Pile Spacing Effects on Lateral Pile Group Behavior: Analysis

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Abstract: Using the results from three full-scale lateral pile group load tests in stiff clay with spacing ranging from 3.3 to 5.65, computer analyses were performed to back-calculate *p* multipliers. The *p* multipliers, which account for reduced resistance due to pile–soil–pile interaction, increased as pile spacing increased from 3.3 to 5.65 diameters. Extrapolation of the test results suggests that group reduction effects can be neglected for spacings greater than about 6.5 for leading row piles and 7–8 diameters for trailing row piles. Based on analysis of the full-scale test results, pile behavior can be grouped into three general categories, namely: (1) first or front row piles; (2) second row piles; and (3) third and higher row piles. *p* multiplier versus normalized pile spacing curves were developed for each category. The proposed curves yield *p* multipliers which are higher than those previously recommended by AASHTO in 2000, the US Army in 1993, and the US Navy in 1982 based on limited test data, but lower values than those proposed by Reese et al. in 1996 and Reese and Van Impe in 2001. The response (load versus deflection, maximum moment versus load, and bending moment versus depth) for each row of the pile groups computed using GROUP and Florida Pier generally correlated very well with measurements from the full-scale tests when the *p* multipliers developed from this test program were employed.

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CE Database subject headings: Piles; Spacing; Pile groups.

Introduction

The lateral load resistance of pile foundations is critically important in the design of structures which may be subjected to earthquakes, high winds, wave action, and ship impacts. Because of the high cost and logistical difficulty of conducting lateral load tests on pile groups, relatively few full-scale load test results are available that show the distribution of load within a pile group (Brown et al. 1987, 1988; Meimon et al. 1986; Ruesta and Townsend 1997; and Rollins et al. 1998). Nevertheless, the results from these tests indicate that the average load for a pile in a closely spaced group [three pile diameter or 3D spacing center-to-center] will be substantially less than that for a single isolated pile at the same deflection and that front (leading) row piles in the group will carry significantly higher loads than trailing row piles at the same deflection. The piles in trailing rows are thought to exhibit less lateral resistance because of interference ("shadowing") with the failure surface of the row of piles in front of them. This shadowing or group interaction effect is expected to become less significant as the spacing between piles increases so that there is less overlap between adjacent failure planes.

The lateral response of piles is typically analyzed using finite-difference (Matlock and Reese 1960) or finite-element methods (Hoit et al. 1997). The pile is modeled as a beam and the soil is modeled using either a constant modulus of subgrade reaction or nonlinear springs that are attached to the pile. The nonlinear springs are defined using *p*–*y* curves at regular depth intervals, where *p* represents the lateral soil resistance per unit length of the pile and *y* is the lateral deflection of the pile at that depth.

One method of accounting for the shadowing or group reduction effects is to reduce the modulus or the soil resistance, *p*, from a single pile *p*–*y* curve using a constant reduction factor or *p* multiplier (f_m) as proposed by Brown et al. (1988). Although this simple approach has provided relatively good estimates of measured pile group behavior (Rollins et al. 1998; Brown et al. 1988), *p* multipliers are extremely restricted in their application. For example, all of the available full-scale pile group tests for which *p* multipliers have been back-calculated, involve pile groups spaced at approximately 3 pile diameters center-to-center as summarized in Table 1. The variation of *p* multipliers with increasing row spacing has thus far been determined only through model tests.

Because of this lack of data, there is considerable variation in the recommendations of various agencies regarding *p* multipliers as a function of spacing. For example, Figs. 1(a and b) show the *p* multipliers back-calculated from full-scale load tests for leading and trailing row piles, respectively, along with reduction factors

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Table 1. Summary of Row Spacing and p Multipliers Back-Calculated for Previous Pile Group Load Tests

Reference	Normalized spacing (S/D)	p multipliers (f_m) Row 1	Row 2	Row 3	Row 4
Rollins et al. (1998)	2.82	0.6	0.4	0.4	—
Ruesta and Townsend (1997)	3	0.8	0.7	0.3	0.3
Brown et al. (1988)	3	0.8	0.4	0.3	—
Brown et al. (1987)	3	0.7	0.6	0.5	—
Meimon et al. (1986)	3	0.9	0.5	—	—

recommended by Reese et al. (1996); Reese and Van Impe (2001); WSDOT (2002); AASHTO (2000); and the US Army (1993) as a function of normalized pile spacing (center to center spacing/pile diameter). The curves recommended by AASHTO (2000) are identical to curves recommended by the US Navy (1982) and the Canadian Geotechnical Society (1992), which suggests that this may be the most widely used curve. Nevertheless, the variation in the curves in Fig. 1 indicates that there is still considerable uncertainty about appropriate reduction factors to account for group effects. For example, at a spacing of 5 pile diameters, recommended p multipliers range from nearly 0.4 to 1.0.

