SIMPLIFIED PROCEDURE FOR ANALYSIS
OF LATERALLY LOADED SINGLE PILES AND PILE GROUPS

A THESIS SUBMITTED TO THE GRADUATE DIVISON OF THE
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ACKNOWLEDGEMENTS

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I would also like to acknowledge Drs. Horst Brandes and Peter Nicholson for reviewing this thesis.

Finally, I would like to thank my wife, Patricia, and son, Timothy, for giving me their support, encouragement and patience throughout my study.
ABSTRACT

Current methods for designing pile groups for lateral loading require a computer program or extensive manual computations. This research presents a spreadsheet- or calculator-amenable approach for estimating lateral deflections and maximum moments in single piles and pile groups. The approach for single piles is an extension of the characteristic load method, used for predicting deflections and moments when the pile top is at the ground surface. The proposed method applies to embedded fixed-head piles, which represents most practical situations. The resulting lateral deflections and moments are less due to the increased embedment. A simplified procedure to estimate group deflections and moments was also developed. Termed the Group Amplification Method (GAM), group amplification factors are introduced to amplify the single pile deflection and bending moment to reflect group effects. This approach provides good agreement with other generally accepted analytical tools and with values measured in load tests on groups of fixed-head piles.
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CHAPTER 1
INTRODUCTION

Most structures are subject to lateral loads as a result of wind, earthquake, impact, waves and lateral earth pressure. If these structures are supported on deep foundations, the foundations have to be designed for lateral loads. Laterally loaded single piles and pile groups should be designed to be safe against geotechnical failure, structural failure and excessive deflections. In general, geotechnical failure seldom governs the design for laterally loaded piles that are long with respect to its diameter or width. Therefore, this study will focus on the analyses of laterally loaded piles for deflections and structural failure.

This report describes the research on developing a simplified method to analyze fixed-head single piles and pile groups subjected to lateral loads. In Chapter 2, a literature review of current state-of-the-art methods for lateral load analyses of single piles and pile groups is presented. A description of the simplified procedures developed in this study to analyze a single and a group of fixed head piles is presented in Chapter 3. In Chapter 4, three full-scale lateral load tests on groups of fixed-head piles are evaluated where the predicted responses are compared with measured results.
CHAPTER 2
LITERATURE REVIEW

Articles providing the current state-of-the-art for analyzing laterally loaded piles and pile groups are summarized based on a literature review. Throughout this section, the term "pile" includes both driven piles and drilled shafts.

2.1 Analysis of Single Piles Under Lateral Load

2.1.1 Evans and Duncan's Procedure

Evans and Duncan (1982) developed a simplified procedure to predict non-linear behavior of laterally loaded single piles under static loading conditions. In this approach, dimensionless load-deflection and load-moment curves were developed for piles in cohesive and cohesionless soils. Separate plots were available for free- and fixed-head piles. However, only fixed head piles are considered since more often than not, piles are used in groups that are connected via a cap. These dimensionless plots are shown in Figures 1 through 4, where $P$ is the applied lateral load, $M$ is the maximum moment induced in the pile due to the lateral load, $y$ is the groundline lateral deflection of the pile and $D$ is the pile width or diameter. Values of load, moment, and deflection are made dimensionless by normalizing with a characteristic load, $P_c$, a characteristic
Figure 1. Dimensionless load-deflection curves for fixed head piles in sand with zero embedment.

Figure 2. Dimensionless load-moment curves for fixed head piles in sand with zero embedment.
Figure 3. Dimensionless load-deflection curves for fixed head piles in clay with zero embedment.

Figure 4. Dimensionless load-moment curves for fixed head piles in clay with zero embedment.
moment, \( M_c \), and the pile width or diameter, \( D \), respectively. \( P_c \) and \( M_c \) are expressed as follows:

\[
P_c = FD^2 \left( \frac{E R_1}{E R_t} \right) \left( \frac{\sigma_p}{\sigma_{50}} \right)^m (\varepsilon_{50})^n \tag{2.1}
\]

\[
M_c = FD^3 \left( \frac{E R_1}{E R_t} \right) \left( \frac{\sigma_p}{\sigma_{50}} \right)^m (\varepsilon_{50})^n \tag{2.2}
\]

where

\( D = \) pile width or diameter (L)

\( E = \) Young’s modulus of the pile (FL^-2)

\( R_t = \) moment of inertia ratio (dimensionless)

\[
R_t = \frac{I}{I_s} \tag{2.3}
\]

\( I = \) moment of inertia of the pile (L^4)

\( I_s = \) moment of inertia of a solid circular cross section of diameter \( D \) (L^4)

\[
I_s = \frac{\pi D^4}{64} \tag{2.4}
\]

The remaining parameters for the equations are summarized in Table 1. The behavior of the pile under lateral load is governed by the soil within the top eight pile diameters. The moist and buoyant unit weights of the soil should be used above and below the water table, respectively. The Evans and Duncan (1982) procedure only applies to piles where the top is at the ground surface.
Table 1. Parameters for Equations 2.1 and 2.2 (after Evans and Duncan, 1982)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Sands</th>
<th>Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F ) = dimensionless parameter related to stress-strain behavior of the soil</td>
<td>( P_c )</td>
<td>( M_c )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( m ) = passive pressure exponent</td>
<td>0.57</td>
<td>0.40</td>
</tr>
<tr>
<td>( n ) = strain exponent</td>
<td>-0.22</td>
<td>-0.15</td>
</tr>
<tr>
<td>( \sigma_p ) = representative passive pressure of soil ((FL^{-2}))</td>
<td>( \sigma_p = 2C_{pΒ}D\tan^2(45^\circ + \phi/2) )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( C_{pΒ} = ) dimensionless modifying factor to account for three-dimensional effect of the passive wedge in front of the pile = ( \phi/10 )</td>
<td></td>
</tr>
<tr>
<td>( \gamma ) = unit weight of soil</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \varepsilon_{50} ) = strain at which 50% of the strength of the soil is mobilized</td>
<td>( \varepsilon_{50} = 0.002 ) for sands</td>
<td>( \varepsilon_{50} ) ranges from 0.004 to 0.020 for plastic clays</td>
</tr>
</tbody>
</table>
2.1.2 Characteristic Load Method (CLM)

The characteristic load method (Duncan et al., 1994) is a modification of the Evans and Duncan procedure, where the equations for $P_c$ and $M_c$ are simplified as shown in Equations (2.5) through (2.8):

For sand

$P_c = 1.57D^2\left(\frac{\gamma'D\phi'tan^2\left(45^\circ + \phi/2\right)}{ER_i}\right)^{0.57}$ \hspace{1cm} (2.5)

$M_c = 1.33D^3\left(\frac{\gamma'D\phi'tan^2\left(45^\circ + \phi/2\right)}{ER_i}\right)^{0.40}$ \hspace{1cm} (2.6)

For clay

$P_c = 7.34D^2\left(\frac{S_u}{ER_i}\right)^{0.86}$ \hspace{1cm} (2.7)

$M_c = 3.86D^3\left(\frac{S_u}{ER_i}\right)^{0.46}$ \hspace{1cm} (2.8)

The parameters have been defined in Section 2.1.1.

