation, then the effective (buoyant) unit weight ($\gamma'$) should be used in place of $\gamma$ in the $N_\gamma$ term. When the GWT is located in the failure wedge zone which extends from the bottom of the footing to a depth equal to $0.5B \tan(45^\circ + \phi/2)$ below the bottom of the footing, an equivalent value of $\gamma$ should be calculated using the following equation:

$$
\gamma_e = (2H - d_w) \frac{d_w}{H^2} \gamma + \frac{\gamma'}{H^2} (H - d_w)^2
$$

(28)

where

- $\gamma_e =$ equivalent unit weight
- $H = 0.5B \tan(45^\circ + \phi/2)$
- $d_w =$ depth to GWT from bearing level (bottom of footing)
- $\gamma =$ total unit weight of soil within depth $d_w$
- $\gamma' =$ effective unit weight below GWT

Different bearing capacity equations for drained and undrained conditions in homogeneous soils have been developed by Terzaghi, Meyerhof and Hansen. It is noted that these equations are based in part of results from small-scale foundation tests, which may not accurately predict the behavior of full-scale footings. Bowles (1996) gives more detailed explanations of all equations and factors. Terzaghi’s (1943) equation (Eq. 29) was derived for rigid, shallow, strip foundations with a rough base where the depth of embedment is less than or equal to the footing width. Additional assumptions and
limitations are that the footing bears on homogeneous soil, the GWT is very deep, and the ground surface is horizontal.

\[ q_{ult} = cN_c s_c + \bar{q} N_q + \frac{1}{2} \gamma BN_\gamma s_\gamma \]  

(29)

where \( \bar{q} \) = effective overburden stress at the bearing elevation outside the footprint of the footing

\( N_c, N_q, \text{ and } N_\gamma = \) bearing capacity factors

\( s_c \) and \( s_\gamma = \) shape factors

Terzaghi assumed a general shear failure below the footing. Vesic (1973) described general shear failure for a typical footing in stress-controlled conditions as a failure that is “sudden and catastrophic”. In strain-controlled conditions, such as jacking, there is a noticeable decrease in the induced load as the footing is displaced downward. The failure surface in this type of shear failure extends from the bottom of the footing to the ground surface. Terzaghi included shape factors in Eq. 29 to account for shapes other than strip footings.

Meyerhof (1963) derived Eq. 30 for both rough shallow foundations subjected to vertical loads and general shear failure:

\[ q_{ult} = cN_c s_c d_c + \bar{q} N_q s_q d_q + \frac{1}{2} \gamma BN_\gamma s_\gamma d_\gamma \]  

(30)

where \( d_c, d_q, d_\gamma = \) depth factors

Meyerhof included depth factors to account for depth of embedment, and included a shape factor \( (s_q) \) with the overburden portion of the equation. Meyerhof’s bearing
capacity factors are different from Terzaghi’s owing to different assumptions. Eq. 30 simplifies to Eq. 31 for unconsolidated undrained (UU) conditions in a saturated soil:

$$q_{ult} = cN_c s_c + q$$  \hspace{1cm} (31)

Hansen’s (1970) equation (Eq. 32) is similar to Meyerhof’s and is applicable for shallow and deep foundations:

$$q_{ult} = cN_c s_c d_c i_c g_c b_c + qN_q s_q d_q i_q g_q b_q + \frac{1}{2} \gamma B'N_{r_f} s_f i_f g_f b_f r_f$$  \hspace{1cm} (32)

Hansen included factors to account for an inclined load, tilted base, sloping ground surface, and base of the footing not parallel to the ground surface. Both Hansen’s and Meyerhof’s bearing capacity factors are the same with exception of $N_r$. Hansen developed Eq. 33 for unconsolidated undrained ($\phi = 0$) conditions:

$$q_{ult} = 5.14s_u (1 + s'_c + d'_c - i'_c - b'_c - g'_c) + q$$  \hspace{1cm} (33)

where $i_c$, $i_q$, and $i_r$ = inclined loading factors

$b_c$, $b_q$, and $b_r$ = tilted base factors

$g_c$, $g_q$, and $g_r$ = sloping ground factors

$s'_c$, $d'_c$, $i'_c$, $b'_c$, and $g'_c$ = factors used when $\phi = 0$

If failure occurs by “squeezing” (Bowles 1996), the following equation can be used to calculate $q_{ult}$:

$$q_{ult} = 4s_u + q$$  \hspace{1cm} (34)
2.3.5 Ultimate Bearing Capacity Analysis of Footings Bearing on Soils Reinforced with Geopier Elements

Bearing capacity analysis of Geopier-reinforced soil is complicated. Therefore, it is prudent to discuss the behavior of a single pier before describing analysis of a group of piers.

2.3.5.1 Bearing Capacity of Soils Reinforced with a Single Pier

Lawton (2001) discussed in detail the behavior of single pier failure mechanisms in homogeneous soils. There are three possible failure mechanisms for a single pier as shown in Fig. 11. Bulging failure can occur in a pier that is installed in relatively weak cohesive soils. Piers are made of cohesionless soils and their behavior is dependent upon the confining effects of the matrix soil. Bulging occurs when the induced horizontal stress in a pier is greater than the lateral resistance of the matrix soil. The bulging may occur in a zone extending from the top of the pier to a depth equal to about two to three times the diameter of the pier \((d_p)\). If the soil is layered, it may only bulge in the weakest layer or in a combination of weaker layers anywhere along the pier where the induced horizontal stresses are greater than lateral resistance of the matrix soil. If the footing covers an area that is greater than the area of the pier, the capacity of the bulging column may be increased in two ways: 1) the matrix soil will carry some of the vertical load, 2) this induced stress carried by the matrix soil will increase the confining pressure on the pier.

Local or general shear failure may occur through the pier and matrix soil. This may occur in the following two cases: 1) if a very short pier \([H_p < (2 \text{ to } 3)d_p]\) bears on a
rigid base, or 2) if the pier is not much stronger than the surrounding matrix soil. This failure mechanism is similar to shallow foundation failures in unreinforced soils.

Punching failure or shearing below the pier is a failure mechanism that occurs when the applied load is greater than the skin friction that develops along the surface of the pier, end bearing resistance, or a combination of both. This type of failure is predominant in very short piers \( H_p < (2 \text{ to } 3)d_p \) that do not bear on a rigid base. It may also occur in piers with substantial cohesion such as lime-cement columns or concrete drilled piers.

FIG. 11. Failure Mechanisms for Single Pier in Homogeneous Soil: (a) Bulging; (b) General or Local Shear; and (c) Punching (modified from Barksdale and Bachus 1983)
The ultimate stress that can be applied to the top of a pier or column \( q_{ult,p} \) for bulging is a product of the limiting radial stress and Rankine’s passive earth pressure for the pier material and is determined by the following equation:

\[
q_{ult,p} = \sigma_{r,lim} k_{pp} = \sigma_{r,lim} \tan^2 \left( 45^\circ + \frac{\phi_p}{2} \right)
\]  

(35)

where  
\( k_{pp} = \) Rankine’s passive earth pressure coefficient of the pier element  
\( \phi_p = \) friction angle of the pier aggregate material  
\( \sigma_{r,lim} = \) limiting radial stress where indefinite expansion occurs

This equation is based on the following assumptions: (a) there is no cohesion in the pier material; (b) the shear stresses that develop at the interface of the pier and the matrix soil are ignored; and (c) the principal stresses act vertically and horizontally (the shearing stresses are ignored along the pier-soil interface). These assumptions are used to simplify the design and are conservative.

The limiting radial stress is the sum of the horizontal stress of the matrix soil at the depth being considered and the ability of the soil to resist the horizontal stress from the pier. Hughes and Withers (1974) idealized the bulging of a pier as a cylindrical expansion into a clay, in a manner similar to a pressuremeter test. Many pressuremeter tests have shown that the soil resists the expansion to a limiting point, after which indefinite expansion occurs. For soil idealized as an elasto-plastic material, Gibson and Anderson (1961) estimated the limiting radial stress as follows:

\[
\sigma_{r,lim} = \sigma_{r0} + s_u \left[ 1 + \ln \left( \frac{E_u}{2s_u (1 + \mu)} \right) \right]
\]  

(36)
where \( \sigma_{ro} = \sigma_{vo} k_{pm} \)

\( k_{pm} \) = Rankine’s lateral earth pressure coefficient for the matrix soil

\( s_u \) = undrained shear strength of the matrix soil

\( E_u \) = undrained modulus of the matrix soil

\( \mu \) = Poisson’s ratio of the matrix soil

If there is insufficient soil information to determine \( E_u \) and \( s_u \), the ratio of undrained modulus and the undrained shear strength (\( E_u / s_u \)) may be conservatively estimated to be 200 and Poisson’s ratio may be estimated to be 0.5. Hughes and Withers (1974) also noted that after examining many field records of quick expansion pressuremeter tests, Eq. 36 can be simplified to

\[
\sigma_{r,lim} = \sigma_{ro}' + 4s_u + u
\]  

(37)

where \( u = \) excess pore pressure

Hughes and Withers noted that there is a lack of drained pressuremeter testing in normally consolidated clay but that Eq. 37 predicts the limiting pressure reasonably well. The excess pore pressure in Eq. 37 can be assumed to be zero, since there is drainage into the pier.

The ultimate stress on top of a pier \( (q_{ult,p}) \) for punching is a combination of the skin friction that develops along the area of the pier and end bearing. Summing the vertical forces of a pier in compression [see Fig. 11 (c)] gives the following equation:

\[
\sum F_y = 0 = q_{ult,p} A_p + W_p - q_{ult,bot,p} A_p - \sum_{i=1}^{n} f_{u,i} p_i \Delta z_i
\]  

(38)
where \( W_p \) is the weight of the pier

\[ f_{si} = \text{skin friction of the i}^{th} \text{ layer} \]

\[ p_i = \text{perimeter length of the pier shaft at the i}^{th} \text{ layer} \]

\[ \Delta z_i = \text{height of the i}^{th} \text{ layer of the pier shaft} \]

\[ q_{ult,bot,p} = \text{bearing capacity of the bottom of the pier end bearing} \]

\[ A_p = \text{nominal area of the pier} \]

\[ n = \text{number of layers} \]

Wissmann (1999) ignored the weight of the pier in Eq. 36 and accounted for the increase in \( A_p \) as follows:

\[
q_{ult,p} = \sum_{i=1}^{n} \left( f_{si} \frac{A_{shaft}}{A_p} \right) + q_{ult,bot,p}
\]

(39)

where \( A_{shaft} \) = area of pier shaft after the increase in area during pier construction,

use \( d_{shaft} = d_p + 76.2 \text{ mm (3 in.)} \) to estimate the area of the shaft

Skin friction is a function of the average horizontal stress of a layer at depth and the frictional characteristics of the materials, as follows:

\[
f_{si} = \left( d_f + \frac{\Delta z_i}{2} \right) \gamma_m k_{pp} \tan(\phi_m)
\]

(40)

where \( d_f = \text{depth to top of i}^{th} \text{ layer} \)

\( \gamma_m = \text{unit weight of matrix soil} \)

\( k_{pp} = \text{Rankine’s lateral earth pressure coefficient of pier soil} \)

\( \phi_m = \text{angle of internal friction of matrix soil} \)
It is noted that this equation ignores any shear stresses that develop during loading of the pier, as discussed by Lawton, et al. (1994). The initial conditions are represented by Mohr’s circle “stress state a” in Fig. 12. As a vertical force is applied to the pier, shearing stresses develop along the pier matrix-soil interface, which causes a rotation of the principal stresses and an arch in the soil (Handy 1985). If the interfacial stresses remain constant up to failure, the failure stresses are represented by circle “b”. An increase in vertical stress would occur ($\sigma_{vb}$) but it is not known if it is sustainable. If it is not sustainable, then the vertical stress would decrease to $\sigma_{vc}$, which would cause a decrease in the horizontal stresses ($\sigma_{hc}$).

Unpublished $K_o$-blade tests were conducted by Handy next to a test pile for the Talmadge Memorial Bridge in Savannah, Georgia. Tests conducted prior to loading indicated that full passive pressures had developed. As the pile was loaded, there was a decrease in horizontal stresses that may have led to the pile failing prematurely by plunging. Lawton, et al. (1994) noted that it is unknown if this same effect will happen to aggregate piers and indicated that additional research is needed to verify if this occurs.

2.3.5.2 Ultimate Bearing Capacity of Soil Reinforced with a Geopier Group

Possible failure mechanisms for pier groups are shown in Fig. 13. Failure in soil reinforced with a pier group is similar to that for a single pier. Bulging, local shear within the pier-reinforced soil matrix, punching below the single pier, and shear below the pier-reinforced matrix zone will be discussed as they pertain to pier groups.

Bulging failure of individual pier elements may occur within a group depending on the spacing of the piers, confining effects of the footing, and the type of soil.
FIG. 12. Theoretical Stress States During Geopier Movement (from Lawton et al. 1994, reproduced by permission of ASCE)
FIG. 13. Possible Failure Mechanisms in Geopier Groups: (a) Local Shear within Pier-Reinforced Matrix Zone; (b) Individual Punching; and (c) Shearing Below Pier-Reinforced Matrix Zone (from Fox and Cowell 1998)
Hughes and Withers (1974) indicated that stone columns act independently if the spacing of the columns is greater than about 2.5 $d_p$. The ultimate bearing capacity for a footing bearing on a group of piers in which bulging failure controls can be derived from Eqs. 17 and 35 as follows:

$$q_{ult} = \frac{q_{ult,p}}{\mu_p}$$  \hspace{1cm} (41)

Local shear failure within the pier-reinforced soil matrix is similar to local shear failure for unreinforced soil except composite parameters are used in the bearing capacity equations (Eqs. 29, 30, 32). This failure is possible if the pier material is only slightly stronger than the matrix soil. Equations for composite strength parameters are as follows (Wissmann 1999):

$$\phi_{comp} = \tan^{-1}\left[\mu'_g R'_a \tan \phi_p + \mu'_m \left(1 - R'_a\right) \tan \phi_m\right]$$  \hspace{1cm} (42)

$$c_{comp} = c_c R'_a + c_m \left(1 - R'_a\right)$$  \hspace{1cm} (43)

$$\gamma_{comp} = \gamma_c R'_a + \gamma_m \left(1 - R'_a\right)$$  \hspace{1cm} (44)

where  

$R'_a = $ area replacement ratio multiplied by a reduction factor of 0.4 to account for shearing planes that extend beyond the footprint of the foundation

$R'_s = $ reduced stress concentration ratio, use 2.8

$\mu'_g$ - use $R'_a$ and $R'_s$ in Eq. 19

$\mu'_m$ - use $R'_a$ and $R'_s$ in Eq. 20

The reduction in $R_s$ is to account for the change in stress and the orientation of the failure plane. A conservative solution is to consider the composite shear strength at a depth of
three-quarters of the footing width and a failure plane of 45 degrees from horizontal (Wissmann 1999).

The composite friction angle is a function of $R_a$, $R_s$, $\phi_p$ and $\phi_m$. The composite cohesion intercept is independent of normal stress and thus is only a function of $R_a$, $c_p$ and $c_m$. Soil can be densified by the applied stress; however this effect is not well understood. It is conservative to ignore this effect so that the composite unit weight is a function of $R_a$, $\gamma_p$, and $\gamma_m$ only.

Wissmann (1999) conservatively suggested that the failure will occur at a depth equal to 0.75$B$. It is recommended to use $R'_s = 2.8$ to account for the increase in shearing area and the orientation of the shear plane. $R'_a$ is a product of $R_a$ and a reduction factor of 0.4 to account for the shearing planes that extend beyond the footprint of the footing.

The bearing capacity analysis for punching failure or shear below the bottom of an individual pier element within a pier group is similar to the analysis for a single pier. Bearing capacity is determined for a single pier using Eq. 39. Eq. 41 is then used to calculate $q_{ult}$ from $q_{ult,p}$.

Shearing or general bearing capacity failure may occur below the pier-reinforced matrix zone. Assuming that the bearing area increases with depth based on the 2:1 method, the length and width of the pier used to estimate bearing capacity are computed with Eqs. 45-46:

\[
L' = L + H \tag{45}
\]

\[
B' = B + H \tag{46}
\]

where $H$ = the nominal length of pier plus one pier diameter
Meyerhof’s or Hansen’s bearing capacity equations (Eqs. 30 or 32) can then be used. The bearing capacity at the bottom of the pier is then compared to the induced stress at depth, which is approximated using the 2:1 method as described by the following equation:

\[
q_{ult} = q_{ult,bot} \left( \frac{(B + H)(L + H)}{BL} \right) = q_{ult,bot} \eta
\]  

(47)

2.3.6 Unit Cell Concept for Settlement and Bearing Capacity of a Pier Group

The concept of a unit cell has been used to conduct settlement and bearing capacity analyses for groups of reinforcing columns (for example, see Barksdale and Bachus 1983). As one example, a group of piers in an equilateral triangular pattern is shown in Fig. 14. The tributary area for each pier consists of a regular hexagon. The resulting equivalent cylinder of material having a diameter \(d_e\) that approximates the tributary soil and one pier is known as a unit cell [Fig. 14(b)]. The pier is concentric to the exterior boundary of the unit cell.

For a large group of piers subjected to a uniform load over the entire area, each interior pier may be considered a unit cell. Owing to symmetry of load and geometry, lateral deformations cannot occur across the boundaries of the unit cell and the shearing stresses on the outside boundaries of the unit cell must be zero. The unit cell can be physically modeled as a cylindrically shaped container having a frictionless and rigid exterior wall that is symmetrically located around the pier. With these idealizations, simplified settlement and bearing capacity analyses of groups of piers can be conducted.
2.3.7 Determination of Ultimate Bearing Capacity from Stress-Settlement Data

To compare experimental results with calculated values, it is necessary to determine when bearing capacity failure occurs during testing. The peak or ultimate load of a footing depends on many factors such as size and shape of the footing, soil type, and rate and frequency of loading. Vesic (1975) recommended that the ultimate load be defined where the slope of the load settlement curve reaches zero as shown by curve 1 in Fig. 15(b), or when the slope of the stress settlement curve reaches a minimum as in curve 2 or 3.
FIG. 15. Determination of Ultimate Bearing Stress from Stress-Settlement Data: (a) Footing Showing Nomenclature; and (b) Sample Load-Settlement Curves (modified from Vesic 1975)
De Beer (1970) described a “double log” method in which the logarithm of normalized settlement (vertical axis) is plotted against normalized contact stress (horizontal axis). Each curve has an upper diagonally-trending line and a lower vertically-trending line. The intersection of these lines is where the bearing capacity failure can be assumed to occur. An example of this method is seen in Fig. 16 where the failure curves are plotted for varying relative densities ($D_r$). De Beer’s parameters are defined as follows:

$$A' = \left( \frac{\bar{q}}{\gamma' B} + \frac{S_i}{B} \right) s_q \left[ 1 + 0.35 \left( \frac{S_i}{B} \right) \right] + s_y \frac{s_y}{2} \tag{48}$$

$$s_q = 1 + 0.2 \left( \frac{B}{L} \right) \tag{49}$$

$$s_y = 1 - 0.4 \left( \frac{B}{L} \right) \tag{50}$$

where $\bar{q} = \text{overburden stress at the bearing level}$

$S_i = \text{total settlement for the loading increment}$

Another way to determine when ultimate bearing capacity occurs is the double tangent method. In this method, a straight line is drawn through the upper linear portion of the failure curve. Another straight line is drawn through the lower linear portion of the failure curve. Where these lines intersect determines the ultimate load. When the line that determines the ultimate load crosses the failure curve, the settlement at failure is defined.
Vesic (1973) provided guidelines for approximate values of settlement at which \( q_{ult} \) occurs. In saturated clay \( q_{ult} \) occurs at a settlement of about 3 to 7% of the footing width. In sand \( q_{ult} \) occurs at a settlement of about 5 to 15% of the footing width for footings bearing on the ground surface. If no noticeable failure occurs during a full-scale footing test, the test should be continued until the settlement is equal to at least 25% of the footing width.

![FIG. 16. Ultimate Load Criterion Using Plot of Logarithm of Normalized Load vs. Logarithm of Normalized Settlement (modified from De Beer 1967)](image-url)
2.3.8 Types of Bearing Capacity Failure Modes

Vesic (1973) performed model tests of footings bearing on sand and identified three modes of bearing capacity failure. The results of Vesic’s tests (Fig. 17), conducted on Chattahoochee sand, cannot be used directly for other sandy soils. However, they can be used as a way to identify possible failure modes from load-settlement graphs. General shear failure [Fig. 17(a)] is characterized as a well-defined failure surface that extends from one side of the footing to the ground surface. In stress-controlled conditions, such as for a typical structural footing, the failure is sudden and catastrophic. There may be substantial tilting of the footing if the structure is allowed to reach this state. In strain-controlled footings, such as test footings, there is a reduction in load with increasing settlement.

Punching shear failure is not easily noticed [Fig. 17(c)]. As the load increases, the settlement increases with a corresponding increase in the density of the soil under the footing. As the footing settles, the overburden pressure increases, giving additional capacity to the footing. There is also shear around the perimeter of the footing; the soil outside the footing area does not add to the capacity. Additional load is required to continue the footing movement.

Local shear failure [Fig. 17 (b)] is a combination of both general and punching shear failures. The failure surface does not extend to the surface but ends somewhere within the bearing soil. As the footing displaces vertically in the soil, the embedment provides additional strength and thus additional load is required for vertical movement.
FIG. 17. Modes of Bearing Capacity Failure and Corresponding Load-Settlement Curves Beneath Spread Footings Overlying Chattahoochee Sand: (a) General Shear Failure; (b) Local Shear Failure; and (c) Punching Shear Failure (modified from Vesic 1963)
There are no set criteria to determine which failure mode will govern, but some general guidelines from Vesic (1973) can be used:

1. Footings in clay are typically governed by general shear failure.
2. Sands with a relative density ($D_r$) greater than about 67% are typically governed by general shear failure (Fig. 18).
3. Footings on loose to medium dense sands (30% < $D_r$ < 67%) are probably governed by local shear (Fig. 18).
4. Footings on very loose sand ($D_r < 30\%$) are probably governed by punching shear (Fig. 18).

Terzaghi, Meyerhof, and Hansen’s equations for $q_{ult}$ (Eqs. 29, 30, 32) are valid only for general shear failure. If the failure is a local shear failure, then modifications are needed. Terzaghi proposed that the following reduced strength parameters be used in general shear equations to account for local shear:

$$c'' = 0.67c \quad (51)$$
$$\phi'' = \tan^{-1}(0.67 \tan \phi) \quad (52)$$

Eqs. 51-52 may not be applicable to all soils and conditions. Vesic (1973) suggested that the correction factor, 0.67, in Eqs. 51-52 be based on relative density and equal to ($0.67 + D_r - 0.75 D_r^2$) for the range of $0 \leq D_r \leq 0.67$.

### 2.3.9 Distribution of Foundation Contact Pressures

It is typically assumed that the contact pressures exerted by the footing on the soil are perfectly uniform. However, this is generally not the case. There are several factors that can affect the pressure distribution under a footing, including stiffness of the footing,
FIG. 18. Modes of Failure in Chattahoochee Sand (modified from Vesic 1963)
soil type, magnitude of applied load, and roughness of footing bottom.

Das (1997) discussed in depth the footing-soil interaction. Flexible foundations on saturated clays ($\phi = 0$) (Fig. 19) will deform most at the center of the footing and least at the edges, but the contact stresses will be approximately uniform over the contact area. If the same soil is loaded with a rigid footing, the settlement will be uniform but the stresses in the center of the footing will be much lower than the stresses at the edges.

A flexible foundation on sand (Fig. 20) will have uniform distribution of contact stress but will settle most at the edges owing to a lack of confining pressure. Rigid foundations on the same soil will settle uniformly but the stresses will be largest at the center rather than at the edges.
FIG. 19. Profiles of Settlement and Contact Stress for Foundations Bearing on Unconsolidated Undrained, Saturated Clay (modified from Das 1997)

FIG. 20. Profiles of Settlement and Contact Stress for Foundations Bearing on Sand (modified from Das 1997)
CHAPTER 3
RESEARCH PROGRAM

The primary tests conducted for this research study consisted of compressive tests of a full-scale footing on unreinforced soil, a full-scale footing on soil improved with a Geopier group, a unit cell footing supported by one Geopier element, and a single Geopier element. In addition, a plate load test was conducted on unreinforced soil. The tests were conducted under the northbound bridge of I-15 over South Temple street in Salt Lake City, Utah. To supplement the primary laboratory and field-testing program, preliminary laboratory and field tests were conducted on selected soil samples to estimate shear strengths, consolidation properties, and other index and geotechnical engineering properties.

3.1 SOIL TESTS

Three types of in-situ tests were conducted on the soils at the site. These included the borehole shear test (BST), the cone penetration test and seismic cone penetration test (CPTU and SCPTU), and the dilatometer test (DMT). Laboratory testing included unit weight, soil classification, direct shear (DST), consolidation, stress path, specific gravity, water content, and unconfined compression tests.
3.1.1 In-Situ Tests

Compacted road base was encountered near the ground surface at the site. In addition, the upper layer of native soil consisted of very stiff clay that had been contaminated with oil. Due to these conditions, the upper 0.91 m (3 ft) was excavated with a backhoe. The elevation of the bottom of the excavation was left approximately 150 mm (6 in.) higher than the bearing level of the footings until just prior to placement of the footing, at which time the soil was excavated to grade.

Detailed in-situ tests were performed at the location of the footing on unreinforced soil. BSTs were performed in the locations of improved soil with the use of a hand auger to open the holes. The results from these BSTs were used for comparison of soil strengths at all locations.

Drilling during in-situ sampling and testing was accomplished using a Giddings trailer-mounted drill rig equipped with a hollow stem auger. Shelby tube samples were taken continuously from the surface to a depth of 5 m (16 ft). Starting at a depth of 305 mm (1.0 ft), a BST was conducted every 610 mm (2.0 ft) down the drilled hole created upon removal of a Shelby tube. Additional BSTs were conducted where needed. Table 3 summarizes the BST results. An example of the BST results, consisting of a plot of shear stress ($\tau$) versus effective normal stress ($\sigma'$), is presented in Fig. 21.