The p multipliers based on the full-scale test results are significantly lower than the p multipliers recommended in GROUP (Reese et al. 1996). Therefore, use of these p multipliers may be nonconservative and could result in lower than expected resistance. The AASHTO and US Army curves appear to provide relatively conservative estimates of the p multipliers based on the available full-scale tests. This is particularly true for the front (leading) row piles. Therefore, use of the AASHTO or US Army curves could lead to unnecessarily expensive pile foundation designs. The WSDOT curve fits well with the full-scale results at 3D spacing but is higher than the other curves at greater spacings. Considering the variation in p multiplier recommendations and the potential for either unsafe or unnecessarily costly foundations, additional full-scale tests are clearly needed to develop reliable p multiplier versus pile spacing curves that can be used for engineering design.

In addition, there is uncertainty about whether the p multiplier measured for the third row in a group is appropriate for subsequent rows in a large pile group or whether the p multipliers will continue to gradually decrease with each additional trailing row as observed for the second and third rows. Recent centrifuge test results in sands (McVay et al. 1998) suggest that p multipliers may become constant for greater numbers of rows; however, no test results are yet available for clays.

To improve our understanding of pile group behavior, a series of full-scale lateral load tests were performed on three pile groups at various spacings and with up to five rows of piles. The results from these load tests are detailed in a companion paper (Rollins et al. 2006). This paper describes methods used to determine appropriate p multipliers as a function of pile spacing based on the pile group tests. p multiplier versus pile spacing curves developed in this study are also compared with previous recommendations. Analyses are then performed to compare the measured response

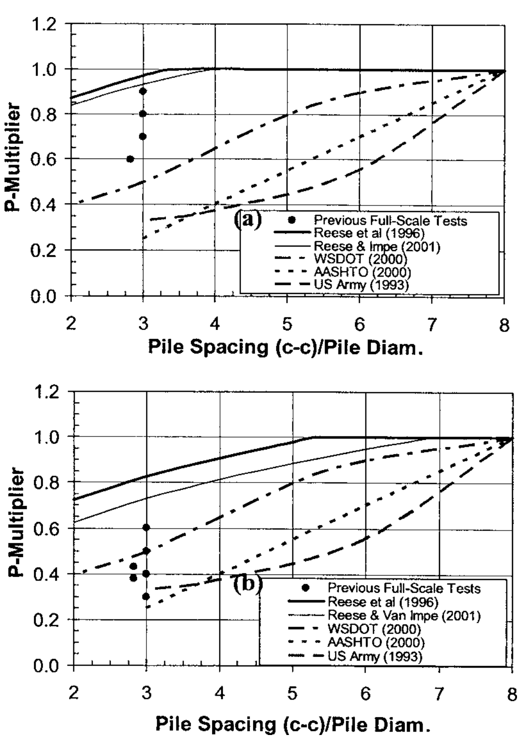

Fig. 1. Available p multiplier versus spacing relationships for: (a) front row; (b) trailing row piles in comparison with values back-calculated from previous full-scale load tests