2.1.3 Brettmann and Duncan's Equations

Brettmann and Duncan (1996) developed exponential equations for the dimensionless nonlinear relationships between load-displacement and load-moment as shown in Figures 1 through 4. The equations are as follows:
\( \frac{y}{D} = a \left( \frac{P}{P_c} \right)^b \) \hspace{2cm} (2.9)

\( \frac{M}{M_c} = c \left( \frac{P}{P_c} \right)^d \) \hspace{2cm} (2.10)

where \( a, b, c \) and \( d \) are summarized in Table 2.

<table>
<thead>
<tr>
<th>Constant</th>
<th>Fixed-Head Piles in Sand</th>
<th>Fixed-Head Piles in Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a )</td>
<td>28.8</td>
<td>14.0</td>
</tr>
<tr>
<td>( b )</td>
<td>1.5</td>
<td>1.846</td>
</tr>
<tr>
<td>( c )</td>
<td>2.64</td>
<td>0.78</td>
</tr>
<tr>
<td>( d )</td>
<td>1.3</td>
<td>1.249</td>
</tr>
</tbody>
</table>

2.1.4 LPILE Plus 3.0 for Windows

LPILE Plus 3.0 for Windows (Reese and Wang, 1997) is commercial software that can be used to analyze the behavior of a single pile under lateral load. The program uses a finite difference technique to estimate deflection, shear, bending moment, and soil reaction varying with depth. Soil behavior is modeled using a series of discrete non-linear springs. The stiffnesses of these springs are characterized using p-y curves, which can be user-defined or internally generated by the computer program following published recommendations for various types of soils. These p-y curves were developed based on results of full-scale lateral load tests on piles in a variety of soils and
loading conditions. A wide variation of pile-head boundary conditions may be selected in the program. The properties of the pile can also vary as a function of depth. The program also has the capability of considering the non-linear behavior of concrete piles as a result of cracking.

2.1.5 Modified Characteristic Load Method

Wang (2000) extended the Evans and Duncan (1982) procedure to estimate deflection and maximum bending moment of fixed head piles embedded below ground surface. Wang's work applies to piles embedded in cohesionless soils only. Termed the Modified Characteristic Load Method (MCLM), modification factors $C_y$ and $C_m$ that account for the effect of pile embedment are introduced as shown in Equations (2.11) through (2.14).

\[ P'_c = P_c C_y \]  
\[ M'_c = M_c C_m \]  
\[ C_y = \left[ 1 + \frac{Z}{10D} \left( \frac{\gamma'}{\gamma} \right)^{0.1} \right]^{0.065 - 1.05} \]  
\[ C_m = \left[ 1 + \frac{Z}{2D} \left( \frac{\gamma'}{\gamma} \right)^{0.1} \right]^{0.034 - 0.65} \]  

(2.11)  
(2.12)  
(2.13)  
(2.14)
Wang also proposed modifying Brettmann and Duncan's equations as follows:

\[
\left(\frac{Y}{D}\right) = 0.515\left(\frac{P}{P_c'}\right) + 142\left(\frac{P}{P_c'}\right)^2
\]  
(2.15)

\[
\left(\frac{M}{M_c'}\right) = 2.4\left(\frac{P}{P_c'}\right)^{1.3}
\]  
(2.16)

where \(P_c'\) = modified characteristic load

\(M_c'\) = modified characteristic moment

\(Z\) = embedment depth (L)

\(\gamma\) = total unit weight of soil (FL⁻³)

\(\gamma'\) = effective unit weight of soil (FL⁻³)

= buoyant unit weight for submerged soil

= moist unit weight if the ground water table is not within the top 8 pile diameters.

As part of this study, Wang’s procedure has been expanded to include laterally loaded single piles in cohesive soils.
2.2 Analysis of Pile Groups Under Lateral Load

Pile groups can be divided into two categories:

1. widely spaced piles which interact only through the pile cap connection. In these piles, the group response can be estimated by summing the individual pile responses.

2. closely spaced piles are defined as those in which the response of an individual pile is influenced through the adjacent soil by the response of other nearby piles. This “shadowing” effect is commonly termed pile-soil-pile interaction.

This study deals primarily with closely spaced piles, which involves the majority of pile groups in practice.

2.2.1 The Concept of p-multipliers

For a group of closely-spaced piles, pile-soil-pile interaction can be taken into account by introducing reduction factors to the soil reaction (p) portion of the p-y curves for single piles as shown in Figure 5 (Brown et al., 1988). These p-multipliers are typically less than or equal to one. They have been experimentally derived either from full-scale load tests or from tests in a
centrifuge. Reducing the p-value on the p-y curve results in a reduction of the ultimate soil resistance leading to a "softening" of the group response. According to Brown et al. (1988), p-multipliers are approximately constant with depth. The magnitude of the p-multipliers varies with pile row position, pile spacing and soil type. Piles in the leading row have the highest p-multipliers. Brown et al. (2001) also indicated construction method (bored versus driven piles) has an influence on p-multipliers, the case in point being the Chaiyi Taiwan load test. A summary of p-multipliers obtained from the literature is provided in Table 3. Mokwa et al (2001) plotted p-multipliers as a function of pile spacing and row locations in four graphs (Figures 6 and 7).