Descriptions of the BST are given in Handy and Fox (1967) and Handy (1986). The multistage tests were conducted in a vertical borehole 7.6 cm (3 in.) in diameter and consisted of expanding a shear head into the adjacent soil to generate normal stress (Fig. 22) and then pulling the shear head upward to generate shear stress. Fig. 23 shows the
TABLE 3. Summary of BST Results at Test Footing Locations

<table>
<thead>
<tr>
<th>Depth from Ground Surface (m)</th>
<th>Footing over Unreinforced Soil</th>
<th>Depth from Ground Surface (m)</th>
<th>Footing over Pier Group</th>
<th>Depth from Ground Surface (m)</th>
<th>Unit Cell Footing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>φ' (deg)</td>
<td>c' (kPa)</td>
<td></td>
<td>φ' (deg)</td>
<td>c' (kPa)</td>
</tr>
<tr>
<td>0.28</td>
<td>39</td>
<td>7</td>
<td>0.18</td>
<td>39</td>
<td>3</td>
</tr>
<tr>
<td>0.89</td>
<td>35</td>
<td>7</td>
<td>0.79</td>
<td>42</td>
<td>0</td>
</tr>
<tr>
<td>1.16</td>
<td>27</td>
<td>2</td>
<td>1.40</td>
<td>29</td>
<td>1</td>
</tr>
<tr>
<td>1.50</td>
<td>29</td>
<td>10</td>
<td>2.01</td>
<td>32</td>
<td>1</td>
</tr>
<tr>
<td>1.80</td>
<td>31</td>
<td>0</td>
<td>2.62</td>
<td>39</td>
<td>1</td>
</tr>
<tr>
<td>2.11</td>
<td>27</td>
<td>3</td>
<td>3.23</td>
<td>30</td>
<td>4</td>
</tr>
<tr>
<td>2.72</td>
<td>33</td>
<td>1</td>
<td>3.84</td>
<td>30</td>
<td>0</td>
</tr>
<tr>
<td>3.30</td>
<td>31</td>
<td>0</td>
<td>4.45</td>
<td>38</td>
<td>0</td>
</tr>
<tr>
<td>3.94</td>
<td>33</td>
<td>4</td>
<td>5.06</td>
<td>30</td>
<td>7</td>
</tr>
<tr>
<td>4.55</td>
<td>30</td>
<td>0</td>
<td>5.67</td>
<td>30</td>
<td>7</td>
</tr>
</tbody>
</table>

Note: 1 m = 3.281 ft; 1 kPa = 20.886 psf

FIG. 21. Typical BST Result, Sample of Fat CLAY (CH) at a Depth of 0.89 m (3.0 ft)
FIG. 22. BST Shear Head in Simulated Borehole in the Laboratory  
(photo courtesy of Dr. R. L. Handy, Handy Geotechnical Instruments Inc., Madrid, Iowa)

FIG. 23. BST Test Setup: Shear Base Used to Apply the Shearing Stress (left); Container Box Where the Normal Stress was Controlled (center); and Pore Pressure Transducer Readout Box (right)
BST setup during testing operations. For tests above the groundwater table (GWT), a normal stress was applied and the soil was allowed to consolidate for 5 min. and then the shear head was pulled upward at a constant rate until a maximum shear stress was reached. The shearing stress was relieved and then the normal stress was increased to a higher level and the soil was sheared immediately with no time allowed for consolidation. Every other time a new normal stress was applied, the soil was given 5 min. to consolidate; the other times, it was not. This was done when pore pressures were not recorded to determine if the excess pore pressures owing to the increase in the normal stress had dissipated. If the increases in both the normal stresses and the shearing stresses were not linear, then the test was not a purely drained test. In the tests conducted for this research, there was no distinguishable difference in the results for the unconsolidated test points and the consolidated test points, as can be seen in Fig. 21.

Below the GWT, the soil was allowed to consolidate for 15 min. after application of the initial normal stress. No consolidation time was allowed after application of the second normal stress and then subsequent consolidation times of 5 min. and zero min. were allowed in an alternating pattern. An initial attempt was made to monitor the pore pressures during the test. However, problems arose when the pore water inlet became clogged and thus pore pressures could not be monitored during this phase of the tests. However, additional tests were later conducted during which pore pressures were recorded. The shearing phase was typically performed immediately after the excess pore pressures had dissipated, usually within 1 min. to 2 hrs. Excess pore pressure recordings, plotted as average excess pore pressure versus elapsed time for varying normal stresses,
FIG. 24. Dissipation of Excess Pore Pressure with Time for a BST at a Sample Depth of 2.72 m (9.0 ft)
are presented in Fig. 24 for a test below the GWT at a depth below the ground surface of 2.72 m (9 ft). It is noted that only with the first application of normal stress 10 kPa (209 psf) was there a marked increase in pore pressures of 1.35 kPa (28 psf). This excess pore pressure dissipated in less than 9 min. for the first application of normal stress and less than 5 min. for subsequent applications of normal stress. Thus, the results from tests conducted without pore pressure measurements were essentially drained tests.

CPTs were performed by Contec. A cone penetration test with pore pressure readings (CPTU) was attempted at each test site; however, not all the results were consistent with results of other CPT tests conducted in the area. CPTU-2 was conducted next to the FSFOUS. No sleeve friction was recorded for the first 2 m (6.6 ft) of sounding at this location, which was an important location to determine the engineering properties of the soil (see Appendix F). A seismic cone penetration test (SCPT) was later performed at the site next to the FSFOUS. The results of this test are presented graphically in Fig. 25 and are consistent with historical data.

Dilatometer testing (DMT) was performed by Contec next to the full-scale footing bearing on unreinforced soil prior to testing it and prior to the installation of the piers. The results of this test are presented in Table 4. Overconsolidation ratios (OCR) and lateral stress indices (K₀) were determined using the interpretation method developed by Marchetti (1980).

3.1.2 Laboratory Tests

Relatively undisturbed soil samples for laboratory testing were collected during field operations using thin-walled Shelby tubes with total lengths of 923 mm (36 in.),
FIG. 25. SCPT Results, Surface Elevation at 1288.283 m (4226.86 ft), GWT at 1.86 m (6.1 ft)
TABLE 4. Values of OCR and $K_0$ Determined from Results of Dilatometer Tests

<table>
<thead>
<tr>
<th>Depth from Ground Surface (m)</th>
<th>OCR</th>
<th>$K_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>(2)</td>
<td>(3)</td>
</tr>
<tr>
<td>0.30</td>
<td>21.859</td>
<td>2.300</td>
</tr>
<tr>
<td>0.65</td>
<td>8.772</td>
<td>1.675</td>
</tr>
<tr>
<td>1.00</td>
<td>4.994</td>
<td>1.258</td>
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<tr>
<td>1.30</td>
<td>2.468</td>
<td>0.868</td>
</tr>
<tr>
<td>1.65</td>
<td>7.707</td>
<td>1.238</td>
</tr>
<tr>
<td>2.00</td>
<td>5.431</td>
<td>0.864</td>
</tr>
<tr>
<td>2.30</td>
<td>7.678</td>
<td>1.199</td>
</tr>
<tr>
<td>2.65</td>
<td>11.819</td>
<td>1.253</td>
</tr>
<tr>
<td>3.00</td>
<td>8.610</td>
<td>1.366</td>
</tr>
<tr>
<td>3.30</td>
<td>8.209</td>
<td>1.214</td>
</tr>
<tr>
<td>3.65</td>
<td>6.806</td>
<td>1.085</td>
</tr>
<tr>
<td>4.00</td>
<td>6.258</td>
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</tr>
<tr>
<td>4.30</td>
<td>6.840</td>
<td>1.095</td>
</tr>
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<td>4.65</td>
<td>3.386</td>
<td>1.053</td>
</tr>
<tr>
<td>5.00</td>
<td>3.574</td>
<td>0.898</td>
</tr>
</tbody>
</table>

Note: 1 m = 3.281 ft
inside diameters of 71 mm (2.8 in.), and sample recovery lengths of 610 mm (24 in). Laboratory tests were conducted in the Graduate Geotechnical Laboratory at the University of Utah. Consolidation and direct shear samples were obtained from the central portion of the sample tubes at depths corresponding to the depths at which the BSTs were conducted. The tubes were cut 25 mm (1 in.) above and below the sample using a hand held band saw to limit sample disturbance. The soil samples were removed from the tubes with a hand crank extruder, in the same direction as the soil entered the tube. The samples were then visually classified by grain size, color, and consistency (cohesive soils) or relative density (granular soils). Consolidation and direct shear samples, approximately 25 mm (1 in.) in height, were then trimmed directly into the specimen rings, and placed immediately into the testing device. Unit weight, water content, Atterberg limits, sieve, and hydrometer tests were completed on the remainder of the Shelby tube samples.

Whereas the BST involves vertical shearing of the soil in-situ, the direct shear test (DST) shears the sample horizontally. Multiple multistage direct shear tests (DSTM) were conducted to show the repeatability of the test on soil layers from either directly above or directly below the first test sample. Single stage DSTs were conducted on selected samples to compare results with multistage DSTs and BSTs. The single-stage tests were combinations of single stage test and multistage tests (DSTSM), performed as follows: (1) Samples were obtained from the Shelby tubes and divided into three 25 mm (1 in.) layers; (2) the center layer was tested in multiple stages; (3) the top layer was tested in a single stage with a normal stress higher than the first stage of the multistage
TABLE 5. Comparison of Strength Parameters Obtained from BST, DSTM, and DSTSM

<table>
<thead>
<tr>
<th>Depth From Ground Surface (m) (1)</th>
<th>BST</th>
<th>DSTM</th>
<th>DSTM&lt;sup&gt;a&lt;/sup&gt;</th>
<th>DSTM</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>φ'</td>
<td>c'</td>
<td>φ'</td>
<td>c'</td>
</tr>
<tr>
<td></td>
<td>(deg)</td>
<td>(kPa)</td>
<td>(deg)</td>
<td>(kPa)</td>
</tr>
<tr>
<td>0.28</td>
<td>39</td>
<td>7</td>
<td>27</td>
<td>34</td>
</tr>
<tr>
<td>0.89</td>
<td>35</td>
<td>7</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>1.16</td>
<td>27</td>
<td>2</td>
<td>30</td>
<td>56</td>
</tr>
<tr>
<td>1.50</td>
<td>29</td>
<td>10</td>
<td>36</td>
<td>18</td>
</tr>
<tr>
<td>1.80</td>
<td>31</td>
<td>0</td>
<td>41</td>
<td>4</td>
</tr>
<tr>
<td>2.11</td>
<td>27</td>
<td>3</td>
<td>34</td>
<td>7</td>
</tr>
<tr>
<td>2.72</td>
<td>33</td>
<td>1</td>
<td>29</td>
<td>15</td>
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<td>3.30</td>
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<td>32</td>
<td>0</td>
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<td>3.94</td>
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<td>3</td>
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<td>2</td>
</tr>
<tr>
<td>4.55</td>
<td>30</td>
<td>0</td>
<td>27</td>
<td>9</td>
</tr>
</tbody>
</table>

<sup>a</sup>Additional DSTM test conducted

Note: 1 m = 3.281 ft; 1 kPa = 20.886 psf

FIG. 26. Comparison of BST, DSTM, and DSTSM Strength Data for Lean CLAY (CL) from a Depth of 2.72 m (9 ft)
test; (4) the lower layer was tested in a single stage with a normal stress higher than for
the top layer. Results from single stage tests consisted of data from the first stage of the
multistage test and data from the single stage tests. Table 5 summarizes the shear
strength test results. A comparison plot of BST, DSTM, and DSTSM is shown in Fig.
26. Significant differences in results from BSTs and DSTs at the same depths are likely
due to anisotropy of the soil with respect to strength. Differences also resulted from
nominally identical samples that were not truly identical owing to thin layering of the in-
situ soils.

Stress path testing can be used to predict all components of settlement by
modeling changes in stress, drainage, and moisture that are expected to occur in the field
(Lambre and Whitman 1969). Stress path tests were conducted in this research program
on saturated specimens from below the GWT using the following procedure:

1. The subsoil conditions were established and soil layers contributing to settlement
were identified.

2. Initial in-situ stress conditions at each identified test location were determined.

3. Changes in total stress at each test location from the footing loads were estimated
using elastic theory and uniform contact stresses applied to the ground surface
over a square area.

4. The conditions from steps 2 and 3 were duplicated in a laboratory test. After
application of the initial in-situ stress conditions with the drainage valves open,
the specimen was permitted to come to equilibrium. The drainage valves were
then closed, the changes in total stress were applied, and the vertical deformation
resulting from immediate settlement was measured. The drainage valves were the
opened and the vertical deformation resulting from primary consolidation was measured with time until equilibrium was established.

5. The deformations measured in step 4 were used to calculate vertical strains, from which in-situ settlements were estimated. Undrained strains were used to estimate immediate settlement \((S_i)\) and drained strains were used to estimate primary consolidation settlement \((S_c)\).

6. The time required for the excess pore pressures generated during application of the undrained loads to dissipate during drainage were determined and used to estimate the time rate of \(S_c\).

Similar procedures were used for partially saturated specimens from above the GWT except that drained conditions were maintained throughout the tests. The vertical strains from these tests were used to estimate \(S_i\) plus \(S_c\) in the field tests.

The initial vertical effective stress \((\sigma'_v0)\) was estimated for each layer using the measured unit weight. Based on the results of the DMT, the horizontal stresses \((\sigma'_h0)\) were calculated with an average \(K_o\) value of 1.0, see Table 4, multiplied by \(\sigma'_v0\). The change in surface stress \((\Delta q)\) that was to occur would affect both vertical \((\Delta \sigma_v)\) and horizontal \((\Delta \sigma_h)\) changes in stress with depth. The following equations were used to estimate these changes in stress (Das 1997):

\[
\Delta \sigma_{v,z} = \Delta q \left[ 1 - \frac{z^3}{(b^2 + z^2)^{3/2}} \right] \quad (53)
\]

\[
\Delta \sigma_{h,z} = \frac{\Delta q}{2} \left[ 1 + 2\mu - \frac{2(1+\mu)z}{(b^2 + z^2)^{3/2}} + \frac{z^3}{(b^2 + z^2)^{3/2}} \right] \quad (54)
\]
where \( z \) = depth below footing to sample location

\[ b = \text{radius of footing} \]

\( \mu = \text{Poisson's ratio} \)

Eqs. 53-54 are valid to calculate \( \Delta \sigma_v \) and \( \Delta \sigma_h \) at depth \( z \) beneath the center of a uniform stress applied over a circular area to the surface of a homogeneous, isotropic, linearly elastic half space. An equation similar to Eq. 53 is available to calculate \( \Delta \sigma_v \) for a uniform stress applied over a rectangular area for the same conditions, which would be more appropriate for the square footings tested in this research program. However, no equation similar to Eq. 54 is available to calculate \( \Delta \sigma_h \) for a rectangular loaded area. Therefore it was decided to use Eqs. 53-54 to calculate \( \Delta \sigma_v \) and \( \Delta \sigma_h \) by converting the square footing to an equivalent circle based on area. In this manner, consistent estimates of \( \Delta \sigma_v \) and \( \Delta \sigma_h \) were obtained. Fig. 27 shows the triaxial apparatus setup for the stress path tests. If the sample came from below the GWT, the specimen was saturated using triaxial saturation techniques described by Head (1988).

Area corrections based on strains were required so the samples had to be monitored continuously. The tests were stress controlled, so that as the strains increased, the effective area of the sample increased, the vertical stress decreased, and the vertical load (force) was increased to maintain a constant vertical stress. The specimens were 73 mm (2.85 in.) in diameter and 76 mm (3.0 in.) tall. The relatively short specimen height compared to standard triaxial strength tests was used to decrease the time required to complete primary consolidation for each loading increment. Drainage was permitted only at the top of the specimen so that a constant backpressure could be maintained at the bottom.
A stress path test is typically conducted for the estimated contact stress from the design load of a footing. As the loads for the full-scale tests were expected to vary over a wide range, the stress path tests in this research were conducted at several loading states to cover the expected range of induced contact stress ($\Delta q$) for the footings. Each strain-settlement curve plotted using data from the stress path tests (Fig. 27) included only immediate and primary consolidation settlement. However, secondary consolidation could have been monitored, but was not required for this research study. The vertical stresses at depth were equated to a single induced surface load that would cause the stress (Fig. 28). This was done so that, for a given induced surface stress, the strain for each layer could be determined and the total predicted immediate plus primary consolidation settlement ($S_i + S_c$) could be calculated.

### 3.1.3 Matrix Soil Profile

The soils at the site consist of alluvial deposits underlain by Bonneville Clay. The upper 5.75 m (19 ft) are recent alluvial deposits from the many streams that come from the nearby canyons. The City Creek stream traveled close to the site before it was placed in underground pipes in 1905 A.D. Table 6 summarizes the general properties of the fourteen layers identified at the site. These properties include angle of internal friction ($\phi'$) and cohesion intercept ($c'$), measured from BSTs; undrained shear strength ($s_u$) from unconfined compression tests, unit weight ($\gamma$), natural water content ($w_n$), and plasticity index ($I_p$) determined from Shelby Tube samples; average tip resistance ($q_c$) from CPTs; overconsolidation ratio ($OCR$) determined from DTM results; and Young’s Modulus ($E_s$) estimated from CPT $q_c$. $E_s$ was determined using Eq. 4 with a constant of 6 for clays and
FIG. 27. Schematic Diagram of Setup for Stress Path Test
FIG. 28. Results from Stress Path Test with the Stresses at Depth Converted to Induced Surface Stress
<table>
<thead>
<tr>
<th>Layer No. (1)</th>
<th>Depth From Ground Surface</th>
<th>USCS Soil Classification (4)</th>
<th>Soil Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>To Top of Layer (m) (2)</td>
<td>To Bottom of Layer (m) (3)</td>
<td>γ (kN/m^3) (5)</td>
</tr>
<tr>
<td>1</td>
<td>0.00</td>
<td>0.28</td>
<td>MH, Sandy Elastic SILT</td>
</tr>
<tr>
<td>2</td>
<td>0.28</td>
<td>0.84</td>
<td>ML, SILT</td>
</tr>
<tr>
<td>3</td>
<td>0.84</td>
<td>1.55</td>
<td>CH, Fat CLAY</td>
</tr>
<tr>
<td>4</td>
<td>1.55</td>
<td>1.80</td>
<td>CL, Sandy Lean CLAY</td>
</tr>
<tr>
<td>5</td>
<td>1.80</td>
<td>2.19</td>
<td>SM, Silty SAND</td>
</tr>
<tr>
<td>6</td>
<td>2.19</td>
<td>2.59</td>
<td>CL-ML, Sandy Silty CLAY</td>
</tr>
<tr>
<td>7</td>
<td>2.59</td>
<td>3.02</td>
<td>CL, Lean CLAY</td>
</tr>
<tr>
<td>8</td>
<td>3.02</td>
<td>3.30</td>
<td>ML, SILT with Sand</td>
</tr>
<tr>
<td>9</td>
<td>3.30</td>
<td>3.57</td>
<td>CL, Sandy Lean CLAY</td>
</tr>
<tr>
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<td>3.57</td>
<td>3.80</td>
<td>ML, SILT</td>
</tr>
<tr>
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<td>3.80</td>
<td>3.99</td>
<td>CL, Lean CLAY</td>
</tr>
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<td>3.99</td>
<td>4.24</td>
<td>ML, SILT</td>
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<td>4.75</td>
<td>CL, Lean CLAY</td>
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<tr>
<td>14</td>
<td>4.75</td>
<td>4.85</td>
<td>CL, Sandy Lean CLAY</td>
</tr>
</tbody>
</table>

aFrom CPT data using Eqs. 4 and 5
bNP=Non-Plastic

Note: 1 m = 3.281 ft; 1 kN/m^3 = 6.366 pcf, 1 kPa = 20.886 psf
3 for silts and sands, then corrected for overconsolidation using Eq. 5. The GWT depth at time of primary testing was 1.86 m (6.1 ft). Table 7 presents the consolidation properties of the fourteen soil layers, including effective preconsolidation stress ($\sigma'_p$), virgin compression index ($C_v$), recompression index ($C_r$), and initial void ratio ($e_0$). A representative CPT log for the site is presented in Fig. 25.

### 3.1.4 Geopier Materials

The Geopier elements were constructed with three types of aggregate. 75 to 150 mm (3 to 6 in.) cobble rock was used below the GWT on the 914 mm (36 in.) diameter reaction piers. 20 to 75 mm (0.75 to 3 in.) gravel was used for the test piers below the GWT. A sieve analysis was performed on the gravel with the results presented graphically in Fig. 29. The USCS classification for this smaller cobble rock is GW, well graded gravel. The aggregate used above the GWT for all piers was crushed rock used in road construction, typically called road base, with a USCS classification of SW-SM, well graded sand with silt and gravel. Fig. 30 shows the particle size distribution of the road base, with particle size varying from less than 0.1 mm to as great as 19 mm (0.004 in. to 0.75 in.).

Previous tests performed at this site have shown that the road base compacts to an average density of 21.7 kN/m$^3$ (138 pcf) with an effective unit weight of 11.9 kN/m$^3$ (76 pcf) (Hsu 2000). Standard and modified Proctor compaction tests were performed on the road base material, and the results are plotted in Fig. 31. Previous correlations with results of CPTs indicated an approximate angle of internal friction of 50° for the road base above the GWT and 47° for the cobble rock below (Lawton 1999).
### TABLE 7. Consolidation Properties of Matrix Soil Layers

<table>
<thead>
<tr>
<th>Layer No. (1)</th>
<th>Depth From Ground Surface</th>
<th>OCR DTM(^a) (4)</th>
<th>OCR Consolidation Test(^b) (5)</th>
<th>(\sigma'_p) (kPa) (6)</th>
<th>(C_c) (7)</th>
<th>(C_r) (8)</th>
<th>(e_0) (9)</th>
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<td>0.00</td>
<td>0.28</td>
<td>22</td>
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<tr>
<td>2</td>
<td>0.28</td>
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<td>10</td>
<td>46</td>
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<td>3</td>
<td>0.84</td>
<td>1.55</td>
<td>4</td>
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<td>7</td>
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<td>2.19</td>
<td>3</td>
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<td>--</td>
</tr>
<tr>
<td>6</td>
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<td>2.59</td>
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</tr>
</tbody>
</table>

\(^a\)OCR DTM = OCR determined from results of dilatometer test

\(^b\)OCR Consolidation test = OCR determined from results of consolidation test

Note: 1 m = 3.281 ft; 1 kPa = 20.885 psf
FIG. 29. Particle Size Distribution for Gravel Used in the Construction of Test Piers Below the GWT

FIG. 30. Particle Size Distribution of Road Based Used in Construction of Test Piers above the GWT
Moisture Content (%)

Dry Density (kN/m³)

FIG. 31. Moisture-Density Relationships for Road Base Used in Construction of Piers Above the GWT

1 kN/m³ = 6.366 pcf
3.2 PRIMARY FIELD TESTS

The primary field tests consisted of compressive tests on a full-scale footing on unreinforced soil and a footing on soil reinforced with a Geopier group (Table 8 and Fig. 32). As used in this paper, the term “full-scale” refers to tests in which a 1.981 m by 1.981 m (6.5 ft by 6.5 ft) square footing was employed. To provide results for comparison with those from the full-scale tests, compressive tests were conducted on a “unit cell footing” and a single pier. The unit cell footing was used to predict the behavior of the full-scale footing over a pier group by using one pier and one-fifth the footing area. Also, the single pier load test was used to predict the behavior of the full-scale pier group footing. Finally, a plate load test, consisting of a 610 mm (24 in.) diameter circular plate over unreinforced soil was conducted to predict the behavior of the full-scale footings. Table 8 and Fig. 32 also show the location uplift tests that were a part of this overall research program but are discussed in Singh (2004).

<table>
<thead>
<tr>
<th>Location No. (1)</th>
<th>Description (2)</th>
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<tbody>
<tr>
<td>1</td>
<td>Full Scale Unreinforced Compression Test</td>
</tr>
<tr>
<td>2</td>
<td>Full Scale Pier Group Compression Test</td>
</tr>
<tr>
<td>3</td>
<td>Unit Cell Compression Test</td>
</tr>
<tr>
<td>4</td>
<td>Single 0.61 m (2 ft) Pier Compression Test</td>
</tr>
<tr>
<td>5</td>
<td>Single 0.61 m (2 ft) Drilled Shaft Compression Test</td>
</tr>
<tr>
<td>6</td>
<td>Plate Load Test using 0.61 m (2 ft) Steel Plate on unreinforced soil</td>
</tr>
<tr>
<td>7</td>
<td>Single 0.61 m (2 ft) Drilled Shaft Uplift Test</td>
</tr>
<tr>
<td>8</td>
<td>Single 0.61 m (2 ft) Pier Uplift Test with Internal Pressure Plates</td>
</tr>
<tr>
<td>9</td>
<td>Single 0.61 m (2 ft) Pier Uplift Test without Internal Pressure Plates</td>
</tr>
<tr>
<td>10</td>
<td>Full Scale Pier Group Uplift Test</td>
</tr>
</tbody>
</table>
FIG. 32. Layout of Field Tests
3.2.1 Testing Procedures for Full-Scale Tests

Based on the results of a preliminary analysis, it was determined that eight uplift reaction piers were required to test the full-scale footing bearing on unreinforced soil. These reaction piers were located on the north and south side of the footing. Twelve uplift reaction piers were used for the full-scale pier group footing. All uplift reaction piers were 910 mm (36 in.) in diameter and 4.57 m (15 ft) in height. A steel uplift plate 25.4 mm (1.0 in.) in thickness and 864 mm (34 in.) in diameter was located at the bottom of each pier. Four vertical threaded steel bars were connected to the plate to transfer the surface uplift force to the bottom of the pier. The uplift bars were either #8 Grade 75 Williams bars or #7 Grade 75 Dywidag bars. To prevent the bars from vibrating loose during installation of the piers, the nuts that connected the bars to the plate were welded to the plate with a short 6 mm (1/4 in.) filet weld, then the bar was welded to the nut.

Figs. 33-34 show the test setup for the full-scale footings. The main parts of the load frame consisted of two 8.5 m (28 ft) long plate girders, two columns 3.66 m (12 ft) in height used to connect the two plate girders to the secondary reaction beams, and a single 0.9 m (3.0 ft) bearing beam. These items, along with load cells, hydraulic jacks, and pumps, were borrowed from Brigham Young University’s Civil Engineering Department for use in this research study. A secondary reaction beam was designed to connect the piers to the load frame. The secondary reaction beams were also used as the main reaction beams in the single pier tests to be described later. The secondary reaction beam was connected to the piers by steel double channels that were welded to the beam. The steel threaded bars from the piers went between the double channels and were then
FIG. 33. Setup of Load Frame for Full-Scale Compression Tests:
(a) Front View of Load Frame Setup; (b) Side View of Jack Setup; and (c) View of Secondary Reaction Beam Setup
FIG. 34. Photograph of Setup for Full-Scale Compressive Test
bolted to the double channels. To limit the possibility of buckling of the secondary reaction beam, a 13 mm (0.50 in.) flat bar was cut to fit between the uplift reaction bars and then welded to the bottom of the beam. Six Grade 150 threaded bars with two nuts were used to connect each column of the main load frame to the secondary reaction beam. All welds that connected the double channel and flat bar to the beam were cut at the end of each test and re-welded prior to placement of the load frame at the new location. A Lincoln Sam 400 arc welder was used for welding.

During testing of the pier group footing, two bars within one reaction pier broke at the nut. It was noted that the tack weld that connected the bar to the nut on these were above the plate and nut. For future purposes, it is recommended that the bars not be welded, but rather be connected with a double nut. If used, the weld should be placed below the plate so that it does not significantly weaken the bar as was the case for the two broken bars in this research program.

The two jacks that were used had maximum load capacities of 4,400 kN (500 ton) each with a 330 mm (13 in.) stroke. The jacks sat upon a steel bearing plate that was 76 mm (3 in.) in thickness, 914 mm (3 ft) in length, and 610 mm (2 ft) in width. High strength grout was placed between the footing concrete and the steel bearing plate to level the plate and to provide good contact between the plate and the top of the concrete footing. The jacks were powered by a single hydraulic pump. Load cells were used to monitor the induced load. The tests were stress-controlled. A constant force was maintained by activating the pump as needed.
Articulated bearing plates were placed on top of the jacks. The load cells were placed on the bearing plates. The jacks pushed the load cells against a bearing beam that in turn reacted against the two main girders. The load cells were calibrated before and after the field-testing. The computer data acquisition was achieved using a Validyne MC170-32-A1 scanner, with software LabTech Note-Book Pro controlling the data collection. The calibration input factors to convert the measured voltage to load during testing were computed using a linear equation to allow control of loads during the tests. However, the actual calibration curves were non-linear but were not used during the tests because the data collection software would not allow it. During data reduction after the tests were completed, the loads were converted back to voltages using the linear equations and then reconverted to the correct loads using the non-linear equations.