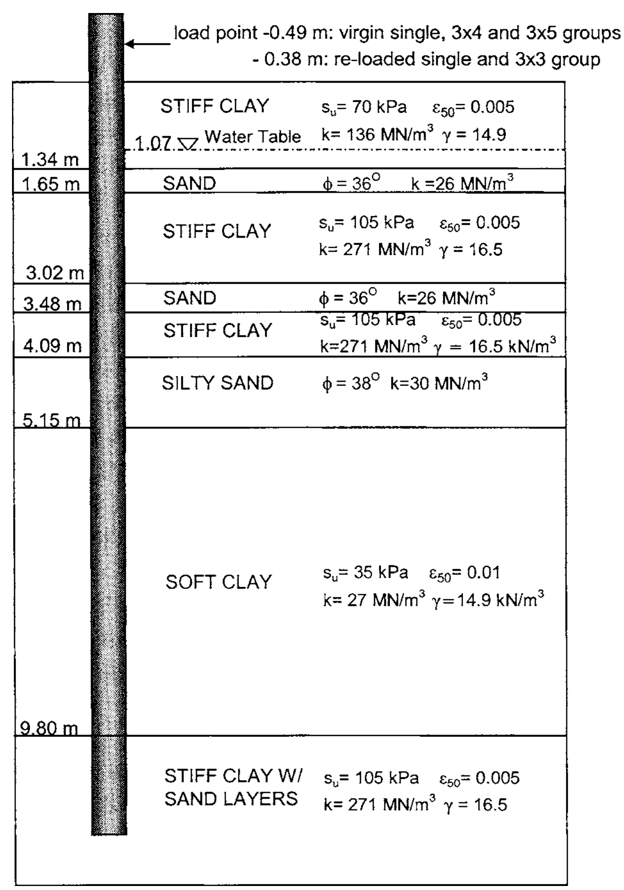

Fig. 2. Idealized soil profile with soil properties used in computer analysis

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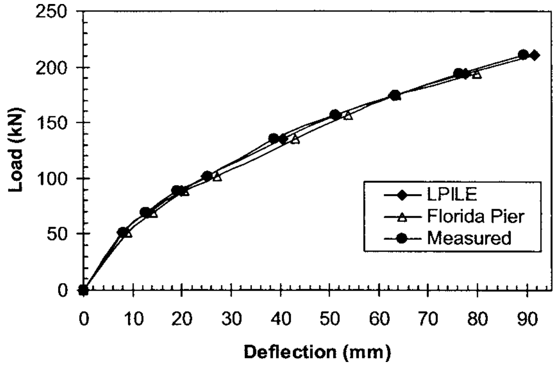

Fig. 3. Comparison of measured load versus deflection curve for 324-mm-diameter single pile with curves computed using computer programs LPILE and FLPIER

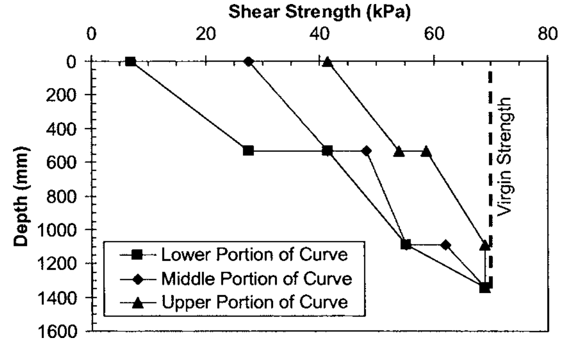

Fig. 5. Three soil strength profiles used in LPILE to model increasing resistance around single pile as gap between pile and soil closed with increased deflection

with that computed using the back-calculated *p* multipliers. Finally, an example is provided for evaluating pile group behavior using the *p* multiplier versus pile spacing design curves developed in this study.

Geotechnical Site Conditions

The subsurface profile was characterized using a variety of methods to provide basic geotechnical data for use in subsequent computer analyses of the test results. Additional details are provided by Rollins et al. (2003) and Rollins et al. (2006). Based on the results of the field and laboratory testing, the soil profile shown in Fig. 2 was developed. The soil profile generally consists of overconsolidated stiff clays with some sand layers to a depth of 5 m. The sand layers were in a medium compact density state ($D_r \approx 60$). Groundwater was located at a depth of 1.07 m during testing. The stiff clay was underlain by softer sensitive clays which were in turn underlain by interbedded layers of silty clay and sand. Cone penetration test (CPT) soundings were performed at each test foundation to define the stratigraphy and the variation across the site. These tests confirmed that the profiles were very similar at each test site.

The vane shear test was the primary means for evaluating the undrained shear strength of the clays. In addition, undrained shear strength was obtained from unconfined compression tests on undisturbed samples and from correlations with the CPT cone resistance. In general, the agreement between the strength evaluation methods was very good. The undrained strength values used subsequently in the analyses are also identified in Fig. 2. The friction angles for the sand layers were determined by borehole shear tests and they typically varied from 36 to 38°.