Figure 5. p-multiplier concept (Brown et al., 1988)
<table>
<thead>
<tr>
<th>Location</th>
<th>Test Type</th>
<th>Size of Pile Group</th>
<th>Pile Type</th>
<th>Center-to-Center Spacing</th>
<th>Pile Fixity at Top</th>
<th>Soil Type</th>
<th>Shear Strength Parameter</th>
<th>Collection (Inches)</th>
<th>1st Row</th>
<th>2nd Row</th>
<th>3rd Row</th>
<th>4th Row</th>
<th>5th Row</th>
<th>6th Row</th>
<th>7th Row</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brittany, France</td>
<td>Full-scale 3 X 2</td>
<td>Steel H-Pile (d=11.2&quot;, b=f=10.6&quot;) with Side Plates Welded to Form a Box Section</td>
<td>3D Free-head Clay</td>
<td>Su = 420 psf</td>
<td>0.6 0.9 0.6 - - - - - -</td>
<td>Mc sensor, 1988</td>
<td></td>
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<tr>
<td>Houston, TX</td>
<td>Full-scale 3 X 3</td>
<td>10.75&quot; OD Steel Pipe Pile with Wall Thickness = 0.365&quot; &amp; Grout Fill</td>
<td>3D Free-head Clay</td>
<td>Su = 1500 psf</td>
<td>1.2 0.7 0.6 0.5 0.4 - - - -</td>
<td>Brown et al., 1987</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Salt Lake City, UT</td>
<td>Full-scale 3 X 3</td>
<td>12&quot; OD Steel Pipe Pile with Concrete Fill</td>
<td>3D Free-head Clay</td>
<td>Su = 1000 psf</td>
<td>1.0 to 2.4 0.6 0.4 0.4 - - - -</td>
<td>Rollins et al., 1998</td>
<td></td>
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<tr>
<td>Salt Lake City, UT</td>
<td>Full-scale 3 X 3</td>
<td>12&quot; OD Steel Pipe Pile with Concrete Fill</td>
<td>3D Fixed-head Clay</td>
<td>Similar to Free-Head According to Rollins et al., 1998</td>
<td>2001</td>
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<tr>
<td>Houston, TX</td>
<td>Full-scale 3 X 3</td>
<td>10.75&quot; OD Steel Pipe Pile with Wall Thickness = 0.365&quot; &amp; Grout Fill</td>
<td>3D Free-head Sand</td>
<td>Ø = 38° Dr &gt; 90%</td>
<td>1.0 to 1.5 0.8 0.4 0.3 - - - -</td>
<td>Brown et al., 1988</td>
<td></td>
<td></td>
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<tr>
<td>Chaiyi, Taiwan</td>
<td>Full-scale 2 X 3</td>
<td>15-m Drilled Shaft</td>
<td>3D Fixed-head Sand</td>
<td>Ø = 35°</td>
<td>1.2 0.5 0.4 0.3 - - - -</td>
<td>Brown et al., 2001</td>
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<tr>
<td></td>
<td>Full-scale 3 X 4</td>
<td>9.8-m OD Prestressed Concrete Pile</td>
<td>3D Fixed-head Sand</td>
<td>Ø = 35°</td>
<td>4.9 0.9 0.7 0.5 0.4 - - - -</td>
<td>Brown et al., 2001</td>
<td></td>
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<tr>
<td>Stuart, FL</td>
<td>Full-scale 4 X 4</td>
<td>30&quot;X30&quot; Square Prestressed Concrete Piles</td>
<td>3D Free-head Sand</td>
<td>Ø = 32°</td>
<td>1.0 to 3.0 0.8 0.7 0.3 0.3 - - - -</td>
<td>Sparks &amp; Rollins, 1997</td>
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<tr>
<td>- Centrifuge</td>
<td>3 X 3</td>
<td>16.9&quot; OD Pipe Pile (42.7 ft long)</td>
<td>3D Free-head Sand</td>
<td>Dr = 33%</td>
<td>- 0.65 0.45 0.35 - - - - -</td>
<td>McVay et al., 1998</td>
<td></td>
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<tr>
<td>- Centrifuge</td>
<td>3 X 3</td>
<td>16.9&quot; OD Pipe Pile (42.7 ft long)</td>
<td>5D Free-head Sand</td>
<td>Dr = 33%</td>
<td>- 1 0.65 0.7 - - - - -</td>
<td>McVay et al., 1998</td>
<td></td>
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</tr>
<tr>
<td>- Centrifuge</td>
<td>3 X 3</td>
<td>16.9&quot; OD Pipe Pile (42.7 ft long)</td>
<td>3D Free-head Sand</td>
<td>Dr = 65%</td>
<td>- 0.8 0.4 0.3 - - - - -</td>
<td>McVay et al., 1998</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Centrifuge</td>
<td>3 X 3</td>
<td>16.9&quot; OD Pipe Pile (42.7 ft long)</td>
<td>5D Free-head Sand</td>
<td>Dr = 65%</td>
<td>- 1 0.85 0.7 - - - - -</td>
<td>McVay et al., 1998</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Centrifuge</td>
<td>3 X 3</td>
<td>16.9&quot; OD Pipe Pile (45 ft long)</td>
<td>3D Free-head Sand</td>
<td>Dr = 36 &amp; 55%</td>
<td>- 0.8 0.4 0.3 0.3 - - - - -</td>
<td>McVay et al., 1998</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Centrifuge</td>
<td>3 X 4</td>
<td>16.9&quot; OD Pipe Pile (45 ft long)</td>
<td>3D Free-head Sand</td>
<td>Dr = 36 &amp; 55%</td>
<td>- 0.8 0.4 0.3 0.3 - - - - -</td>
<td>McVay et al., 1998</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Centrifuge</td>
<td>3 X 5</td>
<td>16.9&quot; OD Pipe Pile (45 ft long)</td>
<td>3D Free-head Sand</td>
<td>Dr = 36 &amp; 55%</td>
<td>- 0.8 0.4 0.3 0.3 - - - - -</td>
<td>McVay et al., 1998</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Centrifuge</td>
<td>3 X 6</td>
<td>16.9&quot; OD Pipe Pile (45 ft long)</td>
<td>3D Free-head Sand</td>
<td>Dr = 36 &amp; 55%</td>
<td>- 0.8 0.4 0.3 0.3 0.3 - - - -</td>
<td>McVay et al., 1998</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Centrifuge</td>
<td>3 X 7</td>
<td>16.9&quot; OD Pipe Pile (45 ft long)</td>
<td>3D Free-head Sand</td>
<td>Dr = 36 &amp; 55%</td>
<td>- 0.8 0.4 0.3 0.3 0.3 0.3 - - -</td>
<td>McVay et al., 1998</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*The first number refers to the number of plies in each row. The second number refers to the number of rows of piles in the group.*
Figure 6. p-multiplier as a function of pile spacing for leading and first trailing row

(after Mokwa and Duncan, 2001)
Figure 7. p-multiplier as a function of pile spacing for the second and third trailing rows (after Mokwa and Duncan, 2001)
2.2.2 Brown and Bollman’s Method

Brown and Bollman (1993) developed a procedure to analyze the behavior of pile groups under lateral load by performing separate p-y analysis for each row using appropriate values of p-multipliers for each row. The series of p-y analyses is set up as follows for symmetric pile groups:

1. Determine the p-multiplier for each row according to row position, pile spacing, and soil type.

2. Perform p-y analyses for one pile in each row using the appropriate p-multiplier for that row.

3. Plot the lateral load versus deflection for the pile analyzed in each row (see Figure 8). The lateral load per pile-deflection curve of the pile group can be estimated by summing the lateral loads for the pile in each row at the same lateral deflection and dividing this sum by the number of rows.