String pull displacement transducers (DT) from Celesco were used to monitor movements of the footing (Fig. 35). All DTs were connected to independent reference frames. The DTs contained a wire that was approximately 1,270 mm (50 in.) in length. The poles for the independent reference frames were monitored with DTs to ensure proper measurements of movements of the footing. There was no movement of the posts during the tests, verifying that they were located far enough from the footings to provide an independent reference. The DTs were placed as far as possible from the connection point. The DTs were calibrated prior to field-testing. For each test, four DTs were placed at each corner of the footing to measure vertical displacement. In addition, horizontal movement was recorded in two perpendicular directions. The 3D displacements were recorded to account for horizontal sliding of the footing and to convert the measured vertical and horizontal movements to true vertical movement. The
FIG. 35. Location of DTs and Possible Footing Movement:
(a) Plan View; and (b) Elevation View
method and equations used to correct the movements are presented in Appendix G. After correction, the four vertical DT values were used to obtain an average value. During one test a DT malfunctioned. Geometry and an assumption that the footing was rigid were used to estimate the movement of the malfunctioned DT to obtain a correct average footing movement.

Vibrating wire pressure plates (PP) from Geokon, with varying capacities, were used to monitor footing contact pressure and induced vertical stress at selected locations within the piers. A Geokon scanner was used to gather the data from all vibrating wire instruments and the computer program Micro-10 was used to control the data acquisition. To measure footing contact pressures, a thin layer of sand was placed on the surface of the soil at each location, the PPs were placed on the sand, and the footing concrete was poured on top of them. In the test on the pier group, PPs were placed on top of each pier and on the adjacent matrix soil to enable determination of the stress concentration ratios \(R_s\). In addition, PPs were located in selected locations within the piers to measure the vertical stress dissipation through the pier.

Soil Instruments spade-type push-in pressure cells (PIC) were used to monitor changes in horizontal stresses at selected locations. Four PICs were placed laterally between the PPs at selected depths for the full-scale footing over unreinforced soil (Fig. 36). Two PICs were installed laterally between the PPs for the group pier footing test (Fig. 37). However, during installation of one of the piers for the group test, one PIC that was located in a sand layer became dislodged and was removed (not shown in Fig. 37).

Plastic vertical inclinometer casing, 70 mm (2.75 in.) in diameter, was installed next to each full-scale footing one month before testing to measure horizontal ground
FIG. 36. Layout of Pressure Plates and Push-In-Cells for Full-Scale Footing on Unreinforced Soil: (a) Plan View at Bearing Level; and (b) Elevation View
FIG. 37. Layout of Pressure Plates and Push-In-Cells for Pier Group Footing: (a) Plan View at Bearing Level; and (b) Elevation View
movements at depth during testing. Each casing was installed to a depth of 9.1 m (30 ft) below the ground surface, centered on the side of the footing and placed 0.30 m (1 ft) away from the vertical face of the footing (see Figs. 36-37). The installation process was modified as suggested by Slope Indicator Co. and is described below.

1. A vertical hole was drilled with a hollow stem auger using a Giddings trailer mounted drill rig. A wooden block was cut to the size and shape of the auger and inserted in the hollow stem to keep soil from entering.

2. When the auger reached the desired depth, the auger was lifted about 75 mm (3 in.) and the inclinometer casing was inserted in the hollow stem of the auger.

3. Water was added in the auger stem so the water level was at least above the GWT. This was done to prevent the inflow of water when the plug was removed, which could cause loose soil to migrate into the bottom of the bore hole and trap the casing in the auger. The casing was used to knock the wooden plug free at the bottom of the auger.

4. Water was placed in the casing and filled to the top of the casing to counteract any buoyancy of the casing. A small weight was added to the first casing only during installation and the casing settled about 150 mm (6 in.) in the soil layer at the bottom of the hole.

5. The auger was lifted vertically out of the ground to limit uplift of the casing.

6. A ChemGrout pump was used to mix grout and water and pump it into the annular space between the casing and auger hole. The grout mixture used should have been approximately the same strength as the surrounding soil. However, it was impossible to place layers of grout in the hole. Thus, a grout mixture
recommended by Slope Indicator that represented a medium soft soil was used
and consisted of 79% water, 6% bentonite, and 15% cement by weight.

7. The grout was pumped from the bottom of the hole until grout appeared at the top
of the hole.

A Digitilt inclinometer probe by Slope Indicator Co. was used to measure the
horizontal movement for every 0.30 m (1.0 ft) of casing. The data were read using a
Digitilt DataMate reader and hand-recorded. The computer software DMM for
Windows, from Slope Indicator Co., was used to process the hand-recorded data.
DigiPro software was used to graph the data from DMM. Inclinometer measurements
were taken before, during and after each full-scale test.

The compressive load tests were performed in general accordance with ASTM
D1194-72, which is the standard test method for bearing capacity of soil with static load
on spread footings. The procedure states that all loads applied should be maintained for
at least 15 min. The load should not be increased until either the settlement ceases or the
rate of settlement is linear when plotted with settlement on a linear scale and time on a
logarithmic scale. The settlement readings were recorded at varying times through each
loading increment and plotted on a semi-log graph with time on the log scale. Once the
settlement became linear on the semi-log graph and the load had been applied for at least
15 min., the loading was increment stopped and the next loading increment was applied.

The full-scale footing on unreinforced soil was tested first. At the maximum
extension of the jack, the load was still increasing and reached a maximum value of 1,760
kN (396 kips). During the test, there were problems with the hydraulic pump. When the
load was 614 kN (138 kips), there was a loss of pressure to the jacks. The load was taken
off the footing and it took about three hours to fix the pump. After the pump was fixed, the load was increased to 645 kN (145 kips) and the test resumed. During the test at a load of 1,334 kN (300 kips), inclinometer measurements were taken. Another set of inclinometer readings were performed at the maximum test displacement.

The pier group footing test was performed next. The test was run until the maximum jack stroke was reached. Again the load was still increasing at the maximum stroke and the load had reached a maximum value of 3,170 kN (712 kips). A set of inclinometer readings was collected during the test and at the maximum test displacement. No major problems were encountered during this test.

3.2.2 Testing Procedure for Other Primary Field Tests

The setup for the unit cell footing, single pier, and plate load tests were similar. A single uplift reaction pier, 910 mm (36 in.) in diameter and 4.57 m (15 ft) in length, was installed on either side of the test footing. The uplift reaction piers were used for more than one test as shown in Fig. 32.

For this research, the area of the unit cell footing was one-fifth the area of the full-scale footing since there were five piers supporting the full-scale footing. The unit cell footing was 889 mm (2 ft 11 in.) square and 914 mm (3.0 ft) thick. The test pier was the same size as the piers below the full-scale group pier footing, 610 mm (24 in.) in diameter and 2.44 m (8.0 ft) in length. The setup for the test on the unit cell footing is shown in Fig. 38. The beam rested on the uplift reaction bars and on wooden walls. The DT setup was similar to that of the full-scale tests (Fig. 35). The reaction beam rested on the reaction uplift bars and on wooden walls. Vertical and horizontal movements were recorded. The ultimate load reached was 939 kN (211 kips) and was still increasing at the
FIG. 38. Test Setup with Pressure Plate Locations for Unit Cell Footing
maximum vertical displacement.

The test on the single pier was used to measure the subgrade modulus of the pier and to evaluate the confining effect and capacity of the footing. The results were also used for comparison with those from the group footing to determine group effects. The test setup [Fig. 39 (a)] was similar to the unit cell footing setup (Fig. 38). However, during the single pier test, four quadrants of the bearing plates on top of the pier were monitored for vertical movement and only one horizontal DT was used to measure lateral movement.

Initially each bearing plate was a single, 25 mm (1 in.) thick, 610 mm (24 in.) diameter steel plate [Fig. 39 (b)]. This plate was placed directly on top of the pier. A 508 mm (20 in.) diameter pressure plate with 3,450 kPa (500 psi) capacity was placed over the bottom plate, with grout placed between them to provide good contact. Two 25 mm (1 in.) thick, 597 mm (23.5 in.) diameter steel plates were placed on top of the pressure plate. The placement of the additional plates was intended to limit bending of the bearing plates and to make the bearing system more rigid. The 4,400 kN (500 ton) load cell, placed between the jacks and the reaction beam, was used to monitor induced vertical load.

During the first day of testing there was excessive rotation of the bearing plates. The test was continued until it was deemed unsafe to continue, which occurred at a load of about 512 kN (115 kips) and a settlement of about 50 mm (2 in.). The following day the bearing plates were removed and grout was placed on top of the footing [Fig. 39 (c)]. Additional plates were placed on top of the pressure plate. Grout was used to completely surround the pressure plate. Three days later, the test was resumed. The ultimate load
FIG. 39. Setup for Compressive Test on Single Pier: (a) Test Setup with Pressure Plate (PP) Layout; (b) Surface PP Positioning at Start of Test; and (c) Surface PP at End of Test
The plate load test on unreinforced soil was the final primary field test conducted for this research. The easternmost pier on each row of reaction piers for the pier group test was used to provide the reaction force (see Fig. 32). The test setup is shown in Figs. 41-42. During this test, only the induced vertical load and plate movement were recorded.

The span between the center of the piers was 7.62 m (25 ft), the same as the reaction beam, so only two uplift bars from each uplift reaction pier could be used. Unbeknownst at the time of testing, one of the bars on the southern reaction pier had broken at the uplift plate. The other bar broke when the load was increased from 67 kN (15 kips) to 89 kN (20 kips) and the test was stopped. The test was continued using the other two bars from the pier. The ultimate load reached was 156 kN (35 kips).

3.2.3 Primary Results from Field Tests on Unreinforced Soil

This section includes a comparison of predicted and measured $q_{ult}$, a comparison of predicted and measured settlement, and a discussion of group effects. Values of subgrade modulus obtained from plate load test results and evaluation of changes in stress are also presented.

3.2.3.1 Load-Settlement Relationships and Bearing Capacity

The results of the tests are presented in load-settlement graphs, which show the amount of settlement that a footing experienced for a given load. For this research, the settlement of the footings is $S_i + S_c$. The loads during each test were maintained as required in ASTM D 1194-72, which was discussed earlier. Analysis of all loading data
FIG. 40. Photograph of Setup for Test on Single Geopier Element
FIG. 41. Schematic Diagram of Setup for Plate Load Test on Unreinforced Soil

1 m = 3.281 ft
1 kN = 0.225 kips

FIG. 42. Photograph of Setup for Plate Load Test on Unreinforced Soil
was required to determine when each component of settlement occurred. $S_i$ occurred when the next load increment was first applied, and ended when the required load was achieved, at which time $S_c$ began. The end of $S_c$ for each load increment was determined by Casagrande’s method (Bowles 1996), the same method used in consolidation testing. Time-settlement data was graphed semi-log and completion of $S_c$ determined for each loading increment.

Table 9 summarizes the measured range of ultimate bearing capacity for the full-scale footing on unreinforced soil (FSFOUS) and the plate load test on unreinforced soil. The load-settlement curve and log load versus log settlement curve for the full-scale footing with a footing area of 3.93 m$^2$ (42.3 ft$^2$) on unreinforced soil are presented in Figs. 43 and 44 respectively. Similar curves are presented for the plate load test with the area of the load plate equal to 0.29 m$^2$ (3.14 ft$^2$) in Figs. 45 and 46. It is noted that as the full-scale test on this unreinforced footing continued, the capacity of the footing increased up to the maximum vertical displacement.

Comparing the load-settlement curves from each test with the curves in Fig.17, it

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<th>Method of Analysis (3)</th>
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<th>(5)</th>
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<td>Ultimate Load (kN)</td>
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<td>Ultimate Bearing Capacity (kPa)</td>
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<tr>
<td></td>
<td>Ultimate Bearing Capacity (kPa)</td>
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</tbody>
</table>

Note: 1 mm = 0.039 in.; 1 kN = 0.225 kips; 1 kPa = 0.021 ksf
FIG. 43. Load-Settlement Curve for FSFOUS

FIG. 44. Log of Normalized Settlement vs. Log of Normalized Load for FSFOUS
FIG. 45. Load-Settlement Curve for Plate Load Test on Unreinforced Soil

FIG. 46. Log of Normalized Settlement vs. Log of Normalized Load for Plate Load Test on Unreinforced Soil
appears that the failure may have been a local shear or punching shear failure. Another possible failure mode is “squeezing”, a special type of local shear failure. Values of theoretical \( q_{ult} \) using Terzaghi, Meyerhof, and Hansen equations for the unreinforced full-scale test and plate load test for general shear are presented in Table 10 (a). Also given are values of \( q_{ult} \) using Terzaghi’s reduced strength parameters for local shear failure [Table 10 (b)] ignoring \( c' \), and with \( c' \) equal to 5 kPa (104 psf) [(Table 10 (c)]. The measured range of \( q_{ult} \) values is also presented. The values of \( \phi' \) and \( c' \) in Table 10 are weighted average values of strength parameters calculated using Eqs. 24 and 25. The height of influence extends from the bottom of the footing to a depth dictated by Eq. 27, which is roughly equal to the width of the footing. The results in Table 10 (a) indicate that for the full-scale unreinforced footing, Hansen’s equation, assuming general shear failure, over predicts \( q_{ult} \) by a range of 88% to 119%, Meyerhof’s equation over predicts \( q_{ult} \) by a range of 358% to 434%, with values of \( q_{ult} \) predicted from Terzaghi’s equation between the values from Meyerhof and Hansen. Hansen’s equation under predicts \( q_{ult} \) for the plate load test by 176% and Meyerhof’s equation under predicts by 5%. Assuming local shear failure [Table 10 (b)], Hansen’s equation under predicts \( q_{ult} \) for the full-scale test by a range of 56% to 62%. Meyerhof’s equation also under predicts \( q_{ult} \) by a range of 11% to 23%. In addition, assuming local shear failure, Hansen’s equation under predicts \( q_{ult} \) for the plate load test by 93%, and Meyerhof’s equation under predicts \( q_{ult} \) by 85%.

The average cohesion intercept of 5 kPa (106 psf) was ignored in the afore-mentioned analyses. If this cohesion intercept is included, there is a large increase in \( q_{ult} \). For instance, for the full-scale footing on unreinforced soil with a cohesion intercept of 5 kPa reduced for local shear failure, Hansen’s equation for local shear failure would be
TABLE 10. Bearing Capacity Equation Variables, Theoretical $q_{ult}$ and Measured Range of $q_{ult}$ for Tests on Unreinforced Soil Assuming: (a) General Shear Failure; (b) Local Shear Failure ($c' = 0$ kPa); and (c) Local Shear Failure ($c' = 5$ kPa)

<table>
<thead>
<tr>
<th>Test (1)</th>
<th>$B$ (m) (2)</th>
<th>$\phi'$ (deg.) (3)</th>
<th>$c'$ (kPa) (4)</th>
<th>$\gamma$ (kN/m$^3$) (5)</th>
<th>$\bar{q}$ (kPa) (6)</th>
<th>$D_f$ (m) (7)</th>
<th>Theoretical Values</th>
<th>Measured Range (11)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full-Scale Plate Load</td>
<td>1.98</td>
<td>34</td>
<td>0</td>
<td>17</td>
<td>0</td>
<td>0</td>
<td>417</td>
<td>705</td>
</tr>
<tr>
<td></td>
<td>0.54</td>
<td>37</td>
<td>0</td>
<td>16</td>
<td>0</td>
<td>0</td>
<td>138</td>
<td>323</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test (12)</th>
<th>$B$ (m) (13)</th>
<th>$\phi'$ (deg.) (14)</th>
<th>$c'$ (kPa) (15)</th>
<th>$\gamma$ (kN/m$^3$) (16)</th>
<th>$\bar{q}$ (kPa) (17)</th>
<th>$D_f$ (m) (18)</th>
<th>Theoretical Values</th>
<th>Measured Range (22)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full-Scale Plate Load</td>
<td>1.98</td>
<td>24</td>
<td>0</td>
<td>17</td>
<td>0</td>
<td>0</td>
<td>118</td>
<td>118</td>
</tr>
<tr>
<td></td>
<td>0.54</td>
<td>27</td>
<td>0</td>
<td>16</td>
<td>0</td>
<td>0</td>
<td>36</td>
<td>52</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test (23)</th>
<th>$B$ (m) (24)</th>
<th>$\phi'$ (deg.) (25)</th>
<th>$c'$ (kPa) (26)</th>
<th>$\gamma$ (kN/m$^3$) (27)</th>
<th>$\bar{q}$ (kPa) (28)</th>
<th>$D_f$ (m) (29)</th>
<th>Theoretical Values</th>
<th>Measured Range (33)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full-Scale Plate Load</td>
<td>1.98</td>
<td>24</td>
<td>3</td>
<td>17</td>
<td>0</td>
<td>0</td>
<td>210</td>
<td>204</td>
</tr>
<tr>
<td></td>
<td>0.54</td>
<td>27</td>
<td>3</td>
<td>16</td>
<td>0</td>
<td>0</td>
<td>152</td>
<td>162</td>
</tr>
</tbody>
</table>

Note: 1 m = 3.281 ft; 1 kPa = 20.886 psf; 1 kN/m$^3$ = 6.355pcf
145 kPa (13.5 ksf), overestimating $q_{ult}$ by about 6% or underestimating by 10%.

Values of theoretical $q_{ult}$ for squeezing are presented in columns 6 and 12 of Table 11. Unconfined compression tests were performed in three layers, and the analysis for squeezing was performed in each of these three layers. These layers were not the uppermost layers, thus the induced bearing stress beneath the footing is not the same stress at each lower layer. Values of $q_{ult}$ at depth ($q_{ult, z}$) are from Eq. 34; these values were multiplied by $\eta$ (Eq. 47 substituting $z$ for $h$) to equate $q_{ult, z}$ to $q_{ult}$. Assuming squeezing, $q_{ult}$ for FSFOUS is over-predicted by 101% to 135%, and $q_{ult}$ for the plate load test is over-predicted by 95% to 99%.

Comparison of the theoretical and experimental values of $q_{ult}$ in Tables 10 and 11 indicate that the type of failure for the FSFOUS was probably either local shear or squeezing and for the plate-load test on unreinforced soil was likely general shear failure. The shapes of the load-settlement curves in Figs. 43 and 45 also support these types of failure. The shape of the load-settlement curve for the FSFOUS appears to indicate that a squeezing failure occurred rather than the type of local shear failure described by Vesic [Fig. 17(b)]. If squeezing occurred, the soil within the soft layer(s) would have displaced laterally but would have been increasingly confined by a buildup of displaced and densified soil. The resulting shape of the load-settlement curve would be steep during squeezing followed by a flattening of the curve during confinement by the displaced soil, similar to that shown in Fig. 43.
<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Depth to top of layer (m)</th>
<th>Sw</th>
<th>qult, z (kPa)</th>
<th>η</th>
<th>qult (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.28</td>
<td>72</td>
<td>292</td>
<td>1.30</td>
<td>380</td>
</tr>
<tr>
<td>3</td>
<td>0.84</td>
<td>35</td>
<td>153</td>
<td>2.03</td>
<td>310</td>
</tr>
<tr>
<td>4</td>
<td>1.55</td>
<td>20</td>
<td>106</td>
<td>3.18</td>
<td>337</td>
</tr>
</tbody>
</table>

(b) 

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Depth to top of layer (m)</th>
<th>Sw</th>
<th>qult, z (kPa)</th>
<th>η</th>
<th>qult (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.28</td>
<td>72</td>
<td>292</td>
<td>2.31</td>
<td>675</td>
</tr>
<tr>
<td>3</td>
<td>0.84</td>
<td>35</td>
<td>153</td>
<td>6.53</td>
<td>1,000</td>
</tr>
<tr>
<td>4</td>
<td>1.55</td>
<td>20</td>
<td>106</td>
<td>15.0</td>
<td>1,590</td>
</tr>
</tbody>
</table>

- a Depth to top of layer
- b From Eq. 33
- c Reference Eq. 45 substituting $z$ for $H$

Note: 1 m = 3.281 ft, 1 kPa = 20.886 psf
3.2.3.2 Modulus of Subgrade Reaction of Unreinforced Soil

Values of secant modulus of subgrade reaction of the unreinforced soil ($k_m$) were estimated from the load-settlement curves for both the full-scale test and the plate load test. The results are plotted in Fig. 47. The estimated $k_m$ values for the plate load test were then scaled to predict $k_m$ for the full-scale footing using Terzaghi’s equations (Eqs. 22 and 23b). The first rule is for soil having deformation characteristics that are functions of depth (Type 1 material). The second rule is for material having deformation characteristics that are independent of depth (Type 2 material). The scaled values of $k_m$ are plotted in Fig. 47. Neither scaled curve predicts the behavior of the full-scale footing well. $k_m$ of the full-scale footing decreases rapidly until approximate “failure” of the footing then $k_m$ remains constant.

3.2.3.3 Settlement of Full-Scale Footing on Unreinforced Soil

Settlement of the full-scale footing on unreinforced soil was estimated using theoretical equations and compared to the measured settlement. The engineering properties of the soil layers used for these analyses of settlement are given in Tables 6 and 7. These values were obtained from 1D consolidation, Stress Path, CPT, and DMT tests conducted on specimens obtained to a maximum depth of 4.85 m (15.9 ft) or 2.4 times the width of the FSFOUS.

Table 12 summarizes the predicted immediate settlement, estimated using Schmertmann’s method and Bowles’ Modified Elastic Theory (BMET). Values of $E_s$ used to calculate $S_i$ (see Table 6) were determined from CPT data using Eqs. 4 and 5. The weighted average value of $E_s$ based on layer thicknesses was 16.5 MPa (345 ksf). Poisson’s ratio, $\mu$, was assumed to be 0.4 for all soil layers. The depth of analysis for
FIG. 47. Modulus of Subgrade Reaction for Full-Scale Footing and Plate Load Test on Unreinforced Soil
TABLE 12. Predicted Immediate and Primary Consolidation Settlements for FSFOUS

<table>
<thead>
<tr>
<th>Measured Load and Calculated Contact Stress</th>
<th>Predicted ( S_i ) (mm)</th>
<th>Predicted ( S_c ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F ) (kN)</td>
<td>( \Delta q ) (kPa)</td>
<td>Schmertmann (3)</td>
</tr>
<tr>
<td>----------------</td>
<td>----------------</td>
<td>----------------</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>192</td>
<td>49</td>
<td>3.6</td>
</tr>
<tr>
<td>417</td>
<td>106</td>
<td>8.7</td>
</tr>
<tr>
<td>614</td>
<td>156</td>
<td>13</td>
</tr>
<tr>
<td>646</td>
<td>165</td>
<td>13</td>
</tr>
<tr>
<td>876</td>
<td>223</td>
<td>19</td>
</tr>
<tr>
<td>1,110</td>
<td>282</td>
<td>26</td>
</tr>
<tr>
<td>1,320</td>
<td>336</td>
<td>32</td>
</tr>
<tr>
<td>1,550</td>
<td>395</td>
<td>40</td>
</tr>
<tr>
<td>1,760</td>
<td>448</td>
<td>46</td>
</tr>
</tbody>
</table>

\(^a\)BMET = Bowles’ Modified Elastic Theory

Note: 1 kN = 0.225 kips; 1 kPa = 20.886 psf; 1 mm = 0.039 in.

BMET was 4.85 m (15.9 ft) owing to the presence of a thick, hard layer of sand underlying this depth. The depth of influence for Schmertmann’s method was \( 2B \) or 3.96 m (13.0 ft). It can be seen in Table 12 that the values of \( S_i \) estimated using Schmertmann’s method are greater than the values estimated using BMET. This result is not surprising since it is widely recognized that Schmertmann’s method tends to overestimate immediate settlement compared to other methods and actual immediate settlements.

Primary consolidation settlement was estimated using three different stress distribution methods to predict \( \Delta \sigma_v \) at the midheight of each sublayer (Table 12) - Westergaard, Newmark, and 2:1. The depth of analysis used for \( S_c \) was 4.85 m (15.9 ft). According to the SCPT log (Fig. 24) the soil layer below the depth of 4.85 m is a
relatively thick, very stiff sand layer. This stiff layer would likely distribute the load over a very large area, thereby significantly reducing the magnitude of the induced stresses within the underlying soils, similar to a base course in a pavement system. Therefore, it is likely that the strains within these underlying layers and the resulting settlements would be insignificant and therefore were ignored. Boussinesq derived his equation for a point load on the surface of a semi-infinite, homogeneous, isotropic, linearly elastic material. Newmark integrated this equation for a rectangular area. Since natural soil deposits typically are not homogeneous, Westergaard developed a stress distribution method for layered soils. He used infinitely thin but perfectly rigid layers to represent alternating layers of rigid and compressible soils. These rigid layers were assumed to allow vertical strains but no lateral strains. The 2:1 method is an empirical method that assumes the area of the load increases with depth in a systematic way, thus the stress decreases with depth. Both the Westergaard and Newmark methods are valid for loads applied to the ground surface, which was the case for the footings tested in this research. However, these methods should not be used for embedded foundations. Skopek (1961) and Nishida (1966) developed solutions for embedded loads applied over rectangular and circular areas, respectively, using Boussinesq’s assumptions. These methods are more appropriate for embedded foundations.

The known engineering soil properties are in Table 7; however, not all soil layers were tested for all engineering soil properties. In order to perform complete settlement analysis, pertinent soil properties were required for each soil layer. Regression analyses were conducted on known soil properties to estimate the unknown soil properties. These regression analyses are described in more detail in Appendix D. The predicted properties
from these regression analyses, along with the measured experimental values, are summarized in Table 13.

The equations for $S_c$ (Eqs. 9-11) are for 1-D settlement and therefore modifications are required to correct for 3D effects. Leonards (1976) and Skempton and Bjerrum (1957), as discussed earlier, developed methods to determine $S_c(3D)$ by multiplying $S_c(1D)$ by a factor to account for the effects of 3D strain (Eqs. 12 and 13).