Single Pile and Pile Group Test Characteristics

Lateral load tests were performed on two isolated single piles and three pile groups. The single pile tests were necessary to provide a comparison to the behavior of the pile groups. The tests piles in all cases were 324 mm o.d. steel pipe piles (9 mm wall thickness) and were driven closed ended to a depth of approximately 11.9 m below the ground surface. The steel conformed to ASTM A252 Grade 3 specifications and had an average yield strength of 404.6 MN/m^2 (58,700 psi) based on the 0.2% offset criteria. The moment of inertia of the piles was 1.16×10^8 mm^4 (279 $in.^4$). Two angle irons were attached to each pile to protect the strain gages, which increased the moment of inertia to 1.43×10^8 mm^4 (344 $in.^4$).

The first pile group consisted of piles in a 3×3 arrangement with a longitudinal spacing of 5.65 pile diameters on centers. The second group consisted of piles in a 3×4 arrangement with a longitudinal spacing of 4.4 pile diameters and the third group consisted of piles in a 3×5 arrangement with a longitudinal spacing of 3.3 pile diameters. The transverse spacing in all cases was 3.3 pile diameters.

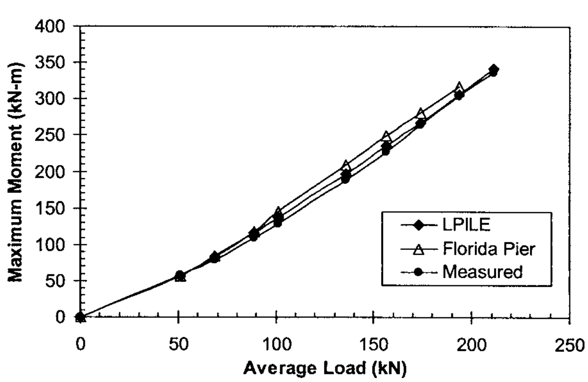

Fig. 4. Comparison of measured maximum bending moment versus load curve with curves computed using computer programs LPILE and FLPIER

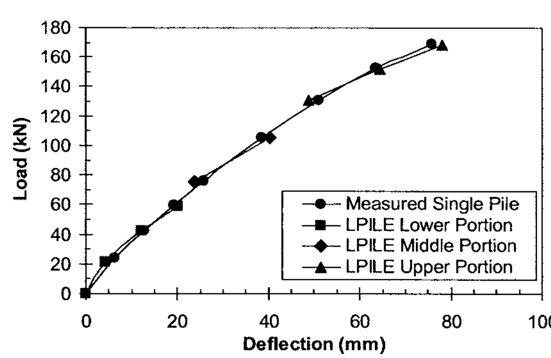

Fig. 6. Measured load versus deflection curves along with computed load versus deflection curves obtained using three soil strength profiles in Fig. 14 to model gap behavior

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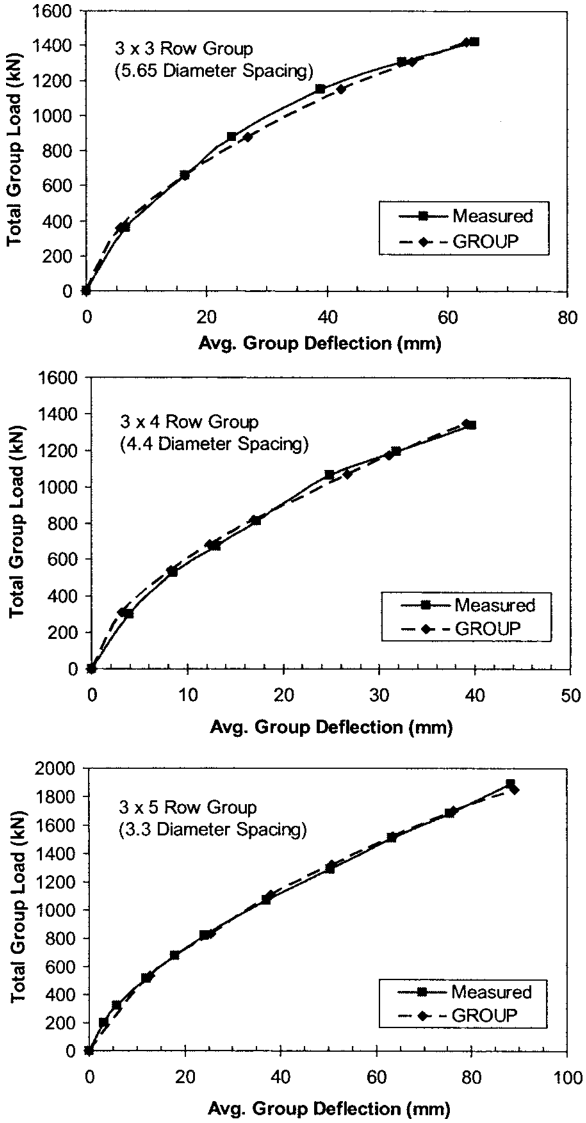