4. The maximum bending moment in the most heavily loaded pile in the group corresponds to the bending moment in the piles in the leading row.
Figure 8. Brown and Bollman's (1993) method (after Hannigan et al., 1997)
2.2.3 Mokwa and Duncan Group Equivalent Pile Procedure

Mokwa and Duncan (2000) proposed analyzing a group as a single pile in LPILE. The group equivalent pile is assigned the same structural properties as the individual pile but the moment of inertia is set equal to the sum of the moment of inertia of all the piles. The p-y curve is modified by using the sum of the p-multipliers for all the piles in the group. Then LPILE Plus is used to predict the lateral group deflection. The bending moment of each pile is determined using equation (2.17)

\[ M_i = M_{\text{gep}} \left[ \frac{p_{\text{mi}} E_i l_i}{\sum_{i=1}^{N} (p_{\text{mi}} E_i l_i)} \right] (p_{\text{mc}}) \]  

(2.17)

where

- \( M_i \) = bending moment in pile \( i \)
- \( M_{\text{gep}} \) = bending moment computed for the group equivalent pile
- \( N \) = number of piles in the group
- \( p_{\text{mi}} \) = p-multiplier for pile \( i \)
- \( l_i \) = moment of inertia of pile \( i \)
- \( p_{\text{mc}} \) = a multiplier for corner piles = 1.0, 1.2 and 1.6 for spacings \( \geq 3D \), = 2D and = 1D, respectively. (Franke, 1988)
2.2.4 GROUP 4.0 for Windows and FBPier

Two state-of-the-art computer programs with the capability for analyzing pile groups are reviewed in this section. GROUP for Windows (Reese and Wang, 1996) is a commercial program for analyzing groups of vertical or battered piles. Moment, lateral loads or a combination of the two may be applied to the group. Possible options of connectivity to the pile cap include fixed, pinned, or elastically restrained. The program solves by iteration for the nonlinear response of each pile under combined loading, and checks for compatibility of geometry and equilibrium of forces between the applied external loads and the reactions of each pile head. The load-deflection and load-moment relationships for each pile in its individual coordinate system are computed by solving nonlinear differential equations using p-y and t-z curves under lateral and axial loading, respectively. For closely-spaced piles, the pile-soil-pile interaction can be taken into account by introducing reduction factors for the p-y curves used for each single pile. The program can compute the deflection, bending moment, shear, and soil resistance as a function of depth for each pile.

FBPier (Bridge Software Institute, 2003) is a more sophisticated non-linear, finite element analysis, soil-structure interaction program developed at the University of Florida. The program can be used to perform a complete substructure design considering both the geotechnical and structural aspects. The program can be used to analyze single piles, pile groups, pile bents,
retaining walls, and high mast lighting structures. Analysis capabilities include combined axial, lateral, and rotation resistance of the piles, pile cap and pier. The structural model includes both linear and non-linear (concrete cracking, steel yielding) capabilities, as well as biaxial interaction diagrams for all sections. It also allows the use of different pile types within a group.

2.3 Limitation of the Procedures Described

The Evans and Duncan procedure and the CLM cannot be used to predict the lateral behavior of single piles embedded below the ground surface. The MCLM can account for pile head embedment but to date, it has only been developed for single piles in cohesionless soils. In this study, the MCLM is developed for single piles in cohesive soils. A new simple technique is also developed to analyze the behavior of pile groups based on the concept of p-multipliers and using the GEP method. This procedure is simple enough that it can be readily performed in a spreadsheet or using a calculator.
CHAPTER 3
DEVELOPMENT OF SIMPLIFIED PROCEDURE FOR ANALYSIS OF
LATERALLY LOADED SINGLE PILES AND PILE GROUPS

3.1 Parametric Study for Single Piles in Cohesive Soils

LPILE Plus 3.0 for Windows was used to perform a parametric study on the response of laterally loaded fixed-head piles in saturated cohesive soils with different embedment depths. The study included three hundred analyses generating about 1,700 load cases. The main parameters considered include pile type, pile size, undrained shear strength, and embedment depth. A range of lateral loads was applied to the top of the piles to obtain load-deflection and load-moment curves.

Analyses were conducted on both driven piles and drilled shafts. Driven piles analyzed included steel pipe piles, steel H-piles, and precast prestressed concrete piles. Detail properties of the piles used in the parametric study are shown in Table 4. Pile diameters or widths analyzed were between 9.7 inches and 48 inches. The flexural stiffness of the piles ranged from $3.31 \times 10^6$ to $9.38 \times 10^8$ kip-in$^2$. When the pile extreme fiber stresses exceeded the allowable values, the pile failed structurally. These load cases are not included in the parametric study.
Table 4. Summary of pile types and properties used in this study

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Pile Description</th>
<th>Pile Width or Diameter (inches)</th>
<th>Area (in²)</th>
<th>E&lt;sup&gt;a&lt;/sup&gt; (ksi)</th>
<th>I&lt;sup&gt;b&lt;/sup&gt; (in⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel H-pile</td>
<td>HP10x42</td>
<td>9.7</td>
<td>12.40</td>
<td>29000</td>
<td>210</td>
</tr>
<tr>
<td>Steel H-pile</td>
<td>HP12x74</td>
<td>12.13</td>
<td>21.80</td>
<td>29000</td>
<td>569</td>
</tr>
<tr>
<td>Steel H-pile</td>
<td>HP14x102</td>
<td>14.01</td>
<td>30.00</td>
<td>29000</td>
<td>1050</td>
</tr>
<tr>
<td>Steel Pipe Pile</td>
<td>10½-inch-diameter 7/8-inch-thick wall</td>
<td>10.75</td>
<td>8.25</td>
<td>29000</td>
<td>114</td>
</tr>
<tr>
<td>Steel Pipe Pile</td>
<td>12½-inch-diameter 3/8-inch-thick wall</td>
<td>12.75</td>
<td>14.60</td>
<td>29000</td>
<td>279</td>
</tr>
<tr>
<td>Steel Pipe Pile</td>
<td>14-inch-diameter 3/8-inch-thick wall</td>
<td>14</td>
<td>16.10</td>
<td>29000</td>
<td>373</td>
</tr>
<tr>
<td>Precast Prestressed</td>
<td>14-inch-square f&lt;sub&gt;c&lt;/sub&gt;;&lt;sup&gt;a&lt;/sup&gt;=5000psi</td>
<td>14</td>
<td>196.00</td>
<td>4300</td>
<td>3201</td>
</tr>
<tr>
<td>Concrete Pile</td>
<td>14-inch-square f&lt;sub&gt;c&lt;/sub&gt;;&lt;sup&gt;a&lt;/sup&gt;=5000psi</td>
<td>16</td>
<td>256.00</td>
<td>4300</td>
<td>5461</td>
</tr>
<tr>
<td>Drilled Shaft</td>
<td>18-inch-diameter f&lt;sub&gt;c&lt;/sub&gt;;&lt;sup&gt;c&lt;/sup&gt;=4000psi</td>
<td>18</td>
<td>254.47</td>
<td>3600</td>
<td>5153</td>
</tr>
<tr>
<td>Drilled Shaft</td>
<td>24-inch-diameter f&lt;sub&gt;c&lt;/sub&gt;;&lt;sup&gt;c&lt;/sup&gt;=4000psi</td>
<td>24</td>
<td>452.39</td>
<td>3600</td>
<td>16286</td>
</tr>
<tr>
<td>Drilled Shaft</td>
<td>36-inch-diameter f&lt;sub&gt;c&lt;/sub&gt;;&lt;sup&gt;c&lt;/sup&gt;=4000psi</td>
<td>36</td>
<td>1017.88</td>
<td>3600</td>
<td>82448</td>
</tr>
<tr>
<td>Drilled Shaft</td>
<td>48-inch-diameter f&lt;sub&gt;c&lt;/sub&gt;;&lt;sup&gt;c&lt;/sup&gt;=4000psi</td>
<td>48</td>
<td>1809.56</td>
<td>3600</td>
<td>260576</td>
</tr>
</tbody>
</table>