Leonards method uses Eq. 13 and Fig. 7 and is applicable for saturated soils. The GWT was at a depth of 1.86 m (6.10 ft) below the bearing level (ground surface). The degrees of saturation (S) of the soil layers above the GWT were as follows: 75%, 75%, 81%, 89%, and 89% for layers 1 through 5, respectively. These values of S are high enough that the air in the voids was likely trapped as bubbles and thus the soils would

### TABLE 13. Properties of Matrix Soil Layers Used for Analyses of Primary Consolidation Settlement

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Depth From Ground Surface</th>
<th>OCR</th>
<th>$\sigma'_p$ (kPa)</th>
<th>$C_c$ (7)</th>
<th>$C_r$ (8)</th>
<th>$e_0$ (9)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>To Top of Layer</strong> (m)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>To Bottom of Layer</strong> (m)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.00</td>
<td></td>
<td>22</td>
<td>60</td>
<td>0.210</td>
<td>0.002</td>
</tr>
<tr>
<td>2</td>
<td>0.28</td>
<td></td>
<td>20</td>
<td>201</td>
<td>0.240</td>
<td>0.003</td>
</tr>
<tr>
<td>3</td>
<td>0.84</td>
<td></td>
<td>8</td>
<td>172</td>
<td>0.222</td>
<td>0.003</td>
</tr>
<tr>
<td>4</td>
<td>1.55</td>
<td></td>
<td>7</td>
<td>195</td>
<td>0.252</td>
<td>0.003</td>
</tr>
<tr>
<td>5</td>
<td>1.80</td>
<td></td>
<td>3</td>
<td>102</td>
<td>0.180</td>
<td>0.019</td>
</tr>
<tr>
<td>6</td>
<td>2.19</td>
<td></td>
<td>7</td>
<td>266</td>
<td>0.273</td>
<td>0.019</td>
</tr>
<tr>
<td>7</td>
<td>2.59</td>
<td></td>
<td>5</td>
<td>212</td>
<td>0.138</td>
<td>0.019</td>
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<tr>
<td>8</td>
<td>3.02</td>
<td></td>
<td>8</td>
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<td>0.230</td>
<td>0.024</td>
</tr>
<tr>
<td>9</td>
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<td></td>
<td>4</td>
<td>192</td>
<td>0.173</td>
<td>0.007</td>
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<tr>
<td>10</td>
<td>3.57</td>
<td></td>
<td>5</td>
<td>253</td>
<td>0.287</td>
<td>0.007</td>
</tr>
<tr>
<td>11</td>
<td>3.80</td>
<td></td>
<td>2</td>
<td>105</td>
<td>0.146</td>
<td>0.009</td>
</tr>
<tr>
<td>12</td>
<td>3.99</td>
<td></td>
<td>5</td>
<td>273</td>
<td>0.315</td>
<td>0.009</td>
</tr>
<tr>
<td>13</td>
<td>4.24</td>
<td></td>
<td>4</td>
<td>239</td>
<td>0.228</td>
<td>0.025</td>
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<tr>
<td>14</td>
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<td>3</td>
<td>184</td>
<td>0.200</td>
<td>0.025</td>
</tr>
</tbody>
</table>

Note: 1 m = 3.281 ft; 1 kPa = 20.885 psf
respond to changes in load as if they were saturated (Lawton 2001). Therefore, all layers from the ground surface to a depth of 4.85 m (15.9 ft) were assumed as saturated in the theoretical prediction of $S_c$.

The height of the compressible zone, $H$, is 4.85 m (15.9 ft), which gives a value for $H/B$ of 0.41. The values of OCR for the compressible layers mostly range from 3 to 7 (see Table 7), which correspond to moderate levels of overconsolidation. From Table 1, a value of 0.25 is estimated for Skempton’s pore pressure parameter $A$, which corresponds to the borderline of heavily OC sandy clays and OC clays. From Table 2, a value of $\alpha_{cir}$ of 0.30 is determined. Using Eq. 13, a value of $K = 0.48$ is calculated for Skempton and Bjerrum’s method. Using Table 7 for Leonards’ method with a value of $B/H = 2.5$ gives $K = 0.64$ for OCR = 3 and $K = 0.86$ for OCR = 7, with an average value of $K = 0.75$. Averaging the values of $K$ from the two methods gives $K = 0.62$, which was used in all subsequent calculations of $S_c$.

The stress path method was also used to estimate $S_i + S_c$. The soil beneath the footing was conceptually divided into four layers and four laboratory triaxial stress path tests were performed on specimens taken from each layer. The layer thicknesses were 0.55 m (1.80 ft), 0.95 m (3.12 ft), 1.30 m (4.27 ft), and 1.30 m (4.27 ft). The test depths were 0.38 m (1.25 ft), 1.71 m (5.61 ft), 2.43 m (7.97 ft), and 4.01 m (13.2 ft). Estimated induced vertical and horizontal stresses ($\Delta\sigma_v$ and $\Delta\sigma_h$) were calculated for various values of contact stress ($\Delta q$) at the surface. These values of $\Delta\sigma_v$ and $\Delta\sigma_h$ were then applied to the appropriate specimens in a triaxial setup and the corresponding vertical strains determined. The strains determined from the stress path tests were then plotted as a function of $\Delta q$ (see Fig. 28). The stresses from the actual footing test were then used to
predict vertical strains within each layer. The strains were then multiplied by the layer thicknesses to compute $S_i + S_c$.

Values of total settlement ($S_t = S_i + S_c$) calculated using the various methods described above are presented in Fig. 48 and summarized in Table 14. Although values of predicted $S_t$ are shown for induced stresses up to those developed during the test, the settlement methods are valid only for induced stresses less than $q_{ult}$. As indicated in Table 14, there is some variation in predicted settlement values. Each method overestimated the pre-failure settlements by at least several tens of percent. This result is consistent with standard geotechnical practice. The use of BMET for $S_i$ and Westergaard’s theory for $S_c$ predicted values closest to the measured values; the use of BMET and the 2:1 method gave nearly the same values. The use of the stress path method resulted in the largest and therefore worst predictions of settlement. This result is surprising because the stress path method is the most fundamentally sound method if highly undisturbed specimens are obtained and tested and changes in the state of stress within the bearing zone can be predicted reliably. In this case, methods valid for homogeneous soils and alternating compressible-incompressible layers were used to predict induced vertical stresses. In reality, the upper portions of the bearing zone were much stiffer than the underlying portions, likely resulting in induced vertical stresses much lower than those predicted and hence smaller settlements. The undisturbed specimens were obtained by Shelby tube sampling. Perhaps less disturbance would have occurred had the specimens been obtained by piston sampling or another more advanced method for undisturbed sampling.
FIG. 48. Measured and Predicted Total Settlement for FSFOUS Using Four Methods for $S_c$ and Schmertmann and BMET for $S_i$
TABLE 14. Comparison of Measured and Predicted Settlements for FSFOUS Using Four Methods for $S_i$ and Schmertmann and BMET for $S_c$

(a)

<table>
<thead>
<tr>
<th>$F$ (kN)</th>
<th>$S_i$ (mm)</th>
<th>Measured Values</th>
<th>Predicted $S_i = S_i + S_c$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Method 1 (3)</td>
<td>Method 2 (4)</td>
</tr>
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<td>0</td>
<td>0</td>
</tr>
<tr>
<td>192</td>
<td>1.8</td>
<td>6.0</td>
<td>5.4</td>
</tr>
<tr>
<td>417</td>
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<td>18</td>
<td>16</td>
</tr>
<tr>
<td>614</td>
<td>19</td>
<td>27</td>
<td>25</td>
</tr>
<tr>
<td>646</td>
<td>32</td>
<td>28</td>
<td>26</td>
</tr>
<tr>
<td>876</td>
<td>70</td>
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<td>36</td>
</tr>
<tr>
<td>1,106</td>
<td>108</td>
<td>56</td>
<td>45</td>
</tr>
<tr>
<td>1,319</td>
<td>129</td>
<td>72</td>
<td>56</td>
</tr>
<tr>
<td>1,550</td>
<td>157</td>
<td>88</td>
<td>70</td>
</tr>
<tr>
<td>1,760</td>
<td>168</td>
<td>103</td>
<td>83</td>
</tr>
</tbody>
</table>

(b)

<table>
<thead>
<tr>
<th>$F$ (kN)</th>
<th>$S_i$ (mm)</th>
<th>Measured Values</th>
<th>Predicted $S_i = S_i + S_c$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Method 1 (3)</td>
<td>Method 2 (4)</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>192</td>
<td>1.8</td>
<td>5.6</td>
<td>5.0</td>
</tr>
<tr>
<td>417</td>
<td>5.7</td>
<td>17</td>
<td>15</td>
</tr>
<tr>
<td>614</td>
<td>19</td>
<td>24</td>
<td>22</td>
</tr>
<tr>
<td>646</td>
<td>32</td>
<td>26</td>
<td>23</td>
</tr>
<tr>
<td>876</td>
<td>70</td>
<td>35</td>
<td>31</td>
</tr>
<tr>
<td>1,106</td>
<td>108</td>
<td>49</td>
<td>38</td>
</tr>
<tr>
<td>1,319</td>
<td>129</td>
<td>62</td>
<td>46</td>
</tr>
<tr>
<td>1,550</td>
<td>157</td>
<td>75</td>
<td>56</td>
</tr>
<tr>
<td>1,760</td>
<td>168</td>
<td>87</td>
<td>67</td>
</tr>
</tbody>
</table>

Method 1: $S_i = S_i$(Schmertmann or BMET) + $S_c$(Newmark)
Method 2: $S_i = S_i$(Schmertmann or BMET) + $S_c$(Westergaard)
Method 3: $S_i = S_i$(Schmertmann or BMET) + $S_c$(2:1)
Method 4: $S_i = S_i$(Stress Path) + $S_c$(Stress Path)
NA = Not Available
Note: 1 kN = 0.225 kips; 1 mm = 0.039 in.
3.2.3.4 Contact Pressure of Full-Scale Footing on Unreinforced Soil

Contact pressures were measured under the full-scale footing on unreinforced soil with nine pressure plates (see Fig. 36). One plate was placed under the center, one placed under each corner, and one placed under the middle along the edge of each side. The measured induced contact pressures in the three locations are shown in Fig. 49. The measured pressures at the corner of the footing are the largest values for the three locations.

Average values of induced contact pressure at the three locations were compared to determine the distribution of stresses. Fig. 50 shows the contact pressure ratios for the plates along the sides of the footing and at the center of the footing, normalized to the pressure at the corners. Fig. 50 indicates that the pressures along the sides of the footing were only 80% of the pressures at the corners before the ultimate load was reached. After this, there was a gradual increase in pressure along the sides until the side pressures were 95% of the pressure at the corners. The initial pressure at the center was around 38% of the pressure at the corners. This pressure gradually increased until the ultimate load was reached. The trend of larger pressures at the corners and lower at the center of the footing is consistent with the expected contact stress profile for a rigid footing bearing on a clay (see Fig. 19).

3.2.3.5 Horizontal Pressures and Excess Pore Pressures

The changes in horizontal stresses and excess pore pressures were recorded at various depths within the soil underneath the full-scale footing on unreinforced soil (see Fig. 36 for locations). The changes in horizontal stresses and excess pore pressures that developed during the test are plotted in Figs. 51 and 52 and are compared with the load-
FIG. 49. Measured Induced Contact Pressure Compared to Measured Settlement versus Induced Vertical Load for FSFOUS

FIG. 50. Contact Pressure Ratios Compared to Total Settlement versus Induced Vertical Load for FSFOUS
FIG. 51. Measured Settlement and Changes in Horizontal Stress at Various Depths Below the Bottom of the FSFOUS

FIG. 52. Measured Settlement and Excess Pore Pressures at Various Depths Below FSFOUS (GWT = 1.86 m)
settlement curve of the full-scale footing. The GWT location was 1.86 m (6.1 ft). There is relatively little change in horizontal stresses and excess pore pressures until $q_{ult}$ is reached between 518 kN and 605 kN (117 kips to 136 kips). After this, there is a marked increase in excess pore pressures due to shearing, at a depth of 1.32 m (4.33 ft). This depth is 0.54 m (1.77 ft) above than the GWT. This increase in pore pressure is accompanied by a relatively high increase in horizontal stresses at the same depth, indicating a soft layer. This soft layer may be the location of a squeezing failure in the FSFOUS. Two layers in this zone are layers 3 and 4 (Table 6). Layer 3 is classified as CH (Table 4) with relatively low $q_c$ and $s_u$, at depths from 0.84 m (2.76 ft) to 1.55 m (5.09 ft) below the bearing level. Layer 4 is classified as CL with relatively low $q_c$ and $s_u$, with depth of 1.55 m to 1.80 m (5.91 ft) depth bearing level. Bearing capacity analysis assuming failure by “squeezing” indicated that these layers may fail in this manner. The other layers experienced larger increases in horizontal stresses, with little change in pore pressure, indicating stiffer and higher permeability soils.

3.2.3.6 Vertical Inclinometer Measurements

Vertical inclinometer measurements were taken twice during the test on the FSFOUS to monitor the horizontal ground movement. The results from the inclinometer measurements are plotted in Fig. 53. The baseline measurements were taken on September 26, 2002 (prior to the load test). Two more sets of measurements were taken during the load test. The first set was taken upon completion of primary consolidation settlement [after application of the 1,106 kN (249 kip) load]. The second set was taken upon completion of primary settlement [after the last load of 1,760 kN (396 kips) was applied].
FIG. 53. Vertical Inclinometer Results for FSFOUS with Cumulative Horizontal Displacement Measured (a) Perpendicular to the Side of the Footing; and (b) Parallel to the Side of the Footing
Fig. 53 indicates that about 25 mm (1.0 in.) of cumulative horizontal movement occurred at a depth of about 1.0 m (3.28 ft) below the bottom of the footing at the end of the test. At this time the footing was embedded a total depth of 305 mm (1.0 ft) below the original surface elevation. The majority of the movement occurred in the soft CH layer that experienced large increases in pore pressure and horizontal stress compared to the other layers, as discussed previously. This movement and increase in pore pressure indicate that squeezing probably occurred in this layer. There was less movement recorded parallel to the side of the footing [about 1.0 mm (0.039 in.)] than perpendicular [up to 27 mm (1.05 in.)].

3.2.4 Primary Results of Field Test on Single Pier and Unit Cell Footing

3.2.4.1 Bearing Capacity of Footings

The same three methods used to estimate $q_{ult}$ for the full-scale footing on unreinforced soil were used to estimate $q_{ult}$ from the field data for the unit cell and single pier load tests. The estimated settlement and ultimate load for the unit cell and single pier load tests are plotted as settlement vs. load and log normalized settlement vs. log normalized load curves in Figs. 54-57 and summarized in Table 15.

Table 16 summarizes the values of theoretical $q_{ult}$ for different failure mechanisms compared with measured ranges for load tests on reinforced soils. The measured range of $q_{ult}$ at failure for the unit cell footing was 803 to 886 kPa (16.8 to 18.5 ksf). The estimated failure mode was shearing through the pier-reinforced matrix zone. $q_{ult}$ was predicted to be 455 kPa (9.6 ksf) using Hansen’s equation and 1,360 kPa (28.6 ksf) using Meyerhof’s equation. There is a large discrepancy between these two values, which is due to the variation in Hansen’s and Meyerhof’s basic bearing capacity.
FIG. 54. Load-Settlement Curve for Unit Cell Footing

FIG. 55 Log Load - Log Settlement Curve for Unit Cell Footing
FIG. 56. Load-Settlement Curve for Single Pier

FIG. 57. Log Load - Log Settlement Curve for Single Pier
### TABLE 15. Settlement, Load, and Ultimate Bearing Capacity for Unit Cell and Single Pier

<table>
<thead>
<tr>
<th>Test (1)</th>
<th>Parameter (2)</th>
<th>Method of Analysis</th>
<th>Vesic (3)</th>
<th>De Beer (4)</th>
<th>Double Tangent (5)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Unit Cell Footing</strong></td>
<td>Settlement at Ultimate Load (mm)</td>
<td>Vesic</td>
<td>39</td>
<td>28</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td>Ultimate Load (kN)</td>
<td>De Beer</td>
<td>700</td>
<td>668</td>
<td>634</td>
</tr>
<tr>
<td></td>
<td>Ultimate Bearing Capacity (kPa)</td>
<td>Double Tangent</td>
<td>886</td>
<td>845</td>
<td>803</td>
</tr>
<tr>
<td><strong>Single Pier</strong></td>
<td>Settlement at Ultimate Load (mm)</td>
<td>Vesic</td>
<td>12</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>Ultimate Load (kN)</td>
<td>De Beer</td>
<td>397</td>
<td>397</td>
<td>397</td>
</tr>
<tr>
<td></td>
<td>Ultimate Bearing Capacity (kPa)</td>
<td>Double Tangent</td>
<td>1,360</td>
<td>1,360</td>
<td>1,360</td>
</tr>
</tbody>
</table>

Note: 1 mm = 0.0394 in.; 1 kN = 0.2248 kips; 1 kPa = 0.021 ksf

### TABLE 16. Values of Theoretical $q_{ult}$ for Different Failure Mechanisms Compared with Measured Ranges for Unit Cell and Single Pier ($R_s$ Assumed to be 10)

<table>
<thead>
<tr>
<th>Failure Mechanism (1)</th>
<th>Unit Cell (kPa) (2)</th>
<th>Single Pier (kPa) (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bulging</strong></td>
<td>842</td>
<td>705</td>
</tr>
<tr>
<td><strong>Shearing Below Single Pier</strong></td>
<td>1,640 (M)$^a$</td>
<td>1,420 (H)$^b$</td>
</tr>
<tr>
<td><strong>Shearing within Pier/Matrix Zone</strong></td>
<td>1,360 (M)</td>
<td>455 (H)</td>
</tr>
<tr>
<td><strong>Shearing Below Pier/Matrix Zone</strong></td>
<td>35,820 (M)</td>
<td>35,750 (H)</td>
</tr>
<tr>
<td><strong>Measured Range</strong></td>
<td><strong>Low / High</strong></td>
<td>803 / 886</td>
</tr>
</tbody>
</table>

$^a$M = Meyerhof's Bearing Capacity Equation  
$^b$H = Hansen's Bearing Capacity Equation

Note: 1 kPa = 0.021 ksf
equations (Eqs. 30 and 32). Meyerhof’s $N_γ$ term is 23% higher than Hansen’s and Meyerhof’s $s_c$ term is 153% higher than Hansen’s. Compared to the calculated value, of $q_{ult}$ by Hansen’s method the measured value was under predicted by about 43% to 49%. Meyerhof’s method over predicted the measured value by 53% to 69%. Predicted values of $q_{ult}$ for other failure modes were much larger than the measured values.

The measured $q_{ult}$ at failure of the single pier was 1,360 kPa (28.6 ksf). The likely failure mode was bulging with predicted $q_{ult} = 705$ kPa (14.8 ksf). Bulging was predicted to occur in the CH layer. Shearing through the pier-reinforced matrix zone was another possibility with $q_{ult} = 713$ kPa (15.0 ksf) based on Hansen’s equation and 2,500 kPa (52.5 ksf) according to Meyerhof’s equation. Predicted $q_{ult}$ was approximately 48% less than the measured value according to Hansen and over-predicted by 83% according to Meyerhof.

3.2.4.2 Modulus of Subgrade Reaction for Single Pier

Values of the subgrade modulus of the single pier ($k_p$) were estimated from the load-settlement curve of the single pier load tests. The results are plotted in Fig. 58. These curves will be used later in comparison with the subgrade modulus of the FSFORS and to predict the settlement behavior of the FSFORS.  

Up to a settlement of 12 mm (0.47 in.), the value of $k_p$ fluctuates from 110 MN/m$^3$ to 135 MN/m$^3$ (700 kcf to 860 kcf). After this, $k_p$ decreases with settlement. If a conservative limiting settlement of 25 mm (1.0 in.) is used, then $k_p$ is 63 MN/m$^3$ (400 kcf). The design load for a pier that is 0.7 m (2 ft) by 2.4 m (8 ft) is then calculated to be 440 kN (99 kips). However, if the settlement is limited to 19 mm (0.75 in), then $k_p$ is 77.4 MN/m$^3$ (500 kcf) with a corresponding design load of 430 kN (97 kips).
FIG. 58. Modulus of Subgrade Reaction for Load Tests on Single Pier as a Function of (a) Settlement; and (b) Contact Stress
3.2.4.3 Stress Concentration Ratios for Unit Cell Footing

The stress concentration ratio was not measured directly for the unit cell footing. There was insufficient space to place a pressure plate on the matrix soil for this test, so the following equation was used to determine $R_s$:

$$R_s = \frac{q_p A_m}{F - q_p A_p}$$  \hspace{1cm} (54)

where $q_p$ = Vertical stress on pier
$A_m$ = Area of matrix soil beneath footing
$A_p$ = Area of pier beneath footing
$F$ = Induced vertical force

The variation of stress concentration ratio ($R_s$) versus induced loading for the Unit Cell Footing is presented in Fig. 59. Values of stress concentration ratio ranged 2 to 11 for the parts shown in the figure. During failure almost the entire load was carried by the pier and the calculated values of $R_s$ were very large. These values are not shown in Fig. 59 so that the curve for the working loads prior to failure is visually distinguishable.

3.2.4.4 Induced Horizontal Stress in Single Pier Test

Changes in horizontal stress ($\Delta \sigma_h$) were recorded at a location 150 mm (6 in.) from the edge of the pier and at a depth of 460 mm (18 in.) below the surface. The results are plotted in Fig. 60.

There is a slight increase in $\Delta \sigma_h$ until the induced surface load reached about 300 kN (70 kips) with a settlement of 8 mm (0.31 in). After this point there is a marked increase in horizontal pressure. A significant increase in $\Delta \sigma_h$ occurred before the
Note: During failure most of the load was carried by the pier and values of $R_s$ get very large.

**FIG. 59.** Measured Stress Concentration Ratio and Measured Settlement versus Induced Vertical Force for the Unit Cell Footing
FIG. 60. Change in Horizontal Pressure at a Depth of 0.46 m (1.5 ft) vs. Settlement of Single Pier
ultimate load of 400 kN (90 kips) was reached. The layer that this was recorded was layer 2. Layer 2 is classified as ML with an OCR of 10 and a relatively high undrained shear strength ($s_u$) of 74 kPa (11 psi). As indicated in Table 16, the possible modes of failure are general or local shearing through the pier matrix zone or by bulging.

3.2.4.5 Stress Dissipation with Depth within Piers

The dissipation of induced vertical stress with depth is presented in Fig. 61 for the unit cell pier and in Fig. 62 for the single pier. All data points are shown in Figs. 61 and 62 but curves for only three loads are shown in each figure for increased clarity of presentation. The three curves from each test are plotted together in Fig. 63 for comparison. The following trends are noted from the results shown in these three figures:

a. Within the unit cell pier (Fig. 61), the dissipation of induced vertical stress with depth decreased with increased load up to a value of 400 kN (90 kips), after which a more rapid dissipation of induced stresses occurred at a load of 445 kN (100 kips). The shape of the curves is concave left, indicating that the rate of dissipation decreased with increased depth. This shape is consistent with the results for induced stress dissipation within a group of ten piers supporting a single footing presented in Lawton and Merry (2000).

b. Within the single pier (Fig. 62), the dissipation of induced vertical stress with depth increased with increased load up to a value of 133 kN (30 kips), after which a slower dissipation of induced stresses occurred up to a load of 400 kN (90 kips). The shape of the curves is concave right, indicating that the rate of dissipation increased with increased depth. This shape is opposite to the shape for the unit
Note: After the 400 kN load the deepest pressure plate stopped working.
1 kN = 0.225 kips

FIG. 61. Normalized Induced Vertical Stress versus Normalized Depth Within the Pier Beneath the Unit Cell Footing

FIG. 62. Normalized Induced Compressive Stress versus Normalized Depth Within the Pier of the Single Pier Load Test
FIG. 63. Normalized Induced Compressive Stress versus Normalized Depth Within the Pier of the Single Pier and Unit Cell Footing Load Tests
Comparing the results for the two types of piers (Fig. 63), it can be seen that all three values of load the induced stresses were dissipated more rapidly with depth in the unit cell pier compared to the single pier.

A simple model is shown in Fig. 64 that illustrates the dissipation of induced vertical stress with depth within the single pier and the unit cell pier. The contact stress for the single pier \( q_{p-s} \) acts only on the top of the pier itself [Fig. 64(a)]. For the unit cell footing, there is contact stress on the matrix soil adjacent to the pier \( q_{m-uc} \) in addition to that which acts on top of the pier \( q_{p-uc} \) [Fig. 64(b)]. The stresses acting on a short section of the pier with height \( z \) are shown in Fig 64(c). The reduction in stress from \( q_p \) at the top of the pier to \( \Delta \sigma_v(z) \) at depth \( z \) below the top of the pier depends on the magnitude of the shearing stresses in the matrix soil adjacent to the pier \( \tau_{vm} \) and the diameter of the pier \( d_p \) according to the following equation obtained by summing forces in the vertical direction:

\[
\Delta \sigma_v(z) = q_p - 4 \int_0^{\delta_z} \frac{\tau_{vm}}{d_p} dz
\]  

(55)

The magnitude of \( \tau_{vm} \) depends on the effective horizontal stress in the matrix soil \( \sigma'_m \) and the relative vertical movement between the matrix soil and the pier material \( \delta_z \) along the interface [Fig. 64(c)]. For \( q_{p-s} = q_{p-uc} \) the magnitude of \( \delta_z \) at any depth near the top of the pier will likely be less for the unit cell pier than for the single pier because the rigid unit cell footing promotes more uniform settlement of the pier and the matrix soil. \( q_{m-uc} \) acting on the surface of the matrix soil will increase the vertical stress and hence the
FIG. 64. Simplified Model Illustrating the Dissipation of Induced Vertical Stress with Depth for the Single Pier and Unit Cell Pier: (a) Induced Contact Stress for Single Pier; (b) Induced Contact Stress for Unit Cell Footing; (c) Free Body Diagram of Short Section of Pier; and (d) Shear Stress-Vertical Displacement Relationships Along Interface of Pier and Matrix Soil.
horizontal stress ($\sigma'_{hm}$) adjacent to the unit cell pier as compared to the single pier. This increased horizontal confining pressure produces a stiffer shear stress-displacement response for the unit cell footing, as illustrated in Fig. 64(d). The result is that even though $\delta_{z-uc} < \delta_{z-s}$, $\tau_{vm-cu} > \tau_{vm-s}$ and the vertical stress is dissipated more rapidly within the unit cell pier than within the single pier.

3.2.5 Primary Field Test Results of Full-Scale Footing on Reinforced Soil

3.2.5.1 Bearing Capacity

The same three methods used previously to estimate $q_{ult}$ for the FSFOUS, unit cell, and single pier load tests were used to estimate $q_{ult}$ for the full-scale footing on reinforced soil (FSFORS) load test. The estimated load-settlement and log load - log settlement curves are plotted in Figs. 65-66. Table 17 summarizes the estimated settlement and ultimate load for the FSFORS.

Calculated values of theoretical $q_{ult}$ for the FSFORS and different assumed mechanisms of failure are compared with the range of experimentally determined values in Table 17. Details of the calculations are given in Appendix C. The analyses were performed for two values of $R_s$, 10 and 16. A value of 10 is generally used in design or analysis if no site-specific data are available to predict $R_s$. A value of 16 represents the value at failure determined from measurements obtained during the test.

The differences in predicted $q_{ult}$ for the two assumed values of $R_s$ are less than 5% for all failure mechanisms and therefore $R_s$ had only a minor effect on predicted $q_{ult}$. The results indicate that the controlling failure mechanism was likely to have been bulging of the piers into the sandy lean clay layer (layer 4 in Table 6). This is the same layer into
FIG. 65. Load-Settlement Curve for FSFORS

FIG. 66. Log Settlement vs. Log Load Curve for FSFORS
TABLE 17. Settlement, Load, and Ultimate Bearing Capacity for FSFORS

<table>
<thead>
<tr>
<th>Method of Analysis</th>
<th>Parameter</th>
<th>Vesic (2)</th>
<th>De Beer (3)</th>
<th>Double Tangent (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Settlement at Ultimate Load (mm)</td>
<td>143</td>
<td>59</td>
<td>77</td>
<td></td>
</tr>
<tr>
<td>Ultimate Load (kN)</td>
<td>3,000</td>
<td>2,610</td>
<td>2,735</td>
<td></td>
</tr>
<tr>
<td>Ultimate Bearing Capacity (kPa)</td>
<td>764</td>
<td>664</td>
<td>697</td>
<td></td>
</tr>
</tbody>
</table>

which bulging failure was likely to have occurred for the single pier. The calculated theoretical values of $q_{ult}$ for bulging failure fall within the range of experimentally determined values of $q_{ult}$ for both values of $R_s$.