Fig. 7. Comparison of measured total load–deflection curves with curves computed by GROUP using *p* multipliers developed in this study

Each pile in each group was attached to the load frame by a tie-rod load cell with a pinned connection which produced a free-head condition. The frame was designed to be essentially rigid relative to the piles so that each pile was constrained to have the same deflection for a given load. In addition to measuring the load and deflection in each pile throughout the test, strain was measured along the length of the center test pile within each row to determine bending moment profiles.

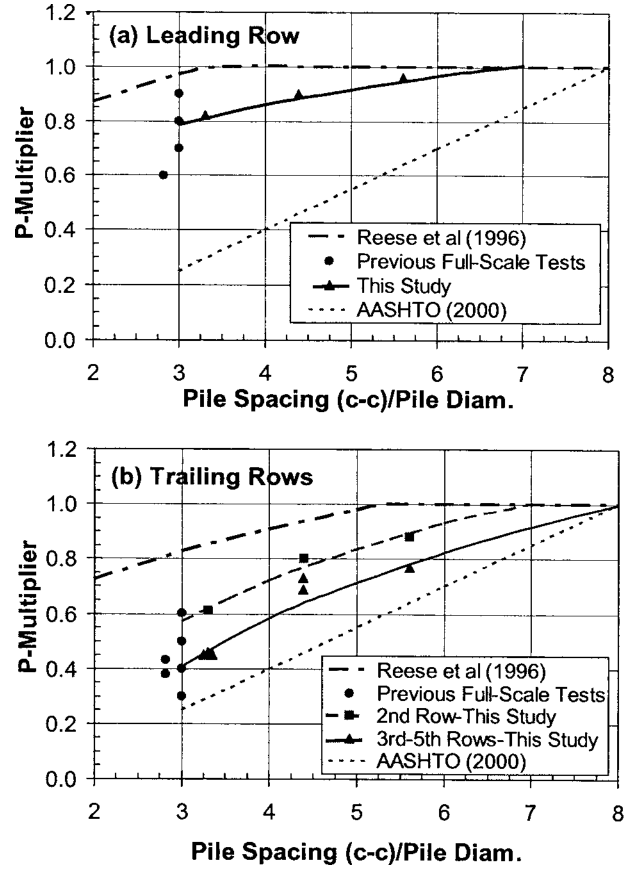

Fig. 8. Back-calculated *p* multipliers for: (a) leading row; (b) trailing row piles from this study and previous full-scale load tests along with recommended design curves

The first single pile test was a virgin load test, while the second pile test was performed on a pile that had previously been loaded in another direction. The reload test was necessary to provide a comparison with one of the pile groups that was loaded statically in one direction after it was loaded dynamically in the opposite direction using the statnamic method.

Analysis of Static Load Tests and Determination of *p* Multipliers

Using the measured soil profile and properties along with the pile properties described previously, computer analyses were performed to obtain the best possible match between the measured and computed response for the single pile tests. During this phase of the analysis, minor modifications in soil properties were per-

Table 2. Summary of Normalized Row Spacing and *p* Multipliers Back-Calculated for Each Row in Each Pile Group During This Study

Normalized spacing (S/D)	Deflection range (mm)	*P* multipliers (f_m) Row 1	Row 2	Row 3	Row 4	Row 5
5.65	0–65	0.95	0.88	0.77	—	—
4.4	0–40	0.90	0.80	0.69	0.73	—
3.3	0–90	0.82	0.61	0.45	0.45	0.51–0.46[a]

[a]0.51 for deflections less than 50 mm and 0.46 for greater deflections.