<sup>a</sup> E = Young’s modulus of pile

<sup>b</sup> I = moment of inertia of pile

<sup>c</sup> f<sub>c</sub>' = 28-day compressive strength of concrete
Four values of undrained shear strength (0.5, 1, 2 and 4 ksf) were assumed. Matlock's (1970) soft clay model was used for cohesive soils with undrained shear strengths of 0.5 and 1 ksf while Reese and Welch's (1975) stiff cohesive soil model was used for undrained shear strengths of 1, 2 and 4 ksf. p-y parameters for the soils are summarized in Table 5. Both models were used to analyze cohesive soils with an undrained shear strength of 1 ksf to observe any differences between the two models. A comparison of the two models showed no major differences when analyzing cohesive soils with an undrained shear strength of 1 ksf.

Table 5. p-y parameters for piles in cohesive soils

<table>
<thead>
<tr>
<th>Soil model</th>
<th>Undrained shear strength (ksf)</th>
<th>Soil modulus (pci)</th>
<th>ε50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft clay</td>
<td>0.5</td>
<td>0</td>
<td>0.02</td>
</tr>
<tr>
<td>Soft clay</td>
<td>1</td>
<td>500</td>
<td>0.01</td>
</tr>
<tr>
<td>Stiff clay</td>
<td>1</td>
<td>500</td>
<td>0.007</td>
</tr>
<tr>
<td>Stiff clay</td>
<td>2</td>
<td>1000</td>
<td>0.005</td>
</tr>
<tr>
<td>Stiff clay</td>
<td>4</td>
<td>2000</td>
<td>0.004</td>
</tr>
</tbody>
</table>

The pile top was embedded at several depths below ground surface. Embedment depths considered include 0, 2, 4, 6 and 10 feet.
3.1.1 Modified Characteristic Load for Single Piles in Cohesive Soils

Initially, LPILE analyses were performed for piles with zero embedment to confirm that the analyses yielded results that are consistent with Brettmann and Duncan’s (1994) equation. The results plotted in Figure 9 show good agreement. The analyses were then extended to include piles with non-zero embedment and the results are presented in Figure 10. Evans and Duncan’s characteristic load for the case of zero-embedment was used to normalize the lateral load. The figure shows wide scatter, and the Evans and Duncan procedure tends to overestimate deflection. Thus, as embedment depth increases, the deflection decreases under the same lateral load indicating that the increase in lateral earth pressure significantly influences the lateral behavior of piles.

To make the CLM applicable to the cases with non-zero embedment depth, the equation for the characteristic load must be modified so that the calculated load-deflection points all fall on the Evans and Duncan trendline. An embedment correction factor $C_y$ was developed for the characteristic load ($P_c$) to obtain a better match between the LPILE results and the Evans and Duncan procedure. The correction factor is applied to the characteristic load as follows:
Figure 9. Dimensionless load-deflection LPILE Plus data points for piles in cohesive soils with zero embedment.

Figure 10. Dimensionless load-deflection LPILE Plus data points for piles embedded 2, 4, 6 and 10 ft in cohesive soils.
\[ P'_c = P_c C_y \]  \hspace{2cm} (3.1)

\[ C_y = \left(1 + \frac{Z}{D}\right)^{-\left(\frac{S_u}{1000 \gamma D^{0.16}}\right)} \]  \hspace{2cm} (3.2)

where

\[ P'_c = \text{modified characteristic load (F)} \]
\[ D = \text{pile diameter or width (L)} \]
\[ Z = \text{embedment depth of pile top below ground surface (L)} \]
\[ \gamma = \text{effective unit weight of soil over top 8D (FL}^{-3}\text{)} \]
\[ \gamma = \text{buoyant unit weight for submerged soil} \]
\[ \gamma = \text{moist unit weight for soils above the ground water table} \]
\[ S_u = \text{undrained shear strength (FL}^{-2}\text{)} \]

\[ C_y \] equals one when the embedment depth is zero, is greater than 1.0 for positive embedment and is not applicable for piles extended above ground.

When the modified characteristic load is used to normalize the lateral load, the revised dimensionless load-deflection points all fall within a narrow range close to the Brettmann and Duncan curve as shown in Figure 11.

26
Figure 11. Modified dimensionless load-deflection LPILE Plus data points for piles embedded 2, 4, 6 and 10 ft in cohesive soils

Figure 12. Dimensionless load-moment LPILE Plus data points for piles in cohesive soils with zero embedment
3.1.2 Modified Characteristic Moment for Single Piles in Cohesive Soils

Using the LPILE results, dimensionless load-moment curves for zero embedment are plotted in Figure 12. The lateral load is normalized by the characteristic load, \( P_c \), while the maximum bending moment is normalized by the characteristic moment, \( M_c \). The LPILE data points all fall within a narrow range illustrating the reliability of the Evans and Duncan procedure. Again, the dimensionless load-moment points for non-zero embedment show poor agreement as seen in Figure 13, where a large scatter can be observed.

A correction factor, \( C_m \), was developed to modify the characteristic moment, \( M_c \), for non-zero embedment depth as follows:

\[
M'_c = C_m M_c \tag{3.3}
\]

where \( M'_c \) is the modified characteristic moment.

\[
C_m = \left(1 + \frac{Z}{60D}\right)^{\left(\frac{8}{10D}+0.2\right)} \tag{3.4}
\]
Figure 13. Dimensionless load-moment LPILE Plus data points for piles embedded 2, 4, 6 and 10 ft in cohesive soils.

Figure 14. Modified dimensionless load-moment LPILE Plus data points for piles embedded 2, 4, 6 and 10 ft in cohesive soils.
When the modified characteristic load, $P_c'$ and modified characteristic moment, $M_c'$, are used to normalize the load-moment points for the case of non-zero embedment, the scatter diminishes as shown in Figure 14.

3.1.3 Steps for the Modified Characteristic Load Method

Steps for analyzing the behavior of laterally loaded piles in cohesive soils using the MCLM are:

1) Calculate the modified characteristic load using Equations (2.1), (3.1), and (3.2).