Two important clarifications are provided here. First, layer 4 was above the GWT and had a degree of saturation of about 91%. At a high degree of saturation (greater than about 70 – 90%), a partially saturated soil responds to changes in load and deformation as if it were saturated because the air is occluded (trapped within the water as bubbles) and there is no continuous air phase (Lawton 2001). Therefore, bulging into this layer is a plausible failure mechanism. Second, the bulging failure likely occurred outward away from the four corner piers. Within the interior of the group, bulging of the piers is restrained by bulging in the opposite direction from adjacent piers. That is, bulging of the central pier was restrained in virtually all directions by bulging from the four corner piers, and bulging of each of the corner piers was restrained in the directions of the two adjacent corner piers and the central pier by bulging from those piers.
TABLE 18. Values of Theoretical $q_{ult}$ for Different Failure Mechanisms Compared with Measured Ranges for FSFORS with $R_s$ Assumed to be 10 and Measured $R_s$ at Failure of 16

<table>
<thead>
<tr>
<th>Failure Mechanism (1)</th>
<th>$q_{ult}$ (kPa)</th>
<th>$R_s = 10$ (2)</th>
<th>$R_s = 16$ (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulging</td>
<td>842</td>
<td>810</td>
<td></td>
</tr>
<tr>
<td>Shearing Below Single Pier</td>
<td>2,320 (M)$^a$</td>
<td>1,890 (H)$^b$</td>
<td>1,580 (M) 1,360 (H)</td>
</tr>
<tr>
<td>Shearing within Pier/Matrix Zone</td>
<td>3,340 (M)</td>
<td>1,100 (H)</td>
<td>3,160 (M) 1,060 (H)</td>
</tr>
<tr>
<td>Shearing Below Pier/Matrix Zone</td>
<td>11,800 (M)</td>
<td>11,500 (H)</td>
<td>12,260 (M) 11,550 (H)</td>
</tr>
<tr>
<td>Measured Range Low / High</td>
<td>664 / 764</td>
<td>664 / 764</td>
<td></td>
</tr>
</tbody>
</table>

$^a$M = Meyerhof’s Bearing Capacity Equation
$^b$H = Hansen’s Bearing Capacity Equation

Note: 1 kPa = 0.021 ksf
3.2.5.2 Contact Pressures and Stress Concentration Ratios

Contact pressures and settlement vs. induced vertical force for the FSFORS are presented in Fig. 67. The ratio of contact pressure on the center pier to the average contact pressure on the corner piers is presented in Fig. 68 as a function of induced vertical force. Prior to failure, the contact pressures on the center pier and corner piers were approximately equal. During and after failure, the center pier carried higher contact pressures than the corner piers as the corner piers began to fail owing to lack of confinement in the directions away from the center pier.

Stress concentration ratio, as defined by Eq. 19, is plotted in Fig. 69, along with settlement, as a function of induced vertical force. The values of $R_s$ at pre-failure loads ranged from 9.5 to 16.0, with the maximum value occurring at about the load at which failure began. $R_s$ decreased with increasing settlement after failure. Even though the footing had “failed”, $R_s$ remained very large. At the maximum settlement of 230 mm (9.1 in.), $R_s$ was 9.0.

3.2.5.3 Change in Contact Pressure with Elapsed Time at Constant Load

Measured contact pressures versus time during the FSFORS test are presented in Fig. 70. Curves are shown for contact pressures on the center pier, the average of the corner piers, and the average of the matrix soil. At each loading step except step 6, the average contact pressure was maintained at a constant value throughout the duration of the step. During step 6 the load was kept constant until primary consolidation had ceased and then the jacking was stopped so that inclinometer readings could be taken. The inclinometer readings were taken during the time period ranging from about 166 – 189
FIG. 67. Measured Contact Pressure and Settlement versus Induced Vertical Force for FSFORS
FIG. 68. Contact Pressure Ratio (Center Pier to Corner Piers) and Settlement vs. Induced Vertical Force for FSFORS

FIG. 69. Stress Concentration Ratio and Settlement vs. Induced Vertical Force for FSFORS
FIG. 70. Change in Contact Pressure with Time for the FSFORS
min. The average contact pressure dropped while these readings were being taken.

Measured contact pressures on the piers and matrix soil vs. elapsed time for loading steps 4 and 5 are presented in Fig. 71. Both loads are pre-failure and correspond to vertical forces of 1,370 and 1,810 kN (307 and 407 kips), respectively, and average contact pressures of 348 and 461 kPa (7.27 and 9.63 ksf), respectively. After the intended load was reached for both loading steps, the contact pressure on the matrix soil decreased and the contact pressure on the center and corner piers increased. The same results are plotted in Fig. 72 for loading steps 6 and 7. Both loads are post-failure and correspond to vertical forces of 2,240 and 3,030 kN (504 and 681 kips), respectively, and average contact pressures of 570 and 771 kPa (11.9 and 16.1 ksf), respectively. At these post-failure loads the trends of changes in contact pressure with time are different than for the pre-failure loads. The contact pressures on the center pier and matrix soil increased with time while the pressure on the corner piers decreased with time. These results are consistent with the previously identified failure mechanism whereby the corner piers failed by bulging in directions away from the center pier while the center pier remained stable.

3.2.5.4 Settlement

Analyses and predictions of settlement of the FSFORS were conducted as described earlier. The subsurface profile was divided into the upper zone (UZ) and the lower zone (LZ). Settlements resulting from strains within the upper zone ($S_{UZ}$) and lower zone ($S_{LZ}$) will be discussed separately.

$S_{UZ}$ is a function of $k_p$ and $k_m$, as well as the footing contact stress, $q_0$ (see Eq. 21). The settlement of the UZ was not measured independently from the overall settlement,
FIG. 71. Change in Contact Pressure with Time for the FSFORS During Loading Steps 4 and 5

FIG. 72. Change in Contact Pressure with Time for the FSFORS During Loading Steps 6 and 7
thus a direct comparison of the results from the various predictive methods with the measured values cannot be made.

In Table 19 estimates of settlement of the UZ are shown using values of $q_p$ and $q_m$ based on measured values of $R_s$ from Fig. 73. In practice, $R_s$ is typically assumed, but here the measured values were used to be more accurate since $R_s$ changed as the load on the footing changed. The process of predicting $S_{uz}$ was as follows: (a) an initial settlement was estimated and using Fig. 73, values of $R_s$ and $k_p$ were determined; (b) knowing the value of $q_0$, Eqs. 17 and 21 were used to calculate $S_{uz}$; if the initial estimate and the calculated settlement were not equal, then a new value of settlement was estimated; (c) the iterative process was continued until calculated $S_{uz}$ equaled estimated $S_{uz}$. A similar process was used to estimate $S_{uz}$ from values of $k_m$ predicted using Terzaghi’s scaling relationships and the results from the plate-load test on the

<table>
<thead>
<tr>
<th>Induced Surface Stress</th>
<th>Calculated Settlement Base on Single Pier Measured Values</th>
<th>Calculated Settlement Base on Scaled Plate Load Test Values, Type 1</th>
<th>Calculated Settlement Based on Scaled Plate Load Test Values, Type 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q_0$ (kPa)</td>
<td>$R_s$ (MN/m$^3$) (mm)</td>
<td>$R_s$ (MN/m$^3$) (mm)</td>
<td>$R_s$ (MN/m$^3$) (mm)</td>
</tr>
<tr>
<td>(1)</td>
<td>(2)</td>
<td>(3)</td>
<td>(4)</td>
</tr>
<tr>
<td>65</td>
<td>10</td>
<td>163</td>
<td>0.9</td>
</tr>
<tr>
<td>115</td>
<td>11</td>
<td>132</td>
<td>2.0</td>
</tr>
<tr>
<td>232</td>
<td>13</td>
<td>125</td>
<td>4.4</td>
</tr>
<tr>
<td>348</td>
<td>13</td>
<td>132</td>
<td>6.3</td>
</tr>
<tr>
<td>461</td>
<td>14</td>
<td>127</td>
<td>8.7</td>
</tr>
<tr>
<td>570</td>
<td>16</td>
<td>96.9</td>
<td>14</td>
</tr>
<tr>
<td>771</td>
<td>11</td>
<td>13.6</td>
<td>130</td>
</tr>
<tr>
<td>808</td>
<td>10</td>
<td>10.4</td>
<td>186</td>
</tr>
</tbody>
</table>

$a$Values from single pier load test  
$b$Scaled values from plate load test over unreinforced soil assuming Type 1 material  
$c$Scaled values from plate load test over unreinforced soil assuming Type 2 material  

Note: 1 kPa = 0.021 ksf; 1 mm = 0.039 in.; 1 MN/m$^3$ = 6.366 kcf
Settlement (mm)

Subgrade Modulus of Single Pier (MN/m$^3$)

Stress Concentration Ratio

FIG. 73. Measured Subgrade Modulus from Single Pier Load Test and Measured Stress Concentration Ratio versus Settlement for FSFORS
unreinforced soil. In this process, an initial value of $S_{uz}$ was estimated for a known value of $q_0$, and $R_s$ was found from Fig. 73. The contract stress on the matrix soil, $q_m$, was then calculated using Eq. 18, and $k_m$ (either Type 1 or Type 2) was determined using Fig. 47. From these values of $q_m$ and $k_m$, $S_{uz}$ was calculated using Eq. 21. If the calculated and assumed values were not the same, an iterative process was used until they were the same. Examples of these procedures are given in Appendix D.

The calculated values of $S_{uz}$ are fairly close ($\leq 18\%$) for the methods based on $k_p$ and $k_m$-Type 2 at pre-failure loads, while the values of $S_{uz}$ for the method based on $k_m$-Type 1 are substantially lower. Within the failure region, the method based on $k_p$ shows a large increase in $S_{uz}$, whereas the two methods based on $k_m$ show only small increases. Thus, the method based on $k_p$, which is the standard method used in practice, provides a reasonable prediction of the settlement of the entire range of stresses. The methods based on $k_m$ provide a reasonable prediction of settlement for pre-failure loads.

The settlement of the lower zone for this test was a combination of immediate and primary consolidation settlement. The lower zone was conceptually divided into two layers for the purpose of analysis. The first layer extended below the pier from 3.05 m (10 ft) to 3.44 m (11.3 ft). The second layer extended from 3.44 m (11.3 ft) to 4.75 m (15.6 ft). Schmertmann’s method was used to estimate immediate settlement, and Westergaard, Newmark, and 1.67:1 methods of stress dissipation were used for primary consolidation settlement. The compressible layer is below the pier reinforced zone and extends to a depth of 4.85 m. The average value of the 3D strain correction factor ($K$) using Skempton and Bjerrum’s method and Leonards is 0.75 for $S_c$ in the lower zone. In addition, the results from stress path tests were used to estimate immediate and primary
consolidation settlement in the lower zone. Predicted values of settlement for the lower zone are summarized in Table 20. Values are given for immediate settlement, primary consolidation settlement by the various methods of stress dissipation, and combined immediate plus primary consolidation settlement using the stress path method.

The total settlement of the FSFORS was calculated by adding the predicted settlements for the UZ and LZ, and the results are summarized in Table 20. The results for the different methods are compared with the measured experimental values in Figs. 74-76. The following conclusions were determined based on comparing the results shown in these three figures and Table 21:

a. The methods based on $k_p$ provided reliable predictions of total settlement for the entire loading range (pre-failure and post-failure).

b. The methods based on $k_m$-Type 2 provided reliable predictions of total settlement within the pre-failure ranges.

**TABLE 20. Predicted Settlement of Lower Zone for FSFORS**

<table>
<thead>
<tr>
<th>$q_o$ (kPa)</th>
<th>Schmertmann $S_i$ (mm)</th>
<th>Westergaard $S_c$ (mm)</th>
<th>Newmark $S_c$ (mm)</th>
<th>1.67:1 $S_c$ (mm)</th>
<th>Stress Path $S_i + S_c$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>65</td>
<td>0.3</td>
<td>0.3</td>
<td>0.4</td>
<td>0.4</td>
<td>0.0</td>
</tr>
<tr>
<td>115</td>
<td>0.6</td>
<td>0.6</td>
<td>0.7</td>
<td>0.7</td>
<td>0.1</td>
</tr>
<tr>
<td>232</td>
<td>1.4</td>
<td>1.1</td>
<td>1.3</td>
<td>1.3</td>
<td>2.7</td>
</tr>
<tr>
<td>348</td>
<td>2.3</td>
<td>1.5</td>
<td>1.7</td>
<td>1.8</td>
<td>4.4</td>
</tr>
<tr>
<td>461</td>
<td>3.3</td>
<td>1.9</td>
<td>2.6</td>
<td>2.4</td>
<td>7.1</td>
</tr>
<tr>
<td>570</td>
<td>4.3</td>
<td>2.4</td>
<td>3.3</td>
<td>3.2</td>
<td>10</td>
</tr>
<tr>
<td>771</td>
<td>6.4</td>
<td>3.6</td>
<td>4.5</td>
<td>4.5</td>
<td>14</td>
</tr>
<tr>
<td>808</td>
<td>6.8</td>
<td>3.8</td>
<td>4.7</td>
<td>4.7</td>
<td>15</td>
</tr>
</tbody>
</table>

Note: 1 kPa = 0.021 ksf; 1 mm = 0.039 in
FIG. 74. Measured Settlement Compared with Predicted Settlement for FSFORS Using Scaled $k_m$-Type 1 for UZ

FIG. 75. Measured Settlement Compared with Predicted Settlement for FSFORS Using Scaled $k_m$-Type 2 for UZ
FIG. 76. Measured Settlement Compared with Predicted Settlement Using $k_p$ for UZ
### TABLE 21. Comparison of Predicted Settlement Using Twelve Methods with Measured Settlement of FSFORS

<table>
<thead>
<tr>
<th>$F$ (kN)</th>
<th>$q_0$ (kPa)</th>
<th>$S_t$ $^a$ (mm)</th>
<th>$S_{u_t}$, Calculated with $k_m$-Type 1</th>
<th>$S_{u_t}$, Calculated with $k_m$-Type 2</th>
<th>$S_{u_t}$, Calculated with $k_b$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>($S_t$ (mm)</td>
<td>Westergaard$^b$</td>
<td>Newmark$^b$</td>
<td>1.67:1$^b$</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>255</td>
<td>64</td>
<td>0.7</td>
<td>1.3</td>
<td>1.4</td>
<td>1.4</td>
</tr>
<tr>
<td>450</td>
<td>115</td>
<td>2.0</td>
<td>2.2</td>
<td>2.3</td>
<td>2.3</td>
</tr>
<tr>
<td>910</td>
<td>232</td>
<td>5.4</td>
<td>4.5</td>
<td>4.7</td>
<td>4.7</td>
</tr>
<tr>
<td>1,370</td>
<td>348</td>
<td>10</td>
<td>6.8</td>
<td>7.0</td>
<td>7.1</td>
</tr>
<tr>
<td>1,810</td>
<td>461</td>
<td>19</td>
<td>9.3</td>
<td>10.0</td>
<td>9.8</td>
</tr>
<tr>
<td>2,240</td>
<td>570</td>
<td>36</td>
<td>12</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>3,030</td>
<td>771</td>
<td>143</td>
<td>17</td>
<td>17</td>
<td>17</td>
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<tr>
<td>3,170</td>
<td>808</td>
<td>233</td>
<td>18</td>
<td>19</td>
<td>19</td>
</tr>
</tbody>
</table>

$^a S_t = S_{u_t} + S_r$ for this test

$^b$Stress Distribution Method For Lower Zone Settlement

$^c$Settlement of Lower Zone Calculated by Stress Path

Note: 1 kN = 0.225 kips; 1 kPa = 0.021 ksf; 1 mm = 0.039 in.
c. The methods based on $k_m$-Type 1 overestimated the total settlement at values of $q_0$ less than about 100 kPa (2.1 ksf) and underestimated the total settlement at higher values of $q_0$.

b. It is reasonable that the methods based on $k_m$-Type 2 gave better predictions of settlement than those based on $k_m$-Type 1. Type 2 is based on the assumption that the deformation characteristics are constant with depth, which is consistent with the behavior of highly overconsolidated soils like those present within the bearing zone of the FSFORS and the plate load test.

c. The method of stress distribution used to estimate primary consolidation settlement of the lower zone had little influence on the calculated values of total settlement for any of the methods.

d. The stress path method, although theoretically superior to the other methods, did not provide better predictions of total settlement.

In standard practice, $S_{uz}$ is calculated based on $k_p$ from a single pier test, immediate settlement of the lower zone is calculated using Schmertmann’s method, and primary consolidation settlement of the lower zone is calculated based on the 1.67:1 stress distribution method. In this study, this standard method provided results as good or better than all of the other methods used. It is also interesting to note that calculating $S_{uz}$ based on $k_m$ from a plate load test and assuming that the deformation characteristics of the bearing soil are constant with depth provided reasonable estimates of total settlement within the pre-failure stress range. Therefore, although the method based on $k_p$ from a single pier test should be used in practice whenever possible, it appears that the method
based on $k_m$ from a plate load test conducted on the matrix soil can be used for reliable prediction of $S_{uc}$ if appropriate values of $R_s$ can be estimated or otherwise determined. This method has potential application on very small projects where the cost of performing a single pier test would be prohibitive.

3.2.5.5 Stress Dissipation with Depth

Plots of normalized induced vertical stress ($\Delta \sigma_{v(z)} / \Delta \sigma_{v(z=0)}$) within two piers of the group footing versus normalized depth ($z_d/d_p$), are presented in Figs. 77-78. The results for the center pier are shown in Fig. 77 and for a corner pier in Fig. 78.

A similar trend is evident for both piers relative to changes in the normalized induced stress with changes in load. For each pier at both depths of measurement, the induced vertical stress initially decreases with increased load to a minimum value at a critical load and then increases at loads greater than the critical value. For the center pier, the critical load is about 912 kN (205 kips) at $z/d_p = 0.69$ and is between 450 and 912 kN (101 and 205 kips) at $z/d_p = 1.69$. For the corner pier, the critical load is about 450 kN (101 kips) at $z/d_p = 1.07$ and is between 450 and 912 kN (101 and 205 kips) at $z/d_p = 2.10$. These values of critical load are well below the failure load of about 2,700 kN (about 610 kips) for the pier-reinforced system.

In contrast, the shapes of the curves for the two piers are different – concave downward for the center pier and concave upward for the corner pier. Comparing Figs. 61-62 with 77-78, it can be seen that the shapes of the curves are similar for the center pier of the group footing and the pier for the unit cell footing; and similar for the corner pier and the single pier. With only some slight modifications to the area of matrix soil over which $q_m$ acts for the corner pier compared to the single pier, the simplified model
FIG. 77. Normalized Induced Vertical Stress vs. Normalized Depth Within the Center Pier of FSFORS

FIG. 78. Normalized Induced Vertical Stress vs. Normalized Depth Within a Corner Pier of FSFORS
shown in Fig. 64 can also be used to explain the differences in the dissipation of vertical stresses with depth within the piers supporting a group footing.

One additional result needs to be addressed. At a load of 1,370 kN (307 kips) and a normalized depth of 1.07 within the corner pier, the normalized induced vertical stress is about 1.0 (Fig. 78). The pressure plate from which this measurement of $\Delta\sigma_v$ was recorded was located at a depth just below the zone where the primary bulging failure likely occurred. It is also likely that the zone of bulging extended somewhat above and below this weak zone. Referring to the free body diagram in Fig. 64(c), this result would indicate that the shearing stresses along the interface of the pier and matrix soil were approximately zero. This result is consistent with the initiation of bulging in the corner piers, with outward displacement of the pier and matrix soil as a composite system rather than differential vertical movement along the interface of the two materials.

3.2.5.6 Induced Horizontal Pressures and Excess Pore Water Pressures within the Matrix Soil

Induced horizontal stress ($\Delta\sigma_h$) and excess pore water pressure ($\Delta u$) were measured using a push-in cell at a depth of 1.74 m (5.71 ft) within the matrix soil between the center pier and the northwest corner pier [see Fig. 37(a)]. The soil layer at this depth is classified as CL, sandy lean clay (Layer 4, see Table 6) and is one of the weaker layers into which bulging failure of the corner piers likely occurred. The GWT at time of the test was 1.86 m (6.1 ft). The initial degree of saturation of the soil within this layer was about 89% as determined from undisturbed samples. This value of degree of saturation is sufficiently large that all air within the pore spaces was likely trapped as bubbles within the water. Therefore, even though this depth is above the GWT, this soil would respond
to changes in deformation and stress as if it were saturated. The results are plotted in Fig. 79 and the trends for $\Delta \sigma_h$ and $\Delta u$ are the same. Up to an induced vertical load of about 1,700 kN (382 kips), $\Delta \sigma_h$ and $\Delta u$ gradually increased with increased load. The values of $\Delta u$ within this loading range are very small ($< 6$ kPa $\approx 110$ psf), which confirms that primary consolidation was essentially complete. As the load increased from 1,700 to 3,030 kN (382 to 681 kips), $\Delta \sigma_h$ and $\Delta u$ rapidly increased. From 3,030 to 3,170 kN (681 to 713 kips) $\Delta \sigma_h$ and $\Delta u$ decreased slightly. Although the post-failure levels of $\Delta u$ were larger than desired, the settlement of the footing that would result from the dissipation of these excess pore water pressures would be small because the values of $\Delta u$ were small in comparison to the total stress levels ($\sigma_v$ and $\sigma_h$) within the matrix soil at that depth. Furthermore, the post-failure condition was not a primary focus of this research.

These results are consistent with the development of bulging failure of the corner piers into this layer of matrix soil. As bulging failure developed, the corner piers pushed into the matrix soil. In the direction of the center pier, the movement of the matrix soil was inhibited by the stability of the center pier and by pushing in the opposite direction from bulging of the opposite corner pier. This combination of pushing on the matrix soil from one side and resistance to the movement on the opposite side generated increased total stresses and excess pore water pressures within the matrix soil in the direction of the movement.
FIG. 79. Induced Horizontal Pressure and Excess Pore Pressure within Matrix Soil at a Depth of 1.74 m (5.71 ft) vs. Settlement of FSFORS

1 kN = 0.2248 kips
1 mm = 0.0394 in.
1 kPa = 20.886 psf
3.2.6 General Comparisons

3.2.6.1 FSFOUS and FSFORS

The load-settlement curves for the FSFORS and the FSFOUS are presented for comparison in Fig. 80. It has previously been shown that the ultimate bearing capacity ($q_{ult}$) of the FSFOUS ranged from 132 to 154 kPa (2.76 to 3.22 ksf), with an average value of 143 kPa (2.99 ksf). For the FSFORS, $q_{ult}$ ranged from 664 to 764 kPa (13.9 to 16.0 ksf), with an average value of 714 kPa (14.9 ksf). The increase in $q_{ult}$ obtained by reinforcing the soil with Geopier elements ranged from 331 to 479%, with an average value of 399%. Therefore, the ultimate bearing capacity of the FSFORS was about five times that of the FSFOUS.

Two typical criteria will be used to estimate the allowable bearing pressure ($q_a$) for each of the two footings: (a) A tolerable settlement of 25 mm (1.0 in.), and (b) a factor of safety of 2.0 applied to $q_{ult}$. For the FSFOUS, the induced vertical force at a settlement of 25 mm (1.0 in.) was about 625 kN (141 kips), corresponding to $q_a = 159$ kPa; and for the FSFORS, the induced vertical force at the tolerable settlement was about 2,000 kN (450 kips), corresponding to $q_a = 510$ kPa. Applying a factor of safety of 2.0 to the average values of $q_{ult}$ gives $q_a = 71.5$ kPa (1.49 ksf) for the FSFOUS and $q_a = 357$ kPa (7.46 ksf) for the FSFORS. The criterion based on applying a factor of safety to $q_{ult}$ governs for each footing, giving final values of $q_a$ of 71.5 kPa (1.49 ksf) for the FSFOUS and 357 kPa (7.46 ksf). Hence, the allowable bearing pressure for the Geopier-reinforced bearing soil is about five times that for the unreinforced soil.

It is important to note that in most practical footing design cases, the allowable bearing pressure is controlled by settlement rather than ultimate capacity. So this case is
FIG. 80. Comparison of Load-Settlement Curves for FSFOUS and FSFORS

1 kN = 0.225 kips
1 mm = 0.039 in.
unusual in that ultimate bearing capacity governed rather than settlement. Furthermore, it is unusual that the FSFOUS “failed” at a settlement less than 25 mm (1.0 in.). This failure at such a small settlement results primarily from two factors: (a) Bearing capacity failure occurred as a result of squeezing rather than general shear failure or the traditional local shear failure, and (b) the footing was not embedded. Embedment of the footing, which is the most common situation in practice, would both reduce the settlement for any given bearing pressure and increase the ultimate bearing capacity. Thus, if the FSFOUS had been embedded to a typical depth, it would not likely have failed at a settlement less than 25 mm (1.0 in.). Embedding the FSFORS also would produce reductions in settlement and an increase in $q_{ult}$. Therefore, the comparative results would still be the same, with the pier-reinforced bearing soil being several times stronger and stiffer than the unreinforced soil. However, an important lesson to be learned from these results is that it is imperative for foundation design engineers to check for the possibilities of squeezing or local shear failure and not arbitrarily assume that general shear failure will be the controlling mechanism for all shallow foundations.

The pressure plates used to measure contact stress for the FSFOUS and the FSFORS were purposely placed in the same locations within the footprint (compare Figs. 36 and 37) so that comparisons of contact stresses could be made for the two footings. These comparisons are shown in Fig. 81 as a function of induced vertical force, with open symbols used for the FSFOUS and filled symbols used for the FSFORS. Symbols of the same type (e.g. circle) represent the same location within the footprint. Comparison of the contact stresses for the two footings clearly shows the dramatic changes in contact stresses produced by incorporation of the piers. For the FSFORS the
FIG. 81. Comparison of Contact Pressures for FSFOUS and FSFORS

1 kN = 0.225 kips
1 kPa = 20.886 psf
contact stresses at the locations of the piers are several times greater than for the FSFOUS. In contrast, the stresses on the matrix soil for the FSFORS are much smaller than for the FSFOUS. These results demonstrate the re-distribution of contact stresses for the footing supported by piers compared to the footing on unreinforced soil, as well as the concentration of stress on the stiff piers compared to the much more compressible matrix soil for the footing on reinforced soil.

3.2.6.2 Pier Group Footing and Single Pier Test

A comparison of the full-scale pier group footing and the single pier footing in terms of contact stress on top of the pier(s) versus settlement is presented in Fig. 82. The results compared well for \( q_p \) less than about 500 kPa (10 ksf), after which the settlement of the single pier is less than the settlement of the pier group at the same values of \( q_p \). Thus, there is a slight “negative” group effect at these higher stresses. This method shows that the behavior of a pier group loaded in compression can be predicted reasonably well by conducting a relatively inexpensive test on a single pier as compared to the cost of testing a full-scale footing.