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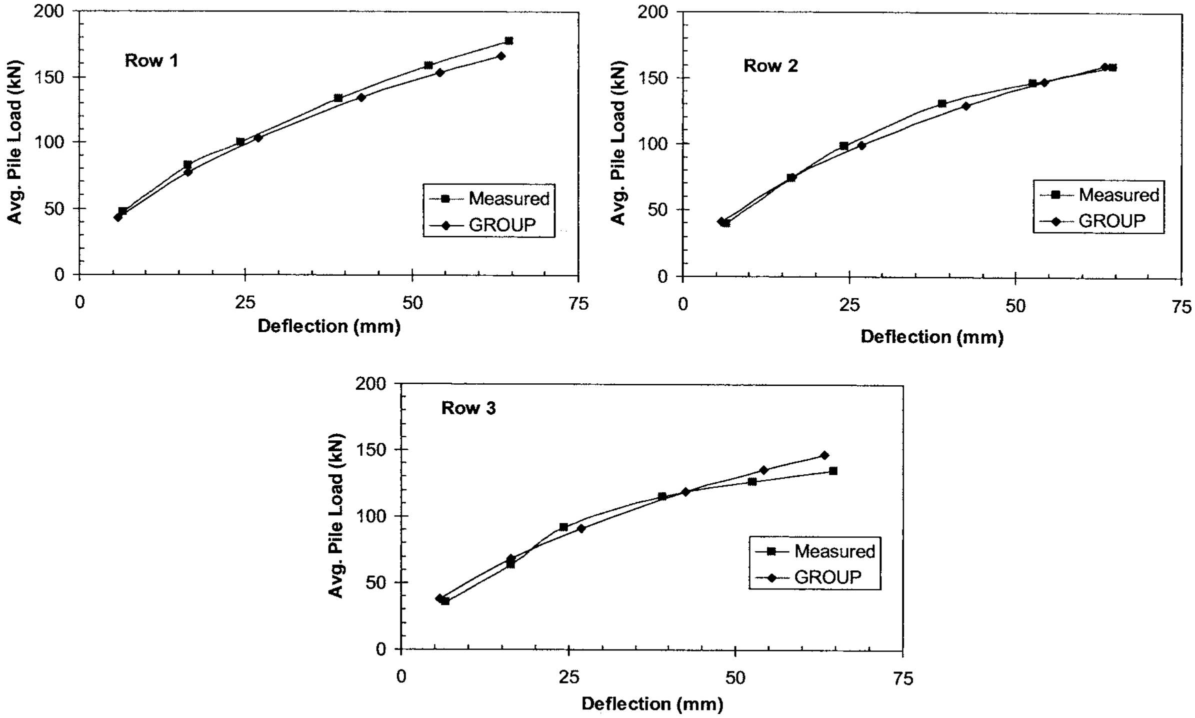

Fig. 9. Comparison of measured load–deflection curves with curves computed by GROUP using back-calculated p multipliers for each row in 3×3 pile group at 5.65 pile diameter spacing

mitted to improve the match. The final soil profile and properties are shown in Fig. 2. Subsequently, these properties were held constant in the analysis of the pile groups and only variations in the p-multiplier values were used to obtain the best agreement between measured and computed pile group response.

Analyses of single pile tests were made using the computer programs LPILE (Reese and Wang 1997) and Florida Pier (FLPIER) (Hoit et al. 1997). LPILE uses a finite difference approach while FLPIER uses a finite element approach. The p–y curves for the stiff clay were obtained using the model developed by Reese and Welch (1975) and the p–y curves for soft clay were computed using the method proposed by Matlock (1970). p–y curves in the sand layers were obtained from equations developed by Reese et al. (1974). Values for k and ϵ_{50} were selected based