2) For a given lateral load, $P$, calculate the dimensionless lateral load $P/P_c'$.

3) Use Brettmann and Duncan’s Equation (2.9) to estimate the dimensionless deflection $Y/D$ corresponding to the dimensionless load from Step 2.

4) Calculate the deflection by multiplying the estimated dimensionless deflection from Step 3 with the pile width or diameter.

5) Calculate the modified characteristic moment using Equations (2.2), (3.3), and (3.4).

6) Use Brettmann and Duncan’s Equation (2.10) to estimate the dimensionless moment $M/M_c'$ corresponding to the value of $P/P_c'$ from Step 2.

7) Multiply the dimensionless moment from Step 6 with the modified characteristic moment from the Step 5 to determine the maximum bending
moment M. Since the pile head is fixed, the maximum bending moment occurs at the top of the pile.

This method has the following limitations.

(1) The MCLM applies only to vertical piles.
(2) This study was performed assuming long piles only which are very common in practice. This method is not applicable to short piles with low L/D ratios.
(3) Cases where the pile failed structurally are not included in the parametric study.
(4) This method cannot automatically account for nonlinear behavior of reinforced concrete piles after cracking. However, the engineer may manually reduce the flexural rigidity of the pile to account for this non-linear behavior.
(5) The maximum embedment depth involved in this study was 10 feet.
(6) Only static lateral loads were considered in this study.
(7) The method assumes uniform soil properties within the top 8 pile diameters. For layered systems, the average properties within the top eight pile diameters should be used.
3.2 Parametric Study for Pile Groups in Cohesive and Cohesionless Soils

3.2.1 Group Amplification Method

Groups of closely-spaced piles deflect more and are subject to higher moments compared to single piles for the same load per pile. This is due to pile-soil-pile interaction. The objective of this part of the study was to develop group amplification factors to amplify the single pile lateral deflection and maximum moment to values appropriate for the pile group. Termed as the Group Amplification Method (GAM), the pertinent equations are as follows:

\[ y_g = y_s A_y \quad \text{(3.5)} \]
\[ M_g = M_s A_M \quad \text{(3.6)} \]

where

\[ y_g = \text{lateral deflection of pile group (L)} \]
\[ y_s = \text{lateral deflection of single pile (L)} \]
\[ A_y = \text{deflection amplification factor (dimensionless)} \]
\[ M_g = \text{maximum moment of a pile in pile group (F.L)} \]
\[ M_s = \text{maximum moment of a single pile (F.L)} \]
\[ A_M = \text{moment amplification factor (dimensionless)} \]
Amplification factors were developed by running numerous LPILE analyses to obtain single pile deflections and moments, and by estimating group deflections and moments using the group equivalent pile (GEP) procedure (Mokwa and Duncan, 2000). The group amplification factors are then obtained from the ratio of group response to single pile response. The parametric study was performed for groups of fixed-head piles in cohesive and cohesionless soils with different soil parameters, embedment depths, pile arrangements and lateral load values. The analyses for cohesive and cohesionless soils were studied separately. When the pile extreme fiber stresses exceeded the allowable values, the pile failed structurally. These load cases are not included in the parametric study.

Before using the GEP procedure, the accuracy of the method was first verified by comparing the results of the GEP analyses with those using GROUP. More cases were analyzed showing good agreement overall but only two cases are illustrated in Figures 15 and 16 for load versus deflection, and in Figures 17 and 18 for load versus maximum pile moment. They included a 5 x 5 pile group in a soft cohesive soil and a 3 x 2 pile group in a loose cohesionless soil. In general, the two procedures provide close agreement.
Figure 15. Comparison of load versus deflection using GEP and GROUP for a R5C5S3 group of 48-inch-diameter drilled shafts with zero embedment in soft clay with $S_u = 0.5\, \text{ksf}$.

Figure 16. Comparison of load versus deflection using GEP and GROUP for R3C2S3 group of 48-inch-diameter drilled shafts embedded 4 feet in loose sand with $\phi = 30^\circ$. 

---

GROUP 4.0 data points — Group Equivalent Pile data points

---

GROUP 4.0 data points — Group Equivalent Pile data points
Figure 17. Comparison of load versus maximum moment using GEP and GROUP for a R5C5S3 group of 48-inch-diameter drilled shafts with zero embedment in soft clay with $S_v=0.5ksf$.

Figure 18. Comparison of load versus maximum moment using GEP and GROUP for a R3C2S3 group of 48-inch-diameter drilled shafts embedded 4 feet in loose sand with $\phi=30^\circ$.
The following pile spacings and embedment depths were used to develop the group amplification factors:

1. Pile spacing: 3, 4, 5 pile diameters (or pile widths)

2. Embedment depth below ground surface: 0, 2, 4, 6 and 10 feet

$p$-multiplier values vary with pile spacing, row position with respect to the direction of loading and soil type. In the GEP procedure, $p$-multipliers for all the piles in a group are first determined and then added for use in the analyses. $p$-multiplier values that were used to develop the group amplification factors are summarized in Table 6.

Table 6. $p$-multipliers for various pile spacings, pile locations and soil types

<table>
<thead>
<tr>
<th>S/D</th>
<th>$p$-multipliers for groups in cohesive soils (Rollins et al., 1998)</th>
<th>$p$-multipliers for groups in cohesionless soils (Pinto et al., 1997)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Leading row</td>
<td>1st trailing row</td>
</tr>
<tr>
<td>3</td>
<td>0.6</td>
<td>0.4</td>
</tr>
<tr>
<td>4</td>
<td>0.75</td>
<td>0.65</td>
</tr>
<tr>
<td>5</td>
<td>0.9</td>
<td>0.86</td>
</tr>
</tbody>
</table>

The same set of pile and soil properties used in the study on single piles was used in this portion of the study. The pile types used represent a wide range of flexural rigidities.
A total of 260 GEP analyses were performed to generate 2,200 load cases for groups in cohesive soils. The pile configuration, pile spacing, and the corresponding sum of the p-multipliers are summarized in Table 7.