3.2.6.3 Pier Group Footing and Unit Cell Footing

The concept of a unit cell for a group of piers was discussed previously in the literature review (see Fig. 14 and the corresponding discussion). The unit cell approach for this research involved using a square footing supported by one pier. The contact area of this unit cell footing was one-fifth the contact area of the FSFORS. Therefore, the unit cell footing had the same contact area per pier as the FSFORS. The primary purpose for
FIG. 82. Comparison of Contact Stress-Settlement Curves for Single Pier and Piers of FSFORS
conducting the test on the unit cell footing was to determine if this test could be used in place of the standard load test on a single pier to provide better information to predict the settlement and ultimate bearing capacity of the full-scale footing and to provide site-specific data on stress concentration ratio by applying load directly to the matrix soil in addition to the pier. It should be noted, however, that this test was not a true “unit cell” test because two conditions for the idealized case were not met – lateral deformations did occur across the fictitious boundary of the unit cell (perimeter of the unit cell footing extended vertically downward) and the shearing stresses were not zero on this boundary.

Curves of settlement versus induced contact stress on the pier are plotted together in Fig. 83 for the FSFORS and the unit cell footing. Comparing Figs. 82 and 83 shows that the settlement-pier contact stress relationship was better predicted from the results of the test on the single pier compared to the test on the unit cell footing. A comparison of these two figures also shows that the ultimate bearing capacity of the piers in the group was better predicted from the test on the single pier than from the test on the unit cell footing.

In Fig. 84, the relationships between stress concentration ratio and induced contact stress on piers for the FSFORS and the unit cell footing are compared. It can be clearly seen that within the working range of stresses ($q_p < 1,300$ kPa = 27 ksf), the stress concentration ratios ($R_s$) were significantly lower for the test on the unit cell footing than for the test on the FSFORS. At the smaller values of $q_p$, the values of $R_s$ for the unit cell footing were about half of those for the FSFORS. At values of $q_p$ in the range of about 750 to 1,300 kPa (16 to 27 ksf), the values of $R_s$ for the unit cell footing were about two-thirds of those for the FSFORS.
FIG. 83. Comparison of Contact Stress-Settlement Curves for Piers of FSFORS and Unit Cell Footing
FIG. 84. Comparison of Contact Stress-Stress Concentration Ratio Curves for Piers of FSFORS and Unit Cell Footing
The reasons for the relatively poor prediction of settlement, ultimate bearing capacity, and stress concentration ratio from the results of the test on the unit cell footing compared to the test on the single pier are numerous. First, the test on the unit cell footing did not meet two important criteria of the idealized unit cell, as discussed earlier in this section. Second, the test on the unit cell footing did not appropriately account for the important group interaction effects that are discussed in the next section. In addition, the depth of influence for the test on the unit cell footing was substantially less than for the FSFORS. Since the soils were highly stratified with significantly different engineering properties for many of the layers, this difference in depth of influence produced a lack of correlation between the results for the two tests. For all of these reasons, it can be concluded that at this site, the results from the test on the unit cell footing could not be used to obtain a reliable prediction of the engineering response and behavior of the FSFORS with respect to settlement, ultimate bearing capacity, and stress concentration. However, it does appear that in practice the test on the unit cell footing could be used to obtain very conservative estimates of stress concentration ratio.

3.2.6.4 Group Effect

Several phenomena occur during the installation and loading of a group of piers that do not occur during installation and loading of a single pier. During subsequent installation of adjacent piers after a pier has been installed, the matrix soil between the piers is pushed toward the existing pier. This movement is resisted by the existing pier, thereby stiffening and strengthening the matrix soil and resulting in an increase in the horizontal stress (unless the limiting passive resistance has already been achieved). During compressive loading of a group of piers, two primary group phenomena occur:
a. The large compressive stresses induced on the top of the piers are distributed outward with depth, resulting in greater stresses on the lower portions of the group piers compared to a single pier.

b. With increased load, the stresses are concentrated on the piers and the piers tend to bulge outward. In the direction between adjacent piers, this outward movement occurs in opposite directions and the movement is resisted by the opposite movement from each adjacent pier.

The combination of these phenomena and some other less significant phenomena is called the *group effect*. The stiffening of the matrix soil and the increase in horizontal stress during installation of the group piers tends to increase the stiffness of the pier group compared to the single pier. The two primary group phenomena during compressive loading tend to offset each other to some degree. The increase in stresses within the lower portions of the piers tends to increase the settlement compared to the single pier case, while the bulging toward adjacent piers tends to decrease the settlement. Looking at Fig. 82, the group effect is insignificant at values of $q_p$ less than about 500 kPa (10 ksf). At values of $q_p$ between 500 kPa (10 ksf) and failure, there is a somewhat negative group effect (the pier group settled more than the single pier for the same magnitudes of $q_p$). Overall it can be said that within the normal range of working stresses, the group effect is insignificant and therefore the results from the test on the single pier can be used for reliable predictions of the load-settlement response of the group footing.
CHAPTER 4

SUMMARY, CONCLUSIONS,
AND RECOMMENDATIONS

4.1 SUMMARY

Research was conducted to achieve the following primary objectives with respect to Geopier-reinforced bearing soil supporting a shallow footing loaded in compression: (a) Determine the improvement in settlement and ultimate bearing capacity compared to a shallow footing bearing on unreinforced soil; (b) compare the response and behavior of a group of piers compared to a single isolated pier subjected to comparable loads (that is, the “group effect”); and (c) determine the effectiveness of existing design methods to predict the settlement and bearing capacity of a pier-supported footing. A secondary objective was to determine the effectiveness of existing design methods to predict the settlement and bearing capacity of a footing bearing on unreinforced soil.

An extensive program was undertaken to attain these objectives. First, the geologic profile at the testing site was determined and engineering and index properties of the relevant soils were obtained using a variety of laboratory and field techniques. Next, the primary field tests were conducted and consisted of compressive load tests to failure of a full-scale footing on pier-reinforced soil, a full-scale footing and a plate-load on unreinforced soil, a “unit cell” footing supported by one pier, and a single isolated pier. Finally, the data from the primary tests were reduced and in-depth evaluations and
analyses of the results were undertaken.

4.2 RESULTS AND CONCLUSIONS

The primary results and conclusions of this research are discussed below according to four categories – full-scale footing on unreinforced soil, full-scale footing on Geopier-reinforced soil, comparisons, and general.

4.2.1 Full-Scale Footing on Unreinforced Soil

1. Failure likely occurred by “squeezing” of a soft layer sandwiched between harder layers.

2. Estimates of ultimate bearing capacity using traditional methods (Terzaghi, Meyerhof, and Hansen) were several hundred percent greater than the measured value. This result is reasonable since these methods assume a general shear failure whereas the actual failure likely occurred as a result of squeezing.

3. Estimates of ultimate bearing capacity using Terzaghi’s method of reduced strength parameters for local shear failure were 31 to 98% greater than the measured value. This result is reasonable since the assumed and actual failure mechanisms are different.

4. The results described in 1, 2, and 3 above illustrate the importance of using theories that are appropriate for the actual conditions. If this had been a real design situation and the geotechnical engineer had unwittingly assumed that the traditional theories based on general shear failure were appropriate and had used a typical factor of safety in the range of 2 to 3, it is likely that a bearing capacity
failure would have occurred during service.

5. The ultimate bearing capacity of the full-scale footing on unreinforced soil was reached when the footing had settled in the range of 10 to 19 mm (0.39 to 0.75 in.). Thus, in a typical design situation with a tolerable settlement of 25 mm (1.0 in.) or more, the allowable bearing pressure for this footing would be controlled by ultimate bearing capacity rather than settlement. This result is contrary to most shallow foundations for which allowable bearing pressure is governed by deformation rather than strength.

6. Several methods were used to estimate the settlement of the FSFOUS. Each method overestimated the pre-failure settlements by several tens of percent. This result is consistent with standard geotechnical practice. No method was found to be clearly superior to the others in this case. However, the use of the stress path method resulted in the worst predictions. This result is surprising because the stress path method is the most fundamentally sound method if changes in the state of stress within the bearing zone can be predicted reliably. In this case, methods valid for homogeneous soils were used to predict induced vertical stresses. In reality, the upper portions of the bearing zone were much stiffer than the underlying portions, likely resulting in induced vertical stresses and settlements much smaller than those predicted.

### 4.2.2 Full-Scale Footing on Geopier-Reinforced Soil

1. The value of ultimate bearing capacity predicted by the standard method used in practice (710 kPa = 14.8 ksf) was in the middle of the range of actual values
calculated using a variety of methods to determine when failure occurred (664 to 764 kPa = 13.9 to 16.0 ksf). The predicted method of failure was bulging of the piers into the same soft layer that likely squeezed beneath the footing on unreinforced soil. The available physical evidence also supports this method of failure. Thus, use of the standard method of practice worked well in this case.

2. The standard methods of practice for calculating settlement of the upper and lower zones provided estimates of total settlement that were as good or better than the other methods used in this study. These standard methods were able to predict reliably the entire load-total settlement relationship, including both the pre-failure and the post-failure regions.

4.2.3 Comparisons

1. The ultimate bearing capacity of the full-scale footing bearing on Geopier-reinforced soil was about 5.0 times that of the same footing bearing on unreinforced soil.

2. Use of Geopier reinforcement beneath the full-scale footing substantially reduced the settlement compared to the unreinforced soil at the same loads. Thus, the allowable bearing pressure for any typical value of tolerable settlement was substantially increased for the pier-reinforced soil. Using criteria of a tolerable settlement of 25 mm (1.0 in.) and a factor of safety of 2.0 applied to ultimate bearing capacity, the allowable bearing pressure for the Geopier-reinforced bearing soil is about 5.0 times that for the unreinforced soil.

3. A test was conducted on a unit cell footing bearing on a single pier and having the
same contact area per pier as the full-scale footing. The purpose of this test was to determine if the results from it could provide better predictions of the settlement and bearing capacity of the full-scale footing than the standard single pier test used in practice. However, comparison of the results from the tests on the unit cell footing, single pier, and full-scale footing on pier-reinforced soil showed that the single pier test provided better prediction of the full-scale footing performance than did the unit cell test. Within the pre-failure range of stresses, the stress concentration ratios for the unit cell footing were about one-half to two-thirds the values from the full-scale footing at comparable stress levels. Thus, it appears that the only potential use for the unit cell footing test would be to obtain a very conservative estimate of stress concentration ratio as a function of pier contact stress at a particular location.

4. Comparison of the results from the tests on the single pier and the full-scale footing on reinforced soil showed that within the range of working stresses, there was no significant group effect. Therefore, the single pier test can be used to provide reliable predictions of settlement for a full-scale footing. As the loads approached failure, there was a slight negative group effect.

4.2.4 General

1. The methods currently used in practice to predict settlement and bearing capacity of single Geopier elements and footings bearing on Geopier-reinforced soil worked well for the piers and footing tested in this research program. It is recommended that these methods continue to be used in practice.
2. This study and other recent studies have shown that Geopier soil reinforcement is a technology well-suited for the improvement of the soft to medium stiff cohesive soils typically found in the Salt Lake and adjacent valleys. This technology can greatly improve the bearing and uplift capacity of shallow and intermediate foundation systems and also provides an economical alternative to deep foundation systems for bridges and other structures. For this case, it is possible to obtain the required bearing, uplift and settlement performance using shallow foundations (e.g., spread footings) supported by Geopier soil reinforcement.

3. There is an increasing trend toward the use of the design-build project delivery system for major construction projects in Utah. This type of system favors performance-based design and the use of innovative and cost-effective technologies to meet project requirements. Geopier soil reinforcement is a demonstrated technology with sufficiently mature design methods to have a place in such design-build projects. Depending on the ground conditions and specific project needs, Geopier soil reinforcement should be considered for soil stabilization under MSE walls, bridge abutments, and other structures or embankment requiring significant load bearing and/or uplift capacity.

4.3 RECOMMENDATIONS FOR FUTURE RESEARCH

Although the research described in this study provided great insight into the behavior of a group of Geopier elements supporting a footing loaded in compression, the following future research needs have been identified with respect to the response and behavior of a Geopier group:
1. Settlement of the upper zone needs to be monitored separately from the lower zone in order to verify the settlement predictions for both zones.

2. Changes in horizontal stresses were only recorded at one location and one depth during the pier group test. Additional measurements at various locations and depths are required to determine how the horizontal stresses vary with depth and location, and to determine exactly how the piers fail.

3. More sophisticated instrumentation should be employed to measure the horizontal ground movements at numerous locations within the Geopier group and the surrounding matrix soil during the failure of the piers. This will provide greater insight into how failure occurs within the group and how the response and behavior of the interior pier(s) compares with the response and behavior of the exterior piers and their interaction.

4. Several pressure plates are needed with depth within each pier in the group. This will allow a determination of how vertical stresses are developed within each of the piers and the group interaction among the piers.
APPENDIX A

LIST OF NOTATIONS USED IN TEXT
$1D$ = one-dimension or one-dimensional

$2D$ = two-dimensions or two dimensional

$3D$ = three-dimensions or three-dimensional

$A$ = area of footing, or used as a coefficient

$A'$ = DeBeer factor used to determine field bearing capacity failure

$A_{m}$ = area of matrix soil beneath footing

$A_{p}$ = area of pier(s) beneath footing

$A_{shaft}$ = area of pier shaft calculated using $d_{shaft}$ as the diameter

$B$ = width of footing

$B_{p}$ = width of load plate for plate load test

$BST$ = borehole shear test

$b$ = radius of footing or plate

$c$ = cohesion intercept of soil

$c''$ = cohesion intercept of soil corrected for local shear failure to use in bearing capacity equations based on general shear failure

$C_{1}$ = Schmertmann’s strain relief factor

$C_{2}$ = Schmertmann’s time dependent settlement factor for cohesionless soils

$c_{av}$ = average cohesion intercept of soil

$C_{c}$ = compression index

$c_{m}$ = cohesion intercept of matrix soil

$c_{p}$ = cohesion intercept of pier material

$CPT$ = cone penetration test

$CPTU$ = cone penetration tests with pore pressures recorded
\( C_r = \) recompression index

\( d = \) depth below ground surface to ground water table

\( D_f = \) depth of embedment of foundation

\( DMT = \) dilatometer test

\( d_p = \) nominal diameter of pier

\( D_r = \) relative density

\( d_{shaft} = \) estimated actual diameter of shaft, equal to \( d_p \) plus 76 mm (3.0 in.)

\( DST = \) single stage direct shear test

\( DSTM = \) multi-stage direct shear test

\( DSTSM = \) combined single stage and multi-stage direct shear test

\( DT = \) displacement transducer

\( d_w = \) depth to ground water table below bearing elevation (bottom of footing)

\( d_{comp} = \) depth of composite zone used to calculate average soil strength layers

\( e_0 = \) initial void ratio

\( e_p = \) void ratio at preconsolidation stress

\( E_s = \) stress-strain (Young’s) modulus for soil, may be subscripted as \( NC \) for normally consolidated, \( OC \) for overconsolidated, or \( u \) for undrained

\( F = \) force, may be subscripted

\( f_s = \) skin friction

\( FSFORS = \) full-scale footing on reinforced soil

\( FSFOUS = \) full-scale footing on unreinforced soil

\( GWT = \) ground water table

\( H = \) Height of influence
$H_0 =$ initial height, may be subscripted with layer number

$H_p =$ height of pier

$H_{shaft} =$ height of pier plus one pier diameter

$I_p =$ plasticity index

$I_z =$ Schmertmann’s strain influence factor

$K =$ spring constant

$K_0 =$ coefficient of lateral earth pressure at-rest

$k_m =$ subgrade soil modulus coefficient for matrix soil

$k_{mp} =$ subgrade modulus for plate load test

$k_p =$ subgrade soil modulus coefficient for pier

$k_{pg} =$ subgrade soil modulus coefficient for pier group

$k_{pm} =$ Rankine’s lateral earth pressure for matrix soil

$k_{pp} =$ Rankine’s lateral earth pressure for pier material

$L =$ length of footing

$LL =$ liquid limit

$L_{0,h} =$ measured initial length of wire used to measure horizontal movement

$L_{0,h1} =$ measured initial length of wire used to measure theoretical horizontal movement

$L_{0,v} =$ measured initial length of wire used to measure vertical movement

$LZ =$ lower zone of soil beneath pier group footing

$n =$ number of $i^{th}$ layer

$NC =$ normally consolidated

$N_p =$ Number of piers

$OC =$ overconsolidated
\( OCR \) = overconsolidation ratio

\( p \) = perimeter

\( PP \) = pressure plate

\( PIC \) = push in pressure cell

\( q_0 \) = average bearing (contact) stress

\( q_a \) = allowable bearing (contact) stress

\( q_c \) = tip resistance from cone penetration test

\( q_m \) = induced vertical stress on matrix soil

\( q_p \) = induced vertical stress on top of pier

\( q_{ult} \) = ultimate bearing capacity of footing

\( q_{ult,p} \) = ultimate bearing stress at top of single pier

\( q_{ult,bot,p} \) = bearing capacity of the bottom of the pier, end bearing

\( R_a \) = area replacement ratio

\( R'_a \) = factored area replacement ratio

\( R_s \) = stress concentration ratio

\( R'_s \) = factored stress concentration ratio

\( S \) = degree of saturation

\( S_c \) = settlement from primary consolidation

\( S_{c,LZ} \) = primary consolidation settlement of lower zone

\( SCPT \) = seismic cone penetration test

\( S_i \) = immediate settlement, may be subscripted to indicate layer

\( S_{i,LZ} \) = immediate settlement of lower zone

\( S_{UZ} \) = settlement of upper zone, may be subscripted to indicate method of analysis
\( S_m \) = settlement induced by moisture change

\( S_s \) = settlement from secondary compression

\( S_{s,LZ} \) = settlement from secondary compression in lower zone

\( S_t \) = total settlement

\( s_u \) = undrained shear strength

\( u \) = excess pore pressure

\( UU \) = unconsolidated undrained

\( UZ \) = upper zone

\( w_n \) = natural soil water content

\( W_p \) = weight of pier

\( z \) = depth below footing, may be subscripted to indicate layer

\( \gamma' \) = effective soil unit weight

\( \gamma_c \) = unit weight of soil corrected for ground water table when it is within failure wedge

\( \gamma_{sat} \) = saturated unit weight of soil

\( \gamma_w \) = unit weight of water

\( \gamma_{wet} \) = unit weight of soil above ground water table

\( \Delta h \) = measured horizontal movement

\( \Delta h_t \) = theoretical horizontal movement

\( \Delta h_2 \) = measured horizontal movement perpendicular to theoretical horizontal movement

\( \Delta q \) = change in bearing (contact) stress

\( \Delta v \) = measured vertical movement

\( \Delta v_t \) = true vertical movement

\( \Delta z \) = thickness of soil layer
\( \eta \) = multiplier for Eq. 47

\( \sigma = \) total vertical stress

\( \sigma' = \) effective vertical stress

\( \sigma_l = \) major principal stress

\( \sigma_3 = \) minor principal stress

\( \sigma_h = \) horizontal pressure

\( \sigma'_{vp} = \) effective vertical preconsolidation stress

\( \sigma_{r,\text{lim}} = \) limited radial value where indefinite soil expansion occurs

\( \sigma_{r,0} = \) initial or at-rest radial total stress

\( \sigma_{v,0} = \) initial or at-rest vertical total stress

\( \sigma_{v,l} = \) effective vertical stress after change from initial conditions

\( \phi = \) total stress angle of internal soil friction

\( \phi' = \) effective stress angle of internal soil friction

\( \phi'' = \) corrected angle of internal soil friction for determination of local shear bearing capacity failure

\( \phi_{av} = \) average angle of internal soil friction

\( \mu = \) Poisson’s ratio

\( \mu_m = \) multiplier for Eq. 18

\( \mu_p = \) multiplier for Eq. 17

\( \tau = \) shear strength, may be subscripted
APPENDIX B

UNIT CONVERSIONS
## UNIT CONVERSIONS

<table>
<thead>
<tr>
<th>To Convert From</th>
<th>To</th>
<th>Multiply by</th>
</tr>
</thead>
<tbody>
<tr>
<td>mm</td>
<td>in.</td>
<td>0.03937</td>
</tr>
<tr>
<td>m</td>
<td>in.</td>
<td>39.37</td>
</tr>
<tr>
<td>m</td>
<td>ft</td>
<td>3.281</td>
</tr>
<tr>
<td>m²</td>
<td>ft²</td>
<td>10.76</td>
</tr>
<tr>
<td>m³</td>
<td>ft³</td>
<td>35.31</td>
</tr>
<tr>
<td>kN</td>
<td>kip</td>
<td>0.2248</td>
</tr>
<tr>
<td>kN</td>
<td>ton</td>
<td>0.1124</td>
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<td>psi</td>
<td>0.1450</td>
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<tr>
<td>kPa</td>
<td>ksf</td>
<td>0.02089</td>
</tr>
<tr>
<td>kN/m³</td>
<td>pcf</td>
<td>6.366</td>
</tr>
<tr>
<td>kN/m³</td>
<td>kcf</td>
<td>0.006366</td>
</tr>
<tr>
<td>MN/m³</td>
<td>kcf</td>
<td>6.366</td>
</tr>
</tbody>
</table>
Example calculations will be shown for the ultimate bearing capacity of the full-scale footings on both unreinforced soil and soil reinforced with Geopier elements. The footing dimensions and other appropriate properties were as follows:

\[ B = 1.98 \text{ m} \quad D_f = 0 \text{ m} \quad \text{Depth to GWT} = 1.86 \text{ m} \]
\[ L = 1.98 \text{ m} \quad A = 3.93 \text{ m}^2 \quad \gamma_w = 9.81 \text{ kN/m}^3 \]

See Table 6 for soil properties.

Properties of the Geopier element were as follows:

\[ d_p = 0.610 \text{ m} \quad H_p = 2.44 \text{ m} \quad 5 \text{ Geopier elements} \]
\[ A_{\text{Geopier elements}} = 1.85 \text{ m}^2 \quad d_{\text{shaft}} = 0.686 \text{ m} \quad H_{\text{shaft}} = 3.05 \text{ m} \]
\[ R_a = 0.471 \quad R_s = 10 \text{ (assumed)} \]

Above the GWT:

\[ H_{\text{shaft}} = 1.86 \text{ m} \quad \phi'_p = 50 \text{ degrees} \quad c'_p = 0 \text{ kPa} \quad \gamma_p = 21.7 \text{ kN/m}^3 \]

Below the GWT:

\[ H_{\text{shaft}} = 1.19 \text{ m} \quad \phi'_p = 47 \text{ degrees} \quad c'_p = 0 \text{ kPa} \quad \gamma_p = 11.9 \text{ kN/m}^3 \]

The weighted average of soil engineering properties based on Eqs. 24-26 were estimated as shown in Table 22. The height of the wedge zone \((d\text{ in Eq. 27})\) was used as the height of influence to determine the average properties. Since \(H\) is a function of \(\phi'_a\), and \(\phi'_a\) is a function of the height of influence, a trial and error procedure was needed to determine both factors. The results shown in Table 22 are the final iteration with \(H\) calculated as follows:

\[ H = 0.5 \cdot B \tan(45^\circ + \phi'_a / 2) = 0.5 \cdot 1.98 \cdot \tan(45^\circ + 34^\circ / 2) = 1.86 \text{ m} \]
TABLE 22. Weighted Average of Engineering Soil Properties

<table>
<thead>
<tr>
<th>Layer No. (1)</th>
<th>Depth to Top (m) (2)</th>
<th>$H_i$ (m) (3)</th>
<th>$\gamma_{wet}$ (kN/m$^3$) (4)</th>
<th>$H_i\gamma_{wet}$ (kN/m$^3$) (5)</th>
<th>$\phi'$ deg. (6) (7)</th>
<th>$H_i\tan\phi'$ (8)</th>
<th>$c'$ kPa (9)</th>
<th>$H_i c'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00</td>
<td>0.28</td>
<td>16.0</td>
<td>4.5</td>
<td>39</td>
<td>0.23</td>
<td>7</td>
<td>2.0</td>
</tr>
<tr>
<td>2</td>
<td>0.28</td>
<td>0.56</td>
<td>16.0</td>
<td>9.0</td>
<td>35</td>
<td>0.39</td>
<td>0</td>
<td>0.0</td>
</tr>
<tr>
<td>3</td>
<td>0.84</td>
<td>0.71</td>
<td>17.0</td>
<td>12.1</td>
<td>35</td>
<td>0.50</td>
<td>7</td>
<td>5.0</td>
</tr>
<tr>
<td>4</td>
<td>1.55</td>
<td>0.25</td>
<td>18.9</td>
<td>4.7</td>
<td>29</td>
<td>0.14</td>
<td>10</td>
<td>2.5</td>
</tr>
<tr>
<td>5</td>
<td>1.80</td>
<td>0.06</td>
<td>18.9</td>
<td>1.1</td>
<td>27</td>
<td>0.03</td>
<td>3</td>
<td>0.2</td>
</tr>
<tr>
<td>$\Sigma$</td>
<td>1.86</td>
<td>31.4</td>
<td>1.29</td>
<td>9.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$\gamma_{wet-av} = 16.9$ kN/m$^3$  
$\phi_{av}' = 34$ deg. (Eq. 24)  
$\sigma_v' = 0$  
$c'_{av} = 5$ kPa (Eq. 25)

FULL-SCALE FOOTING ON UNREINFORCED SOIL (FSFOUS)

General Shear Failure

Ultimate bearing capacity of the full-scale footing ($B = L = 1.98$ m) on unreinforced soil for general shear failure is determined as follows assuming $c'_{av}$ is zero:

<table>
<thead>
<tr>
<th>Meyerhof</th>
<th>Hansen</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_q =$</td>
<td>29.44</td>
</tr>
<tr>
<td>$s_q =$</td>
<td>1.35</td>
</tr>
<tr>
<td>$d_q =$</td>
<td>1.00</td>
</tr>
<tr>
<td>$N_c =$</td>
<td>42.16</td>
</tr>
<tr>
<td>$s_c =$</td>
<td>1.71</td>
</tr>
<tr>
<td>$d_c =$</td>
<td>1.00</td>
</tr>
<tr>
<td>$N_f =$</td>
<td>31.15</td>
</tr>
<tr>
<td>$s_f =$</td>
<td>1.35</td>
</tr>
<tr>
<td>$d_f =$</td>
<td>1.00</td>
</tr>
</tbody>
</table>

$q_{ult} = 705$ kPa (Eq. 30)  
$q_{ult} = 289$ kPa (Eq. 32)

Local Shear Failure

Using Terzaghi’s method for local shear failure, tan $\phi'$ and $c'$ are reduced by 33% (Eqs. 52 and 52):

$\phi_{ls}' = \tan^{-1}(0.67 \tan \phi') = \tan^{-1}(0.67 \tan 34^\circ) = 24^\circ$

$c_{ls}' = 0.67 c' = 0.67 \cdot 5 = 3$ kPa
Calculations for $q_{ult}$ for local shear are as follows assuming $c'$ is zero:

<table>
<thead>
<tr>
<th></th>
<th>Meyerhof</th>
<th></th>
<th>Hansen</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_q$</td>
<td>9.60</td>
<td>$s_q$</td>
<td>1.24</td>
</tr>
<tr>
<td>$N_c$</td>
<td>19.32</td>
<td>$s_c$</td>
<td>1.47</td>
</tr>
<tr>
<td>$N_f$</td>
<td>5.72</td>
<td>$s_f$</td>
<td>1.24</td>
</tr>
</tbody>
</table>

$q_{ult} = 118 \text{ kPa (Eq. 30)}$

$q_{ult} = 57.7 \text{ kPa (Eq. 32)}$

Squeezing

Calculation of $q_{ult}$ assuming squeezing failure is shown below for Layer 3.