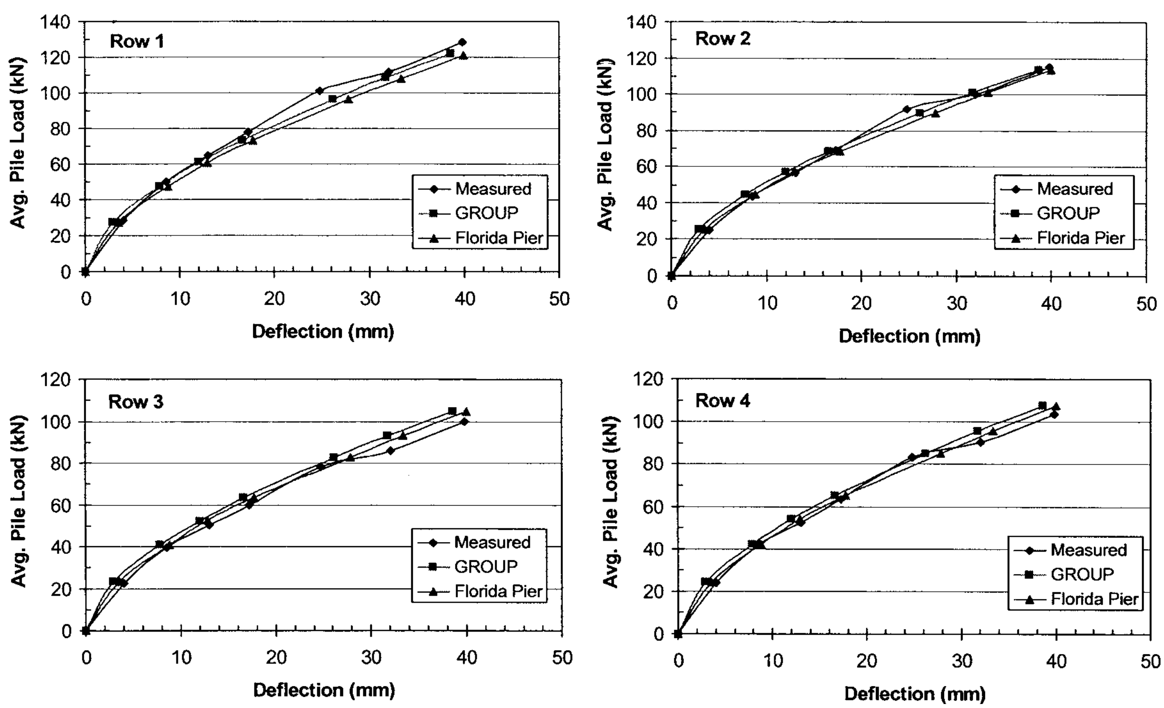

Fig. 10. Comparison of measured load–deflection curves with curves computed by GROUP using back-calculated p multipliers for each row in 3×4 pile group at 4.4 pile diameter spacing

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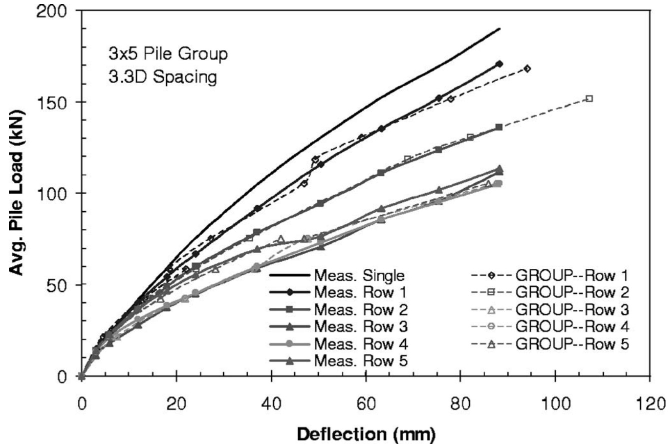

Fig. 11. Comparison of measured load–deflection curves with curves computed by GROUP using back-calculated p multipliers for each row in 3×5 pile group at 3.3 pile diameter spacing

on strength test results and correlations (Reese and Wang 1997). The load versus deflection and bending moment versus load curves computed using these two programs are compared with the measured curves in Figs. 3 and 4, respectively. Very little manipulation of the input parameters was required to achieve this excellent agreement. In general, the changes in the strength properties were less than about 10% of the measured values, which is well within the typical error range for most undrained strength measurements (Duncan 2000).

Despite the excellent agreement shown in Figs. 3 and 4 for virgin load conditions, neither of the computer programs was capable of matching the load–deflection curve for the reloaded single pile without significant manipulation of the input parameters. This result points out the need for improved models to account for pile behavior when gaps are present. To model the measured load versus deflection curve, it was necessary to use three different soil strength profiles within the measured depth of the gap (1.4 m) as shown in Fig. 5. The soil resistance in each model was progressively increased as the deflection increased and the pile came into contact with the soil. The initial strength was not recovered due to gapping on the sides of the pile. The properties of all other layers below the gap depth in the stiff clay layer remained unchanged. In addition, the pile properties were kept the same as described previously.