Pile groups are labeled as follows: the first letter “R” indicates the number of rows, the letter “C” denotes the number of “columns”, and the letter “S” represents the pile spacing in terms of the number of pile diameters. The layout for pile group R3C2S4, which consists of 3 rows and 2 columns of piles spaced 4 diameters center-to-center, is shown in Figure 19. A total of eleven group configurations were analyzed. They include: R1C2, R1C5, R2C1, R2C2, R2C3, R2C4, R3C2, R3C3, R4C4, R5C1 and, R5C5.
Table 7. Configuration, spacing, and sum of p-multipliers for pile groups analyzed

<table>
<thead>
<tr>
<th>Group Designation</th>
<th>Number of Rows</th>
<th>Number of Columns</th>
<th>Dimensionless Pile Spacing (S/D)</th>
<th>$\Sigma p_m$ for Cohesive Soils</th>
<th>$\Sigma p_m$ for Cohesionless Soils</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1C2S3</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>1.2</td>
<td>1.6</td>
</tr>
<tr>
<td>R1C2S4</td>
<td>1</td>
<td>2</td>
<td>4</td>
<td>1.5</td>
<td>1.8</td>
</tr>
<tr>
<td>R1C2S5</td>
<td>1</td>
<td>2</td>
<td>5</td>
<td>1.8</td>
<td>2</td>
</tr>
<tr>
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</table>
Figure 19. Pile Configuration in a R3C2S4 pile group
For the lateral load behavior of pile groups in cohesionless soils, approximately 370 GEP analyses were performed to generate 2,880 load cases. The same pile properties, configurations, embedment depths and pile spacings for cohesive soils were used for cohesionless soils. Friction angles between 30 and 40 degrees were employed and, both submerged and dry conditions were examined. The p-y parameters for piles in cohesionless soils are summarized in Table 8.

Table 8. p-y parameters for piles in cohesionless soils

<table>
<thead>
<tr>
<th>Submerged/Dry</th>
<th>Relative Density</th>
<th>Friction angle (degrees)</th>
<th>Soil modulus (pci)</th>
<th>Effective unit weight (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Submerged</td>
<td>Loose</td>
<td>30</td>
<td>20</td>
<td>57</td>
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<td></td>
<td>Medium dense</td>
<td>35</td>
<td>60</td>
<td>57</td>
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<tr>
<td></td>
<td>Dense</td>
<td>40</td>
<td>125</td>
<td>57</td>
</tr>
<tr>
<td>Dry</td>
<td>Loose</td>
<td>30</td>
<td>25</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td>Medium dense</td>
<td>35</td>
<td>90</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td>Dense</td>
<td>40</td>
<td>225</td>
<td>120</td>
</tr>
</tbody>
</table>

Based on the parametric analyses, the amplification factors $A_y$ and $A_M$ have the following form:

$$A_y = \left( \frac{N_{\text{pile}}}{\sum p_m} \right)^{a_y}$$  \hspace{1cm} (3.7)

$$A_M = a_{M1} \left( \frac{N_{\text{pile}}}{\sum p_m} \right)^{a_{M2}} \left( \frac{S}{D} \right)^{a_{M3}}$$  \hspace{1cm} (3.8)

where
\[ \Sigma p_m = \text{sum of } p\text{-multipliers for all piles (dimensionless)} \]

\[ N_{\text{pile}} = \text{number of piles (dimensionless)} \]

\[ a_y, a_{M1}, a_{M2} \text{ and } a_{M3} = \text{dimensionless parameters for Equations 3.7 and 3.8} \]

(Table 9)

Table 9. Parameters for group amplification factors

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Cohesionless Soils</th>
<th>Cohesive Soils</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a_y)</td>
<td>0.717</td>
<td>1.31</td>
</tr>
<tr>
<td>(a_{M1})</td>
<td>0.567</td>
<td>0.475</td>
</tr>
<tr>
<td>(a_{M2})</td>
<td>1.2</td>
<td>1.194</td>
</tr>
<tr>
<td>(a_{M3})</td>
<td>0.352</td>
<td>0.421</td>
</tr>
</tbody>
</table>

In Figures 20 through 23, the predicted values of deflection and moment using the group amplification factor approach are compared with those from the GEP procedure for pile groups in cohesive and cohesionless soils. It can be seen that the new approach provides estimates of the group response reasonably well with biases of no more than 2% and coefficients of determination all above 0.99.
Figure 20. Comparison of group deflections using GAM versus GEP for piles in cohesive soils

\[ y = 0.9979x \]
\[ R^2 = 0.9991 \]

Figure 21. Comparison of group moment using GAM versus GEP for piles in cohesive soils

\[ y = 1.0227x \]
\[ R^2 = 0.9983 \]
Figure 22. Comparison of group deflections using GAM versus GEP for piles in cohesionless soils

Figure 23. Comparison of group moment using GAM versus GEP for piles in cohesionless soils
3.2.2 Steps for Simplified Analysis of Laterally Loaded Pile Groups

Steps for analyzing the behavior of laterally loaded pile groups using the group amplification factor approach are as follows:

1) Using the pile and soil properties for the group to be analyzed, estimate the lateral deflection and maximum moment of the single pile following the steps presented in Section 3.1.3. The lateral load that should be used for the single pile analysis is the lateral load on the pile group divided by the number of piles ($N_{pile}$) in the group.

2) Estimate the $p$-multipliers for each row of piles based on the center-to-center spacing and pile row.

3) Add the $p$-multipliers for all the piles together ($\Sigma p_m$).

4) Calculate $A_y$ and $A_m$ using Equations 3.7 and 3.8, respectively.

5) Calculate the group deflection ($y_g$) and maximum pile moment ($M_g$) using Equations 3.5 and 3.6, respectively.

The limitations for the GAM include:

1. The group amplification factors were developed for pile groups no larger than 5 x 5.
2. All the piles in each group must be identical.

3. The piles must be uniformly spaced.

4. This approach does not provide the load distribution among the piles in the group nor the maximum moment of each pile with the exception of those in the leading row.

5. Torsional effects are not considered in this study.
Several full-scale lateral load tests on fixed head pile groups have been reported in the literature (Brown et al., 2001; Huang et al., 2001; Kim and Brungraber, 1976; Mokwa and Duncan, 2000; Ng et al., 2001; Rollins and Sparks, 2002). The results for three case studies are compared with values predicted using the procedures developed in this study, namely the MCLM and the GAM. In general, the lateral loads applied in these load tests are either cyclic incremental or they are performed following a Statnamic load test. This may lead to a densification of cohesionless soils and a softening of cohesive soils. Nevertheless for the cyclic load tests, the deflections and moments from the first cycle of each load increment should closely approximate those from a static loading condition, and are therefore useful for validating the analytical tools developed herein.

4.1 Case Study 1 - Pile Group in Cohesive Soils

Rollins and Sparks (2002) performed two series of lateral load tests on a fixed-head pile group at the Salt Lake City International Airport in Utah. A Statnamic load test was performed to study the lateral response of the pile group
followed by a conventional static lateral load test. Loads for the static load test were applied in the same direction as the Statnamic loads.

Nine piles, spaced three diameters apart center-to-center, were driven in a 3 x 3 arrangement. The piles were embedded in a 4-foot-thick and 9-foot-square reinforced concrete pile cap. The pile cap was supported on six inches of compacted granular fill. One side of the pile cap was compacted with granular fill extending to the top of the pile cap (Figure 24) to provide passive resistance. The piles consisted of 30-foot-long, 12½-inch-OD, 0.375-inch-thick-wall, concrete filled, closed-end steel pipe piles.