$z =$ depth to top of Layer 3 $= 0.84 \text{ m}$

\[
\bar{q} = 0.84(16.0) = 13.4 \text{ kPa}
\]

$s_u = 35 \text{ kPa}$

\[
q_{ult,z} = 4s_u + \bar{q} = 4(35) + 13.4 = 153 \text{ kPa}
\]

\[
q_{ult} = q_{ult,z} \cdot \frac{(B + z)^2}{B^2} = 153 \cdot \frac{(1.98 + 0.84)^2}{1.98^2} = 310 \text{ kPa}
\]

FULL-SCALE FOOTING ON REINFORCED SOIL (FSFORS)

In these calculations, Poisson’s ratio for the matrix soil was assumed to 0.5 and the
estimated actual diameter of the piers ($d_{shaft} = 0.686 \text{ m}$) was used rather than the nominal
diameter of the piers ($d_p$). The resulting value of area replacement ratio, $R_a = 0.471$. A
value of stress concentration ratio, $R_c = 10$ was assumed per typical practice.

Bulging Failure of Individual Geopier Elements

Calculations for bulging failure of individual Geopier elements are summarized in Table
23. The controlling layer is Layer 4 with $q_{ult}$ of 842 kPa. Details of the calculations for
Layer 4 are given as an example.
### TABLE 23. Bearing Capacity of FSFORS Assuming Bulging Failure of Piers

<table>
<thead>
<tr>
<th>Layer No. (1)</th>
<th>Depth from GND Surface to Layer</th>
<th>φ′ (deg.) (4)</th>
<th>su (kPa) (5)</th>
<th>Ex (MPa) (6)</th>
<th>σvo (kPa) (7)</th>
<th>Kp,m (8)</th>
<th>σr,o (kPa) (9)</th>
<th>σr,lim (kPa)</th>
<th>qult,p (kPa)</th>
<th>qult (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.28</td>
<td>0.84</td>
<td>35</td>
<td>74</td>
<td>11.3</td>
<td>9</td>
<td>3.69</td>
<td>33</td>
<td>397</td>
<td>3,000</td>
</tr>
<tr>
<td>3</td>
<td>0.84</td>
<td>1.55</td>
<td>35</td>
<td>35</td>
<td>10.1</td>
<td>20</td>
<td>3.69</td>
<td>72</td>
<td>267</td>
<td>2,010</td>
</tr>
<tr>
<td>4</td>
<td>1.55</td>
<td>1.80</td>
<td>29</td>
<td>20</td>
<td>16.8</td>
<td>28</td>
<td>2.88</td>
<td>80</td>
<td>213</td>
<td>1,610</td>
</tr>
</tbody>
</table>

\(^a\text{Eq. 34}\)

\(^b\text{Eq. 33}\)

\(^c\text{Eq. 39}\)

**Example Calculations for Layer 4**

From Table 6: \(\phi' = 29^\circ\), \(s_u = 20\) kPa, and \(E_s = 16.8\) MPa = 16,800 kPa

Depth to midheight of layer = 0.5(1.55 + 1.80) = 1.675 m

\[\sigma_{r,0} = 16.0(0.84) + 17.0(1.55 - 0.84) + 18.9(1.675 - 1.55) = 27.9 \text{ kPa}\]

\[K_{p,m} = \tan^2(45^\circ + \phi'/2) = \tan^2(45^\circ + 29^\circ / 2) = 2.88\]

\[\sigma_{r,0} = \sigma_{r,0} \cdot K_{p,m} = 27.9(2.88) = 80.4 \text{ kPa}\]

\[\sigma_{r,lim} = \sigma_{r,0} + s_u \left[1 + \ln\left(\frac{E}{2s_u(1+\mu)}\right)\right] = 80.4 + 20\left[1 + \ln\left(\frac{16,800}{2 \cdot 20(1+0.5)}\right)\right] = 213 \text{ kPa}\]

\[K_{pp} = \tan^2(45^\circ + \phi'_p / 2) = \tan^2(45^\circ + 50^\circ / 2) = 7.55\]

\[q_{ult,p} = \sigma_{r,lim} \cdot K_{pp} = 213(7.55) = 1,610 \text{ kPa}\]

\[\mu_p = \frac{R_s}{R_s(R_s - 1) + 1} = \frac{10}{0.471(10-1)+1} = 1.91\]

\[q_{ult} = \frac{q_{ult,p}}{\mu_p} = \frac{1,610}{1.91} = 842 \text{ kPa}\]
Shearing Below Bottom of Individual Geopier Elements

End Bearing of Geopier Element

Properties of the Geopier elements are as follows:

- \( B = 0.608 \) m (equivalent square width)
- \( d_{\text{shaft}} = 0.686 \) m
- \( \gamma' = 9.0 \) kN/m\(^3\)
- \( D_f = 3.05 \) m
- \( H_{\text{shaft}} = 3.05 \) m
- \( \phi'_m = 31^\circ \)
- \( c'_m = 0 \) kPa
- \( \sigma'_{v} = 42.8 \) kPa

<table>
<thead>
<tr>
<th>Meyerhof</th>
<th>Hansen</th>
</tr>
</thead>
<tbody>
<tr>
<td>( N_q = 20.63 )</td>
<td>( N_q = 20.63 )</td>
</tr>
<tr>
<td>( s_q = 1.31 )</td>
<td>( s_q = 1.52 )</td>
</tr>
<tr>
<td>( d_q = 1.89 )</td>
<td>( d_q = 1.38 )</td>
</tr>
<tr>
<td>( N_c = 32.67 )</td>
<td>( N_c = 32.67 )</td>
</tr>
<tr>
<td>( s_c = 1.63 )</td>
<td>( s_c = 1.63 )</td>
</tr>
<tr>
<td>( d_c = 2.77 )</td>
<td>( d_c = 1.54 )</td>
</tr>
<tr>
<td>( N_{\gamma'f} = 18.56 )</td>
<td>( N_{\gamma'f} = 17.69 )</td>
</tr>
<tr>
<td>( s_{\gamma'f} = 1.31 )</td>
<td>( s_{\gamma'f} = 0.60 )</td>
</tr>
<tr>
<td>( d_{\gamma'f} = 1.89 )</td>
<td>( d_{\gamma'f} = 1.00 )</td>
</tr>
</tbody>
</table>

Skin Friction Along Shaft of Geopier Element

Values of ultimate stress at the top of the pier resulting from each increment of skin friction are summarized in Table 24. Sample calculations are shown below for Layer 4.

\( \Delta z = 1.80 - 1.55 = 0.25 \) m

\( \sigma_v = 27.9 \) kPa (from previous calculations)

\( K_{pm} = 2.88 \) (from previous calculations)

\( f_s = \sigma_v \cdot K_{pm} \cdot \tan \phi_m = (27.9)(2.88)(\tan 29^\circ) = 44.5 \) kPa

\( p = \pi \cdot d_{\text{shaft}} = \pi(0.686) = 2.15 \) m

\( A_{\text{shaft}} = 0.25 \pi (d_{\text{shaft}})^2 = 0.25 \pi (0.686)^2 = 0.369 \) m\(^2\)

\( f_s \cdot \Delta z \cdot p / A_{\text{shaft}} = (44.5)(0.25)(2.15) / (0.369) = 64.1 \) kPa
TABLE 24. Skin Friction Along Perimeter of Geopier Element

<table>
<thead>
<tr>
<th>Layer No. (1)</th>
<th>Top (m)</th>
<th>Bot. (m)</th>
<th>∆zi (m)</th>
<th>γ or γ' (kN/m³)</th>
<th>σvo or σvo (kPa)</th>
<th>φ' (deg.)</th>
<th>Kp,m (kPa)</th>
<th>fs (kPa)</th>
<th>f_s·p·∆zi/A_shaft (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00</td>
<td>0.28</td>
<td>0.28</td>
<td>16.0</td>
<td>2.2</td>
<td>39</td>
<td>4.40</td>
<td>8.0</td>
<td>13</td>
</tr>
<tr>
<td>2</td>
<td>0.28</td>
<td>0.84</td>
<td>0.56</td>
<td>16.0</td>
<td>9.0</td>
<td>35</td>
<td>3.69</td>
<td>23.2</td>
<td>76</td>
</tr>
<tr>
<td>3</td>
<td>0.84</td>
<td>1.55</td>
<td>0.71</td>
<td>17.0</td>
<td>19.5</td>
<td>35</td>
<td>3.69</td>
<td>50.4</td>
<td>209</td>
</tr>
<tr>
<td>4</td>
<td>1.55</td>
<td>1.80</td>
<td>0.25</td>
<td>18.9</td>
<td>27.9</td>
<td>29</td>
<td>2.88</td>
<td>44.5</td>
<td>64</td>
</tr>
<tr>
<td>5a</td>
<td>1.80</td>
<td>1.86</td>
<td>0.06</td>
<td>18.9</td>
<td>30.8</td>
<td>27</td>
<td>2.66</td>
<td>41.7</td>
<td>15</td>
</tr>
<tr>
<td>5b</td>
<td>1.86</td>
<td>2.19</td>
<td>0.33</td>
<td>10.1</td>
<td>33.0</td>
<td>27</td>
<td>2.66</td>
<td>44.8</td>
<td>87</td>
</tr>
<tr>
<td>6</td>
<td>2.19</td>
<td>2.59</td>
<td>0.40</td>
<td>10.1</td>
<td>36.7</td>
<td>27</td>
<td>2.66</td>
<td>49.8</td>
<td>116</td>
</tr>
<tr>
<td>7</td>
<td>2.59</td>
<td>3.02</td>
<td>0.43</td>
<td>9.0</td>
<td>40.7</td>
<td>33</td>
<td>3.39</td>
<td>89.6</td>
<td>223</td>
</tr>
<tr>
<td>8</td>
<td>3.02</td>
<td>3.05</td>
<td>0.03</td>
<td>9.0</td>
<td>42.8</td>
<td>31</td>
<td>3.12</td>
<td>80.3</td>
<td>15</td>
</tr>
</tbody>
</table>

Σ f_s·p·∆zi/A_shaft = 818

The ultimate stress at top of the Geopier element is calculated as follows using Eq. 39:

\[ q_{ult,p} = \sum_{i=1}^{n} \left( \frac{f_s \cdot \Delta z_i \cdot p_i}{A_{shaft}} \right) + q_{ult,bot,p} = 2,320 + 820 = 3,140 \text{ kPa (Meyerhof)} \]

\[ = 1,890 + 820 = 2,710 \text{ kPa (Hansen)} \]

Ultimate bearing capacity of the footing is determined using Eq. 41 with \( \mu_p = 1.91 \):

\[ q_{ult} = q_{ult,p} / \mu_p = 3,140 / 1.91 = 1,640 \text{ kPa (Meyerhof)} \]

\[ = 2,710 / 1.91 = 1,420 \text{ kPa (Hansen)} \]

Shearing Within Geopier-Reinforced Composite Soil

\( B = L = 3.05 \text{ m} \)

\( R_a = 0.471 \) (from previous calculations)

\( R'_a = 0.4 \cdot R_a = 0.4(0.471) = 0.188 \)

\( R'_s = 2.8 \)

\( \phi'_{m-av} = 34^\circ \) (from previous calculations)
\( \gamma_{m-av} = 16.9 \text{ kN/m}^3 \) (from previous calculations)

\( \phi'_p = 50^\circ \) (above GWT)

\[
\mu'_p = \frac{R'_a}{R'_a(R'_a - 1) + 1} = \frac{2.8}{0.188(2.8 - 1) + 1} = 2.09 \quad \text{(Eq. 17)}
\]

\[
\mu'_m = \frac{1}{R'_a(R'_a - 1) + 1} = \frac{1}{0.188(2.8 - 1) + 1} = 0.747 \quad \text{(Eq. 18)}
\]

\[
\phi'_{comp} = \tan^{-1}[\mu'_p \cdot R'_a \cdot \tan \phi'_p + \mu'_m (1 - R'_a) \tan \phi'_{m-av}]
= \tan^{-1}[(2.09)(0.188)(\tan 50^\circ) + (0.747)(1 - 0.188)(\tan 34^\circ)] = 41^\circ \quad \text{(Eq. 42)}
\]

\( c'_{comp} = 0 \text{ kPa} \) (Eq. 43)

\[
\gamma_{comp} = \gamma_p \cdot R'_a + \gamma_{m-av} (1 - R'_a) = (21.7)(0.188) + (16.9)(1 - 0.188) = 17.8 \text{ kN/m}^3 \quad \text{(Eq. 44)}
\]

\( \bar{q} = 0 \text{ kPa} \)

<table>
<thead>
<tr>
<th>Meyerhof</th>
<th>Hansen</th>
</tr>
</thead>
<tbody>
<tr>
<td>( N_q = 73.90 )</td>
<td>( N_q = 73.90 )</td>
</tr>
<tr>
<td>( s_q = 1.48 )</td>
<td>( s_q = 1.88 )</td>
</tr>
<tr>
<td>( d_q = 1.00 )</td>
<td>( d_q = 1.00 )</td>
</tr>
<tr>
<td>( N_c = 83.86 )</td>
<td>( N_c = 83.86 )</td>
</tr>
<tr>
<td>( s_c = 1.96 )</td>
<td>( s_c = 1.66 )</td>
</tr>
<tr>
<td>( d_c = 1.00 )</td>
<td>( d_c = 1.00 )</td>
</tr>
<tr>
<td>( N_f = 113.99 )</td>
<td>( N_f = 95.05 )</td>
</tr>
<tr>
<td>( s_f = 1.48 )</td>
<td>( s_f = 0.60 )</td>
</tr>
<tr>
<td>( d_f = 1.00 )</td>
<td>( d_f = 1.00 )</td>
</tr>
</tbody>
</table>

\( q_{ult, footing} = 2.980 \text{ kPa} \) (Eq. 29)

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>( q_{ult, footing} = 1.010 \text{ kPa} ) (Eq. 31)</td>
<td></td>
</tr>
</tbody>
</table>

**Shearing Below Geopier-Reinforced Composite Soil**

The assumed bearing area is located at the bottom of the upper zone, which gives:

\( D_f = 3.05 \text{ m} \)

\( B' = B + H = 1.98 + 3.05 = 5.03 \text{ m} \) (Eq. 46)

\( L' = L + H = 1.98 + 3.05 = 5.03 \text{ m} \) (Eq. 45)

\( \sigma'_v = 42.8 \text{ kPa} \) (from previous calculations)

\( \phi'_m = 31^\circ, \ c'_m = 0 \text{ kPa}, \ \gamma'_m = 9.0 \text{ kN/m}^3 \)

\( r_\gamma = 1 - 0.25 \log(B'/2) = 1 - 0.25 \log(5.03/2) = 0.900 \) (see Bowles 1996, p. 230)
\[ \eta = \frac{B' L'}{B L} = \frac{(5.03)^2}{(1.98)^2} = 6.44 \quad \text{(Eq. 46)} \]

\[ q_{ult, footing} = q_{ult, bottom, p} \cdot \eta = 1,830 \cdot 6.44 = 11,800 \text{ kPa (Meyerhof, Eq. 47)} \]

\[ q_{ult, footing} = q_{ult, bottom, p} \cdot \eta = 1,780 \cdot 6.44 = 11,500 \text{ kPa (Hansen, Eq. 47)} \]
APPENDIX D

EXAMPLE CALCULATIONS OF SETTLEMENT
FOR FOOTINGS ON UNREINFORCED
AND REINFORCED SOILS
HEIGHT OF INFLUENCE

The height of influence for immediate settlement calculated using Schmertmann’s method was $2B = 3.96$ m. The height of influence for immediate settlement using Bowles method is defined as the lesser or $5B$ or the depth to a hard layer, where a hard layer is defined as one that has a value of $E_r$ greater than 10 times the value for the layer immediately above it. However, at this site the upper soils (to a depth of about 4.85 m) are much stiffer than the underlying Lake Bonneville deposits and thus the entire upper zone can be considered a hard layer. Therefore, a height of influence of 4.85 m will be used for Bowles’ method. With regard to primary consolidation settlement, the upper hard layers likely reduce the stresses induced to the lower soils to an insignificant level. Therefore, a height of influence of 4.85 m was also used to calculate primary consolidation settlement.

FULL-SCALE FOOTING ON UNREINFORCED SOIL (FSFOS)

In this example, calculation of total settlement ($S_t$) will be shown for a vertical compressive load ($V$) of 417 kN. $S_t$ is based on Eq. 1 except that in the footing tests there was no settlement from secondary compression ($S_s = 0$) or changes in moisture condition ($S_m = 0$). Therefore, the governing equation is simplified to $S_t = S_i + S_c$.

Immediate Settlement ($S_i$)

Schmertmann’s Strain Influence Method

The soil properties in Tables 6 and 7 are used in the estimation of $S_i$. The peak value of strain influence factor is calculated as follows:

$\Delta q = \frac{V}{B^2} = \frac{417}{(1.98)^2} = 106$ kPa

$\sigma_{vp}(z = 0.5B) = 16.0(0.84) + 17.0(0.99 - 0.84) = 16.0$ kPa
\[ I_{zp} = 0.5 + 0.1 \sqrt{\frac{\Delta q}{\sigma_{vp}}} = 0.5 + 0.1 \sqrt{\frac{106}{16.0}} = 0.758 \quad (\text{Eq. 6}) \]

Slope of upper strain influence line = \((0.758 - 0.100) / 0.991 = 0.664\)

Equation for upper influence line: \(I_z = 0.1 + 0.664z\) (valid for \(z \leq 0.991\) m)

Slope of lower strain influence line = \(-0.758 / (3.96 - 0.99) = -0.255\)

Equation for lower influence line: \(I_z = 0.758 - 0.255(z-0.991) = 1.01 - 0.255z\) (valid for \(z > 0.991\) m)

A diagram of \(I_z\) versus \(z\) is given in Fig. 85. Calculation of \(S_i\) for each layer is shown in Table 25.

FIG. 85. Schmertmann’s Strain Influence Factor versus Depth Used to Calculate Immediate Settlement of FSFOUS
TABLE 25. Immediate Settlement of FSFOUS by Schmertmann’s Method

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>$\Delta z_i$ (m)</th>
<th>$z_i$ (m)</th>
<th>$I_{z,i}$ (m)</th>
<th>$E_s$ (kPa)</th>
<th>$S_{i,i}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.280</td>
<td>0.140</td>
<td>0.193</td>
<td>39,600</td>
<td>0.14</td>
</tr>
<tr>
<td>2</td>
<td>0.558</td>
<td>0.559</td>
<td>0.471</td>
<td>11,300</td>
<td>2.47</td>
</tr>
<tr>
<td>3a</td>
<td>0.153</td>
<td>0.914</td>
<td>0.707</td>
<td>10,100</td>
<td>1.13</td>
</tr>
<tr>
<td>3b</td>
<td>0.560</td>
<td>1.271</td>
<td>0.686</td>
<td>10,100</td>
<td>4.05</td>
</tr>
<tr>
<td>4</td>
<td>0.253</td>
<td>1.671</td>
<td>0.583</td>
<td>16,800</td>
<td>0.93</td>
</tr>
<tr>
<td>5</td>
<td>0.381</td>
<td>1.995</td>
<td>0.502</td>
<td>16,700</td>
<td>1.22</td>
</tr>
<tr>
<td>6</td>
<td>0.405</td>
<td>2.388</td>
<td>0.402</td>
<td>10,900</td>
<td>1.58</td>
</tr>
<tr>
<td>7</td>
<td>0.433</td>
<td>2.807</td>
<td>0.295</td>
<td>25,700</td>
<td>0.53</td>
</tr>
<tr>
<td>8</td>
<td>0.279</td>
<td>3.163</td>
<td>0.204</td>
<td>10,700</td>
<td>0.56</td>
</tr>
<tr>
<td>9</td>
<td>0.267</td>
<td>3.436</td>
<td>0.134</td>
<td>19,900</td>
<td>0.19</td>
</tr>
<tr>
<td>10</td>
<td>0.228</td>
<td>3.683</td>
<td>0.071</td>
<td>15,600</td>
<td>0.11</td>
</tr>
<tr>
<td>11</td>
<td>0.165</td>
<td>3.880</td>
<td>0.021</td>
<td>16,900</td>
<td>0.02</td>
</tr>
</tbody>
</table>

$\Sigma \Delta z_i = 2B = 3.963$  \hspace{1cm} $\Sigma S_{i,i} = S_i = 12.95$

Sample calculations are given below for Layer 4.

$\Delta z_i = 1.80 - 1.55 = 0.25$ m

$z_i = 1.55 + 0.5(0.25) = 1.675$ m

$I_{z,i} = 1.01 - 0.255(1.675) = 0.583$

$C_I = 1.00$ (no embedment)

$E_{si} = 16.8$ MPa = 16,800 kPa (from Table 5)

$S_{i,j} = C_I \Delta q \left( \frac{I_z}{E_s} \right)_i \Delta z_i = 1.00(106) \left( \frac{0.583}{16,800} \right)(0.25) = 0.00092$ m = 0.92 mm

From Table 25, the calculated value of $S_j$ for this load is 13.0 mm. In practice, the calculated value of $S_j$ is commonly reduced by one-third owing to the tendency of Schmertmann’s method to overpredict $S_j$. Thus, $S_j$ for this example is calculated as $(2/3)(13.0) = 8.7$ mm.
Bowles’ Modified Elastic Theory

Bowles’ method is valid for a flexible footing. To calculate immediate settlement at the center of the flexible loaded area, the footing is divided into four equal rectangles with the center of the footing at the corner of each rectangle. The average value of $E_s$ is calculated in Table 26. As discussed previously, a height of influence, $H = 4.85$ m was used. $S_i$ is calculated as follows (see Bowles 1996, pp. 303-310):

$$B' = \frac{B}{2} = 0.991 \text{ m}, \quad L' = \frac{L}{2} = 0.991 \text{ m}$$

$$M = \frac{L'}{B'} = 1.0, \quad N = \frac{H}{B'} = 4.85 / 0.991 = 4.89$$

$$I_1 = 0.435, \quad I_2 = 0.031, \quad I_F = 1.00$$

Assuming average Poisson’s Ratio, $\mu_{avg} = 0.40$:

$$I_s = I_1 + \frac{1 - 2 \mu}{1 - \mu} I_2 = 0.445$$ \hspace{1cm} (Eq. 8)

$$S_i = 4\Delta q B' \frac{1 - \mu^2}{E_s} I_s I_F = 4(106)(991)\frac{1 - 0.40^2}{16,500}(0.445)(1.00) = 9.6 \text{ mm}$$ \hspace{1cm} (Eq. 7)

This value of $S_i$ is valid for a flexible footing at the center of the loaded area. Multiplying this value by 0.73 to account for the rigidity of the footing:

$$S_i(\text{rigid}) = 0.73 \times S_i(\text{flexible, center}) = 0.73(9.6) = 7.0 \text{ mm}$$

Primary Consolidation Settlement ($S_c$)

Soil properties used to calculate primary consolidation settlement are summarized in Table 7. Certain properties for some layers were unknown, so regression analyses (Figs. 86-88) and engineering judgment were used to estimate the soil properties for the unknown layers (Tables 27-29). This information was used in Eqs. 9 to 11, to predict $S_c$. $S_c$ from this induced load is 7.6 to 9.4 mm. The combinations of these settlements give $S_i$ of 14.6 to 18.2 mm, compared to the measured value of $S_i$ of 6 mm.
TABLE 26. Determination of $E_s$ for Immediate Settlement by Bowles Method for FSFOUS

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>$q_c$-avg (kPa)</th>
<th>No. for $E_s$</th>
<th>$E_{s,NC}$ (MPa)</th>
<th>OCR</th>
<th>$E_{s,OC}$ (MPa)</th>
<th>$\Delta z$ (m)</th>
<th>$E_s*\Delta z$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2,817</td>
<td>3</td>
<td>8.5</td>
<td>22</td>
<td>39.6</td>
<td>0.28</td>
<td>11.1</td>
</tr>
<tr>
<td>2</td>
<td>1,214</td>
<td>3</td>
<td>3.6</td>
<td>10</td>
<td>11.3</td>
<td>0.56</td>
<td>6.3</td>
</tr>
<tr>
<td>3</td>
<td>872</td>
<td>6</td>
<td>5.2</td>
<td>4</td>
<td>10.1</td>
<td>0.71</td>
<td>7.2</td>
</tr>
<tr>
<td>4</td>
<td>1,061</td>
<td>6</td>
<td>6.4</td>
<td>7</td>
<td>16.8</td>
<td>0.25</td>
<td>4.3</td>
</tr>
<tr>
<td>5</td>
<td>3,205</td>
<td>3</td>
<td>9.6</td>
<td>3</td>
<td>16.7</td>
<td>0.38</td>
<td>6.3</td>
</tr>
<tr>
<td>6</td>
<td>1,369</td>
<td>3</td>
<td>4.1</td>
<td>7</td>
<td>10.9</td>
<td>0.41</td>
<td>4.4</td>
</tr>
<tr>
<td>7</td>
<td>1,434</td>
<td>6</td>
<td>8.6</td>
<td>9</td>
<td>25.7</td>
<td>0.43</td>
<td>11.1</td>
</tr>
<tr>
<td>8</td>
<td>1,355</td>
<td>3</td>
<td>4.1</td>
<td>7</td>
<td>10.7</td>
<td>0.28</td>
<td>3.0</td>
</tr>
<tr>
<td>9</td>
<td>1,440</td>
<td>6</td>
<td>8.6</td>
<td>5</td>
<td>19.9</td>
<td>0.27</td>
<td>5.3</td>
</tr>
<tr>
<td>10</td>
<td>2,261</td>
<td>6</td>
<td>6.8</td>
<td>5</td>
<td>15.6</td>
<td>0.23</td>
<td>3.6</td>
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<tr>
<td>11</td>
<td>1,525</td>
<td>6</td>
<td>9.1</td>
<td>3</td>
<td>16.9</td>
<td>0.19</td>
<td>3.2</td>
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<tr>
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<td>1,008</td>
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<td>3.0</td>
<td>5</td>
<td>7.0</td>
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<td>--</td>
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<tr>
<td>13</td>
<td>711</td>
<td>6</td>
<td>4.3</td>
<td>4</td>
<td>8.2</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>14</td>
<td>1,083</td>
<td>6</td>
<td>6.5</td>
<td>3</td>
<td>11.3</td>
<td>--</td>
<td>--</td>
</tr>
</tbody>
</table>

$E_{s,avg}$ (MPa) = 16.5

3.99 65.8
FIG. 86. Regression Analysis for Virgin Compression Index Using $E_s$

\[ C_c = 0.2762 - 0.005683 * E_s \]

FIG. 87 Regression Analysis for Virgin Compression Index Using $LL$

\[ C_c = 0.1672 + 0.0003932 * LL \]
FIG. 88. Regression Analysis for Virgin Compression Index Using $q_c$

$$C_c = 0.3009 - 0.00009112 q_c$$
TABLE 27. Summary of Primary Consolidation Settlement for Vertical Induced Load of 417 kN on FSFOUS with Westergaard Stress Distribution

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>$\Delta z_i$ (m)</th>
<th>$z_i$ (m)</th>
<th>$I_{zi}$</th>
<th>$\Delta q_i$ (kPa)</th>
<th>$\sigma'_{v0i}$ (kPa)</th>
<th>$\sigma'_{vpj}$ (kPa)</th>
<th>$\sigma'_{vlj}$ (kPa)</th>
<th>$e_{0i}$ (9)</th>
<th>$e_{pi}$ (10)</th>
<th>$c_r$ (11)</th>
<th>$c_c$ (12)</th>
<th>$S_{c,i}$ (mm) (13)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.280</td>
<td>0.140</td>
<td>2.12</td>
<td>97</td>
<td>2</td>
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<td>0.72</td>
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<td>0.210</td>
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<td>239</td>
<td>64</td>
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<td>4.801</td>
<td>6.78</td>
<td>5</td>
<td>61</td>
<td>184</td>
<td>67</td>
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<td>0.95</td>
<td>0.025</td>
<td>0.200</td>
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</tr>
</tbody>
</table>

$^a$ Vertical stress dissipation influence factor

$^b$ Correction Factor of 1D consolidation to 3D using Leonards and Skemptions and Bjerrums

(Note: $S_c (1D) = 12.3$ and $S_c (3D) = 7.6$)

Note: 1 m = 0.305; 1 kPa = 0.021 ksft; 1 mm = 0.039 in.)
TABLE 28. Summary of Primary Consolidation Settlement for Vertical Induced Load of 417 kN on FSFOUS with Newmark Stress Distribution

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>$\Delta z_i$ (m)</th>
<th>$z_i$ (m)</th>
<th>$I_{zi}^a$</th>
<th>$\Delta q_i$ (kPa)</th>
<th>$\sigma'_{\text{v0i}}$ (kPa)</th>
<th>$\sigma'_{\text{vpi}}$ (kPa)</th>
<th>$\sigma'_{\text{vli}}$ (kPa)</th>
<th>$e_{\text{oi}}$ (9)</th>
<th>$e_{\text{pi}}$ (9)</th>
<th>$c_r$ (11)</th>
<th>$c_c$ (12)</th>
<th>$S_{c,i}$ (mm) (13)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.280</td>
<td>0.140</td>
<td>2.12</td>
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<td>2</td>
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<td>0.72</td>
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</tr>
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<td>0.91</td>
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<td>0.85</td>
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<td>0.146</td>
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<td>0.95</td>
<td>0.025</td>
<td>0.200</td>
<td>0.07</td>
<td></td>
</tr>
</tbody>
</table>

$^a$ Vertical stress dissipation influence factor

$^b$ Correction Factor of 1D consolidation to 3D using Leonards and Skemptions and Bjerrums

Note: 1 m = 0.305; 1 kPa = 0.021 ksft; 1 mm = 0.039 in.