The flexural rigidity (EI) of the pile was estimated as follows:

$$EI = E_s I_s + E_c I_c$$  \hspace{1cm} (4.1)$$

where $I_s$ and $I_c$ are the moment of inertia of the steel pipe and concrete, respectively, $E_c (= 2960$ ksi based on $f_c' = 2700$ psi) and $E_s (= 29,000$ ksi) are the Young's modulus of the concrete and steel, respectively. The flexural rigidity of the pile composite was estimated to be $1.11 \times 10^7$ kip-in$^2$. 
Figure 24. Pile test layout for case study 1 (a) plan view, (b) and cross-section (Rollins and Sparks, 2002)

Figure 25. Comparison of measured versus predicted deflections using GAM for case study 1
Subsurface soils at the site consist predominantly of soft clay. The natural ground water table was reported to be immediately below the granular fill. Rollins and Sparks neglected the soil resistance of the top four pile diameters to account for a gap generated between the pile and the soil as a result of the Statnamic testing. Based on this, the clay over the top 12 pile diameters had an average undrained shear strength of 0.53 ksf. The undrained shear strength was estimated based on triaxial unconsolidated undrained tests, vane shear tests, and pressuremeter tests.

For this load test, the lateral resistance of the fixed head pile group can be taken as the sum of the pile group and cap resistance. Rollins and Sparks (2002) calculated the cap resistance and added it to the pile resistance at the same deflection to obtain an overall load-deflection curve. They estimated the pile group resistance at each value of deflection using GROUP. The pile group resistance was also estimated using the methods developed in this study. The predicted load-deflection curve compares quite favorably to the actual test results as shown in Figure 25. The calculated maximum bending moment did not compare as favorably with the measured values for the piles in the leading row as shown in Figure 26. This is probably due to rotation of the pile cap, which led to a reduction in the maximum moment at the pile head. The estimated values of moment are about 79 to 188 percent higher than the measured values.
4.2 Case Study 2 - Pile Group in Cohesive Soils

Kim and Brungraber (1976) reported multiple series of full-scale lateral load tests on three groups of 2 x 3 10BP42 steel H-piles at the Bucknell University campus in Lewisburg, Pennsylvania. A total of three load test series (Series A, B and C) were conducted on three adjacent groups (I, II and III) of piles. Load test Series B on Group II with a spacing of 3.7 pile widths on center was used in this study (Figure 27). The piles were 9.7 inches wide, 40 feet long, and were connected by a concrete pile cap. The moment of inertia of
Figure 27. Pile test layout for case study 2 (Kim and Brungraber, 1976)

Figure 28. Comparison of measured versus predicted deflections using GAM for case study 2
the pile was 224.2 in\(^4\). The pile cap was 4 feet thick, 10 feet long and 8 feet wide.

Subsurface conditions over the top 8 shaft diameters consisted of silty clay with an average undrained shear strength of 2.5 ksf, estimated based on unconfined compression tests and standard penetration test data. The ground water table was reported to be at a depth of 35 feet.

In this load test, the cap was at the ground surface with zero embedment. As a result, the pile cap resistance is not considered significant and the lateral resistance can be considered to be due primarily to the pile group. An axial load of 72 kips per pile was applied. The lateral load-deflection curve for the group was estimated using the procedures developed and is compared to measured values in Figure 28. The measured load deflection curve compares reasonably with the calculated curve. Note that the measured deflections increased by only 70 percent and measured moments increased by only 42 percent when the load was doubled. This indicates inconsistency in the measurements, because these quantities would increase by a factor of two or higher if the behavior was truly nonlinear.

The lateral load-moment curve for the group was estimated using the procedures developed and compared to average measured values of the piles in the leading row. Figure 29 showed that the estimated maximum bending
moments are nine percents below to 52 percent higher than the measured value for the leading row of piles.

![Graph showing comparison of predicted versus measured bending moment for Bucknell University load test]

Figure 29. Comparison of predicted versus measured bending moment for Bucknell University load test

4.3 Case Study 3 - Pile Group in Cohesionless Soils

Huang et al. (2001) and Brown et al. (2001) reported a full-scale lateral load test on a group of six bored piles reacting against a group of twelve precast concrete piles for the proposed high-speed rail system project in Taipao Township of Chaiyi County, Taiwan. The bored piles had a diameter of 59 inches, were 114 feet long, spaced 3 diameters on center and were connected
by a pile cap. The flexural stiffness of the drilled shaft was $2.39 \times 10^9$ kip-in$^2$. The pile cap was 6.5 feet thick, 72 feet long and 49 feet wide. Subsurface conditions over the top 8 shaft diameters consisted of loose silty sand with an average friction angle of 35 degrees (Brown et al., 2001). The ground water table was reported to be at a depth of 3.3 feet. The p-multipliers for the first, second and third rows are 0.5, 0.4 and 0.3, respectively (Brown et al., 2001).

In this load test, the cap was at the ground surface with zero embedment (Figure 30). As a result, the pile cap resistance is not considered to be significant and the lateral resistance can be considered to be due primarily to the pile group. The lateral load-deflection curve for the group is estimated using the procedures developed and is compared to measured values in Figure 31. It can be seen that in general, there is good agreement between the estimated deflections and the measured values. The estimated maximum bending moment (8,850 kip-ft) was 23 percent lower than the measured value in the leading row (11,551 kip-ft). Only one value of bending moment was published corresponding to a lateral load of 410 kips.
Figure 30. Pile test layout for case study 3 (Huang et al., 2001)

Figure 31. Comparison of measured versus predicted deflections using GAM for case study 3
CHAPTER 5

SUMMARY AND CONCLUSIONS

The Modified Characteristic Load Method (MCLM) was developed for estimating single pile head deflection and maximum bending moment for fixed-head piles embedded below ground surface in cohesive soils. This simple procedure requires pile diameter, pile elastic modulus, moment of inertia, pile head embedment depth, undrained shear strength and soil unit weight as input parameters. The MCLM provides reliable values of pile deflections and maximum moments when compared with LPILE Plus.

The Group Amplification Method (GAM) was developed to predict the lateral behavior of fixed-head pile groups. This procedure was derived based on the group equivalent pile method proposed by Mokwa and Duncan (2000). The GAM can be used to amplify the single pile deflection and bending moment to predict the pile group deflection and maximum moment. The amplification factors depend on the number of piles in the group, pile spacing and p-multipliers. The p-multipliers depend on soil type, pile spacing, pile position within the group relative to the direction of the applied force and construction method.
The GAM in conjunction with the MCLM was used to predict the behavior of three full-scale pile groups that were load tested. These predictions indicate that the procedures provide estimates of group deflections and moments that are accurate enough for most practical purposes or they provide results that err on the side of conservatism.
REFERENCES