$S_c (1D) = 15.2$

$S_c (3D) = 9.4$
### TABLE 29. Summary of Primary Consolidation Settlement for Vertical Induced Load of 417 kN on FSFOUS with 2:1 Stress Distribution

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Δz_j (m)</th>
<th>z_i (m)</th>
<th>B_{zi} = L_{zi}</th>
<th>Ωq_i (kPa)</th>
<th>σ'_{v0i} (kPa)</th>
<th>σ'_{vpi} (kPa)</th>
<th>σ'_{vli} (kPa)</th>
<th>ε_{0i} (kPa)</th>
<th>ε_{pi} (kPa)</th>
<th>c_r</th>
<th>c_c (mm)</th>
<th>S_{c,i} (1D)</th>
<th>S_{c,i} (3D)</th>
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<tr>
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<td>0.280</td>
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<td>2.121</td>
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<td>70</td>
<td>0.98</td>
<td>0.95</td>
<td>0.025</td>
<td>0.200</td>
<td>0.07</td>
<td></td>
</tr>
</tbody>
</table>

*a* Length and width used to calculated vertical stress at depth

*b* Correction Factor of 1D consolidation to 3D using Leonards and Skempton's and Bjerrums

Note: 1 m = 0.305; 1 kPa = 0.021 ksf; 1 mm = 0.039 in.
Total Settlement from Stress Path Tests

Results from the stress path tests allowed estimates to be made for the combination of $S_i$ and $S_c$. In Fig. 89, the contact stress-strain results are summarized for the stress path tests with stresses converted to surface contact stress. Four layers were used with thicknesses of 0.55, 0.95, 1.30, and 1.30 m. The settlement is calculated as follows:

$$S_t = [(0.0138)(0.55) + (0.0068)(0.95) + (0.0014 + 0.0002)(1.30)] / 1000 = 16 \text{ mm}$$

Thus, the predicted value of $S_t$ is 16 mm, compared to the measured value of 6 mm.
FULL-SCALE FOOTING ON REINFORCED SOIL (FSFORS)

The total settlement for the footing supported by Geopier elements was estimated using Eq. 14. The example settlement calculations for the FSFORS will be based on a vertical load ($V$) of 1,810 kN, which produced an average bearing stress ($\Delta q = V / B^2$) of 461 kPa. The corresponding value of measured total settlement ($S_t$) was 19 mm.

**Upper Zone**

Settlement of the upper zone ($S_{uz}$) was calculated using Eq. 21. Curves of settlement versus subgrade modulus and stress concentration ratio are presented in Fig. 73. If the settlement is initially assumed to be 9.0 mm and $q_0$ is 461 kPa, then $k_p$ is 127,000 kN/m$^3$ and $R_s$ is 14. Using Eq. 17, $q_p$ is 1,105 kPa. Substituting $q_p$ and $k_p$ into Eq. 21 gives a value $S_{uz}$ of 8.7 mm, which is not equal to the initial predicted settlement. A second settlement prediction of 8.7 mm is used with Fig. 73, giving $k_p = 127,000$ kN/m$^3$ and $R_s = 14$, with $q_p = 1,105$ kPa. The value of $S_{uz}$ from the second iteration equals 8.7 mm, which is equal to the initial prediction. Using $k_m$ assuming soil type 1 and an assumed settlement of 4.0 mm, $k_m = 22,500$ kN/m$^3$ is obtained from Fig. 47 and $R_s = 11.8$ is obtained from Fig. 73, giving $q_m = 91.9$ kPa. From Eq. 21, $S_{uz}$ is 4.1 mm (0.28 in). $k_m$ assuming soil type 2 and an assumed settlement of 7.0 mm, obtain $k_m = 11,500$ kN/m$^3$ from Fig. 47 and $R_s$ is 13.5 from Fig. 73, giving $q_m = 81.6$ kPa. From Eq. 21, $S_{uz}$ is 7.1 mm.
Lower Zone

Calculation of immediate settlement for the lower zone is summarized in Table 30. Note that because the contact stress (\(\Delta q\)) is different for the FSFORS than for the FSFOUS, the \(I_z\) vs. \(z\) relationship is also different. Using the same method as shown for the FSFOUS, \(I_{zp}\) and the equation for the lower line of the \(I_z\) vs. \(z\) relationship were calculated as follows:

\[
I_{zp} = 0.5 + 0.1 \sqrt{\frac{\Delta q}{\sigma_{vp}}} = 0.5 + 0.1 \sqrt{\frac{461}{16.0}} = 1.037
\]  \hspace{1cm} \text{(Eq. 6)}

For \(z > 0.991\) m: \(I_z = 1.382 - 0.349z\)

The top of the upper zone begins at a depth of 3.048 m below the ground surface. The depth of influence for immediate settlement using Schmertmann’s method is \(2B = 3.962\) m. Thus, the total thickness of the lower zone used to calculate \(S_i\) was \(3.962 - 3.048 = 0.914\) m. \(S_i\) for the lower zone was calculated to be 4.9 mm. After multiplying by the reduction factor, \(S_i = 2 / 3 * 4.9 = 3.3\) mm.

<table>
<thead>
<tr>
<th>Layer No. a</th>
<th>(\Delta z_i) (m)</th>
<th>(z_i) (m)</th>
<th>(I_{z,i}) (4)</th>
<th>(E_s) (kPa)</th>
<th>(S_{i,i}) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>0.254</td>
<td>3.175</td>
<td>0.275</td>
<td>10,700</td>
<td>3.01</td>
</tr>
<tr>
<td>9</td>
<td>0.267</td>
<td>3.436</td>
<td>0.184</td>
<td>19,900</td>
<td>1.14</td>
</tr>
<tr>
<td>10</td>
<td>0.228</td>
<td>3.683</td>
<td>0.097</td>
<td>15,600</td>
<td>0.66</td>
</tr>
<tr>
<td>11</td>
<td>0.165</td>
<td>3.880</td>
<td>0.029</td>
<td>16,900</td>
<td>0.13</td>
</tr>
<tr>
<td>(\Sigma \Delta z_i = 0.914)</td>
<td>(\Sigma S_{i,i} = S_i = 4.94)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

aLayer No. refers to numbering system used in Table 5.
Calculations for \( S_c \) of the lower zone are summarized in Table 31. Eqs. 9 to 11 were used to predict \( S_c \). The 1.67:1 stress dissipation was used to estimate stresses at the midheight of each layer. \( S_c \) from this example is 5.7 mm. The summation of all components of settlements gives an estimate of total settlement as follows:

\[
S_t = S_{UZ} + S_{LZ} + S_{C,LZ} = 8.7 + 3.3 + 5.7 = 17.7 \text{ mm} \quad (\text{Eq. 14})
\]

The value of \( S_{UZ} \) calculated based on \( k_p \) was used above because it is the standard method used in practice. The predicted value of total settlement is about 18 mm compared to the measured value of 19 mm.
<table>
<thead>
<tr>
<th>Layer No.</th>
<th>z (m)</th>
<th>ΔZ (m)</th>
<th>B = L/2 (m)</th>
<th>B = L/2 (m)</th>
<th>Δq (kPa)</th>
<th>q (kPa)</th>
<th>σ (kPa)</th>
<th>σ (kPa)</th>
<th>e_pl</th>
<th>e_pl</th>
<th>c_v (kPa)</th>
<th>c_u (kPa)</th>
<th>S_{cd} (mm)</th>
<th>S_{cl} (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.830</td>
<td>0.915</td>
<td>3.077</td>
<td>1.91</td>
<td>22</td>
<td>22</td>
<td>213</td>
<td>213</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>1.218</td>
<td>2.439</td>
<td>4.902</td>
<td>75</td>
<td>51</td>
<td>51</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>0.255</td>
<td>3.176</td>
<td>5.784</td>
<td>49</td>
<td>59</td>
<td>59</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>0.267</td>
<td>3.436</td>
<td>6.097</td>
<td>49</td>
<td>62</td>
<td>62</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>0.228</td>
<td>3.684</td>
<td>6.393</td>
<td>44</td>
<td>64</td>
<td>64</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>0.272</td>
<td>3.894</td>
<td>6.644</td>
<td>41</td>
<td>66</td>
<td>66</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>0.253</td>
<td>4.116</td>
<td>6.911</td>
<td>38</td>
<td>68</td>
<td>68</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>8</td>
<td>0.284</td>
<td>4.497</td>
<td>7.367</td>
<td>33</td>
<td>72</td>
<td>72</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9</td>
<td>0.259</td>
<td>4.801</td>
<td>7.731</td>
<td>30</td>
<td>105</td>
<td>105</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Note: 1 m = 0.305 ft; 1 kPa = 0.021 ksi; 1 mm = 0.039 in.
APPENDIX E

BST AND DST RESULTS
This appendix contains results of BST and DST testing. The ground surface elevation was 1288.283 m (4226.86 ft), and the GWT was 1.859 m (6.10 ft) below the ground surface. Figs. 90-97 were from the FSFOUS area, Figs. 98-107 were from the full-scale pier group area, Figs. 108-110 were from the unit cell area.
FIG. 90. BST and DST Results Beneath the FSFOUS, Soil Type was Sandy Elastic SILT (MH), Depth of 0.28 m (1ft): (a) BST; and (b) DSTM
**FIG. 91. BST and DST Results Beneath the FSFOUS, Soil Type was Fat CLAY (CH), Depth of 1.16 m (4ft):** (a) BST; and (b) DSTM
FIG. 92. BST and DST Results Beneath the FSFOUS, Soil Type was Fat CLAY (CH), Depth of 1.50 m (4.9 ft): (a) BST; and (b) DSTM
FIG. 93. BST and DST Results Beneath the FSFOUS, Soil Type was Sandy Lean CLAY (CL), Depth of 1.80 m (5.9 ft): (a) BST; and (b) DSTM
FIG. 94. BST and DST Results Beneath the FSFOUS, Soil Type was Silty SAND (SM), Depth of 2.11 m (6.9 ft): (a) BST; and (b) DSTM
FIG. 95. BST and DST Results Beneath the FSFOUS, Soil Type was Lean CLAY (CL), Depth of 2.72 m (8.9 ft): (a) BST; and (b) DSTM
FIG. 96. BST and DST Results Beneath the FSFOUS, Soil Type was Lean CLAY (CL), Depth of 3.94 m (12.9 ft): (a) BST; and (b) DSTM
FIG. 97. BST and DST Results Beneath the FSFOUS, Soil Type was Lean CLAY (CL), Depth of 4.55 m (14.9 ft): (a) BST; and (b) DSTM
FIG. 98. BST Results Beneath the FSFORS, Soil Type was Sandy Elastic SILT (MH), Depth of 0.18 m (0.6 ft)

FIG. 99. BST Results Beneath the FSFORS, Soil Type was Sandy Elastic SILT (MH), Depth of 0.79 m (2.6 ft)
FIG. 100. BST Results Beneath the FSFORS, Soil Type was Fat CLAY (CH), Depth of 1.40 m (4.59 ft)

FIG. 101. BST Results Beneath the FSFORS, Soil Type was Silty SAND (SM), Depth of 2.01 m (6.6 ft)
FIG. 102. BST Results Beneath the FSFORS, Soil Type was Lean CLAY (CL), Depth of 2.62 m (8.6 ft)

FIG. 103. BST Results Beneath the FSFORS, Soil Type was SILT with Sand (ML), Depth of 3.23 m (10.6 ft)
FIG. 104. BST Results Beneath the FSFORS, Soil Type was Lean CLAY (CL), Depth of 3.84 m (12.6 ft)

FIG. 105. BST Results Beneath the FSFORS, Soil Type was Lean CLAY (CL), Depth of 4.45 m (14.6 ft)
FIG. 106. BST Results Beneath the FSFORS, Depth of 5.06 m (16.06 ft)

\[ \tau_{BST} = 0.7638\sigma_{BST} + 0.0475 \]

\[ \sigma_{BST} = 0.0475 \text{ kPa} \]

\[ \phi_{BST} = 37.4^\circ \]

Consolidation Time = 0 minutes
Consolidation Time = 5 minutes
Consolidation Time = 15 minutes

1 kPa = 20.886 psf

FIG. 107. BST Results Beneath the FSFORS, Depth of 5.67 m (18.06 ft)

\[ \tau_{BST} = 0.2272\sigma_{BST} + 6.832 \]

\[ \sigma_{BST} = 6.832 \text{ kPa} \]

\[ \phi_{BST} = 12.8^\circ \]

Consolidation Time = 0 minutes
Consolidation Time = 5 minutes
Consolidation Time = 15 minutes

1 kPa = 20.886 psf
FIG. 108. BST Results Beneath the Unit Cell, Soil Type was Sandy Elastic SILT (MH), Depth of 0.19 m (0.6 ft)

FIG. 109. BST Results Beneath the Unit Cell, Soil Type was SILT (ML), Depth of 0.80 m (2.6 ft)
FIG. 110. BST Results Beneath the Unit Cell, Soil Type was Fat CLAY (CH), Depth of 1.41 m (4.6 ft)
APPENDIX F

CPT RESULTS
FIG. 111. CPTU Results, Location was Centered Under FSFOUS, Elevation at 1288.283 m (4226.86 ft), GWT at 1.86 m (6.1 ft)
FIG. 112. CPTU Results, Location was Centered Under FSFORS, Elevation at 1288.118 m (4226.11 ft), GWT at 1.86 m (6.1 ft)
FIG. 113. CPTU Results, Location was Centered Under Unit Cell,
Elevation at 1288.353 m (4226.88 ft), GWT at 1.86 m (6.1 ft)
APPENDIX G

DT CORRECTIONS
String pot displacement transducers (DTs) were used to monitor movements of the footings and plates tested in this study (see Fig. 35). A string pot DT measures movement of an object by determining changes in the length of a wire (“string”) extending from the base of the DT to the object whose movements are being monitored. The primary loads and movements of the footings and plates were vertical. However, there were some unintended but inevitable horizontal movements that also occurred. In each test, the movements of the footing or plate were monitored using DTs in three nominal directions – vertical and two perpendicular horizontal directions. These horizontal movements caused changes in the length of the wires of the DTs measuring vertical movement. Thus, the measured movement obtained from readings of the vertical DTs ($\Delta v$) needed to be corrected to obtain the true vertical movements ($\Delta v_1$). This process required a trial and error procedure because the readings from the horizontal DTs were also affected by the vertical movement and the horizontal movement in the perpendicular direction and therefore the changes in readings were not equal to the actual movements.

A complete correction for the 3D movement is very complex and would have required tens of iterations for each set of three readings. These iterations were not easily done even with a computer program and required a lot of human manipulation. Since there were literally tens of thousands of values of settlement that needed to be determined for this study, the amount of time required for the 3D correction was infeasible. However, it was found that in most cases the horizontal movement in one direction was much less than that in the other direction. An analysis showed that 2D corrections using the larger of the horizontal movements provided answers that were within a few percent of the values obtained using the 3D correction. Therefore, 2D corrections were used to
find values of settlement in this study. Details of the process used are given below.

The first step was to determine if the footing movement could be reasonably approximated as \(2D\). During the test of each footing, the horizontal movement was monitored and the side that moved the least was focused on, which was in the east-west direction for all of the footings. The wires of the DTs that recorded these movements were very long, about 5.2 m (17.1 ft) and thus only small inaccuracies in the readings would be caused by the relatively small vertical and perpendicular horizontal movement. The measured movement from the horizontal DT was compared to the theoretical movement based on the following equation:

\[
\Delta_{hT} = \sqrt{\Delta_v^2 + \Delta_h^2 + L_{0, h}^2} - L_{0, h}
\]  

(55)

where \(\Delta_{hT}\) = theoretical horizontal movement

\(\Delta_v\) = measured vertical movement

\(\Delta_h\) = measured horizontal movement perpendicular to theoretical horizontal movement

\(L_{0, h}\) = initial length of wire used to measure theoretical horizontal movement

The unreinforced full-scale footing will be used as an example. The following values were determined at the largest vertical movement: \(\Delta_v = 288.5 \text{ mm (11.36 in.)}\), \(\Delta_h = 48.5 \text{ mm (1.91 in.)}\), and \(L_{0, h} = 5,168.9 \text{ mm (203.50 in.)}\). \(\Delta_{hT}\) was calculated to be 8.3 mm (0.33 in.) using Eq. 55 and the measured movement from the DT was 9.7 mm (0.38 in.), a difference of 1.4 mm (0.06 in), which was very small compared to \(L_{0, h}\). This value of \(\Delta_{hT}\) was small in comparison to \(\Delta_h\) and very small in comparison to \(\Delta_v\). Thus, it was determined that it was reasonable to approximate the movement as \(2D\).
There were four possible cases of footing movement that needed to be considered (Fig. 114). The first case was if the length of the vertical wire increased and the length of the horizontal wire decreased. The second was if the lengths of the vertical and horizontal wires both increased. The third was if the length of the vertical wire decreased and the length of the horizontal wire increased. The fourth was if the lengths of the vertical and horizontal wires both decreased.

An iterative process was needed to determine the correct value of vertical movement. This procedure was accomplished using a spreadsheet program. The first step of the process was to estimate the appropriate case. The second step was to obtain an initial estimate of $\theta_v$ by assuming that only horizontal movement occurred using the following equation:

$$
\theta_v = \tan^{-1} \left( \frac{\Delta_h}{L_{0,v}} \right)
$$

(56)

The third step was to use the initial value of $\theta_v$ determined from Eq. 56 to calculate and initial value of $\theta_h$. Using these initial values of $\theta_v$ and $\theta_h$, initial estimates of the correct vertical and horizontal movements ($\Delta_{v1}$ and $\Delta_{h1}$) were calculated. This completed the first iteration. The equations used to calculate $\theta_h$, $\Delta_{v1}$, and $\Delta_{h1}$ depended on the movement case as follows:

Case I

$$
\theta_h = \sin^{-1} \left[ \frac{(L_{0,v} + \Delta_v) \cos \theta_v - L_{0,v}}{L_{0,h} - \Delta_h} \right]
$$

(57)
FIG. 114. Four Cases of Possible Footing Movement During Testing:
(a) Case I; (b) Case II; (c) Case III; (d) Case IV
FIG. 114. Continued
\[ \theta_v = \sin^{-1}\left[ \frac{L_{0,h} - (L_{0,h} - \Delta_h \cos \theta_h)}{L_{0,v} + \Delta_v} \right] \]  
(58)

\[ \Delta_{h1} = L_{0,h} - (L_{0,h} - \Delta_h \cos \theta_h) \]  
(59)

\[ \Delta_{v1} = (L_{0,v} + \Delta_v \cos \theta_v - L_{0,v} \]  
(60)

Case II

\[ \theta_h = \sin^{-1}\left[ \frac{(L_{0,v} + \Delta_v \cos \theta_v - L_{0,v})}{L_{0,h} + \Delta_h} \right] \]  
(61)

\[ \theta_v = \sin^{-1}\left[ \frac{(L_{0,h} + \Delta_h \cos \theta_h - L_{0,h})}{L_{0,v} + \Delta_v} \right] \]  
(62)

\[ \Delta_{h1} = (L_{0,h} + \Delta_h \cos \theta_h - L_{0,h} \]  
(63)

\[ \Delta_{v1} = (L_{0,v} + \Delta_v \cos \theta_v - L_{0,v} \]  
(64)

Case III

\[ \theta_h = \sin^{-1}\left[ \frac{L_{0,v} - (L_{0,v} - \Delta_v \cos \theta_v)}{L_{0,h} + \Delta_h} \right] \]  
(65)

\[ \theta_v = \sin^{-1}\left[ \frac{(L_{0,h} + \Delta_h \cos \theta_h - L_{0,h})}{L_{0,v} - \Delta_v} \right] \]  
(66)

\[ \Delta_{h1} = (L_{0,h} + \Delta_h \cos \theta_h - L_{0,h} \]  
(67)

\[ \Delta_{v1} = L_{0,v} - (L_{0,v} - \Delta_v \cos \theta_v \]  
(68)
Case IV

\[
\theta_h = \sin^{-1}\left[ \frac{L_{0,v} - (L_{0,v} - \Delta_v) \cos \theta_v}{L_{0,h} - \Delta_h} \right]
\]  
(69)

\[
\theta_v = \sin^{-1}\left[ \frac{L_{0,h} - (L_{0,h} + \Delta_h) \cos \theta_h}{L_{0,v} - \Delta_v} \right]
\]  
(70)

\[
\Delta_{h1} = L_{0,h} - (L_{0,h} - \Delta_h) \cos \theta_h
\]  
(71)

\[
\Delta_{v1} = L_{0,v} - (L_{0,v} - \Delta_v) \cos \theta_v
\]  
(72)

At least two more iterations were needed to determine the correct value of \(\Delta_{v1}\). In these subsequent iterations, the values of \(\Delta_{v1}\) and \(\Delta_{h1}\) were used to calculate new values of \(\theta_v\) and \(\theta_h\), which in turn were used to calculate new values of \(\Delta_{v1}\) and \(\Delta_{h1}\). These new values of \(\Delta_{v1}\) and \(\Delta_{h1}\) were compared with the values from the previous iteration. If the differences between the new values and the previous values were not within a prescribed tolerance, another iteration was performed. Once final values were determined for the assumed case, it was determined whether or not this was the appropriate case. If not, the calculations were done again with the appropriate case.

The following example from the test on the full-scale footing on unreinforced soil will be used to illustrate the procedure. \(\Delta_v\) and \(\Delta_h\) were compared to estimate the appropriate movement case. \(\Delta_v\) was calculated as the reading from the computer of 868.8 mm (34.21 in.) minus the average initial reading before the test of 603.5 mm (23.76 in.), giving 265.3 mm (10.44 in.). \(\Delta_h\) was calculated as the reading from the computer of 693.4 mm (27.30 in.) minus the average initial reading before the test of 653.3 mm (25.7 in.), giving 40.1 mm (1.58 in.). Both \(\Delta_v\) and \(\Delta_h\) were positive, which tentatively
corresponds to Case II. The initial lengths of wire for vertical and horizontal measurements were $L_{0,v} = 736.6$ mm (29 in.) and $L_{0,h} = 758.8$ mm (29.88 in.). The iterative process is summarized in Table 32. It can be seen that for Case II the sign of the values for $\Delta h_1$ were negative, indicating that this was not the proper case. The data were then analyzed for Case I, which was the proper case as indicated by positive values for $\Delta h_1$ and $\Delta v_1$. The final result is that the initial value of vertical settlement determined from the readings of 265.3 mm (10.44 in.) was corrected to 261.3 mm (10.29 in.) and the initial value of horizontal movement was corrected from 40.1 mm (1.58 in.) to 14.6 mm (0.57 in.). These results are reasonable since the large vertical movement would produce larger errors in the much smaller horizontal movements than vice versa. This effect is shown in the photograph of Fig. 115.
### TABLE 32. Example Illustrating Corrections to Readings from Displacement Transducers for Horizontal and Vertical Movements

**Case II**

<table>
<thead>
<tr>
<th>Iteration</th>
<th>$\theta_h$ (radians)</th>
<th>$\theta_v$ (radians)</th>
<th>$\Delta h_1$ (mm)</th>
<th>$\Delta v_1$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.336544</td>
<td>0.054386</td>
<td>-4.71716</td>
<td>263.819</td>
</tr>
<tr>
<td>1</td>
<td>0.336544</td>
<td>-0.004708</td>
<td>-4.71716</td>
<td>265.289</td>
</tr>
<tr>
<td>2</td>
<td>0.338495</td>
<td>-0.005223</td>
<td>-5.23314</td>
<td>265.286</td>
</tr>
<tr>
<td>3</td>
<td>0.338491</td>
<td>-0.005222</td>
<td>-5.23224</td>
<td>265.286</td>
</tr>
<tr>
<td>4</td>
<td>0.338491</td>
<td>-0.005222</td>
<td>-5.23224</td>
<td>265.286</td>
</tr>
</tbody>
</table>

**Case I**

<table>
<thead>
<tr>
<th>Iteration</th>
<th>$\theta_h$ (radians)</th>
<th>$\theta_v$ (radians)</th>
<th>$\Delta h_1$ (mm)</th>
<th>$\Delta v_1$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.37587</td>
<td>0.054386</td>
<td>15.6709</td>
<td>263.819</td>
</tr>
<tr>
<td>1</td>
<td>0.37587</td>
<td>0.090223</td>
<td>15.6709</td>
<td>261.225</td>
</tr>
<tr>
<td>2</td>
<td>0.37199</td>
<td>0.089203</td>
<td>14.5396</td>
<td>261.316</td>
</tr>
<tr>
<td>3</td>
<td>0.37213</td>
<td>0.089239</td>
<td>14.5793</td>
<td>261.313</td>
</tr>
<tr>
<td>4</td>
<td>0.37212</td>
<td>0.089238</td>
<td>14.5779</td>
<td>261.313</td>
</tr>
<tr>
<td>5</td>
<td>0.37212</td>
<td>0.089238</td>
<td>14.5780</td>
<td>261.313</td>
</tr>
<tr>
<td>6</td>
<td>0.37212</td>
<td>0.089238</td>
<td>14.5780</td>
<td>261.313</td>
</tr>
</tbody>
</table>
FIG. 115. Photograph Showing Effect of Large Vertical Movement on Extension of Wire Nominally Measuring Horizontal Movement at Same Displacement as the Example
REFERENCES