Using the three profiles shown in Fig. 5, three separate segments of the load–deflection curve were computed to match the measured load–deflection curve as shown in Fig. 6. Although this match was obtained by trial and error, the results are still useful in back-calculating p multipliers for the 3×5 pile group as will be discussed subsequently.

Once the soil profile and properties had been established based on the single pile analysis, the same profile and properties were used in the pile group analysis with the computer program GROUP to back-calculate appropriate p multipliers. Initially, p multipliers for each row were estimated based on the ratio of average pile load within a row to the single pile load at the same deflection. The p multipliers were then adjusted to obtain the best match between the measured and computed total load–deflection curves for each group. The measured total load–deflection curves for each group are compared with the load–deflection curves computed by GROUP using these back-calculated p multipliers in Fig. 7. The use of these simple p multipliers generally provided remarkably good agreement with the measured response.

The back-calculated p multipliers for each group test are summarized in Table 2. A review of the results in Tables 1 and 2 indicates that the p multipliers for the row 1 (front or leading row) piles are significantly higher than those for the trailing row piles. In addition, the results from this study suggest that the p multi-

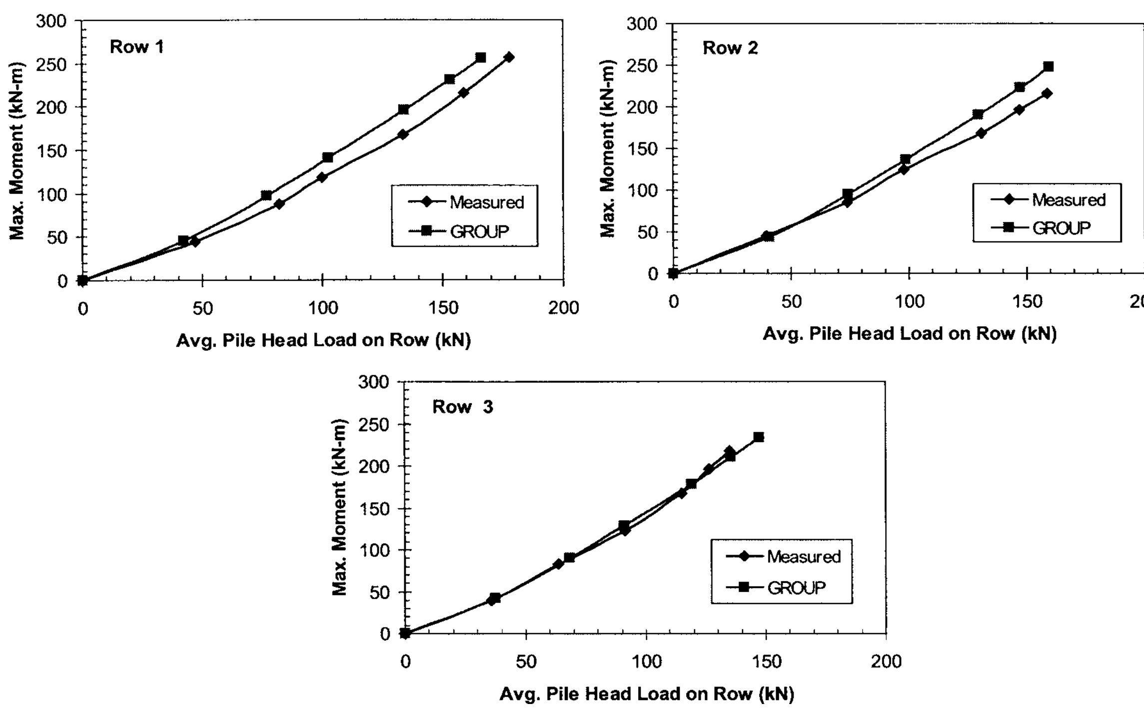

Fig. 12. Comparison of measured maximum bending moment versus pile head load curves for each row of 3×3 pile group at 5.65 pile diameter spacing relative to curves computed using GROUP with p multipliers developed in this study

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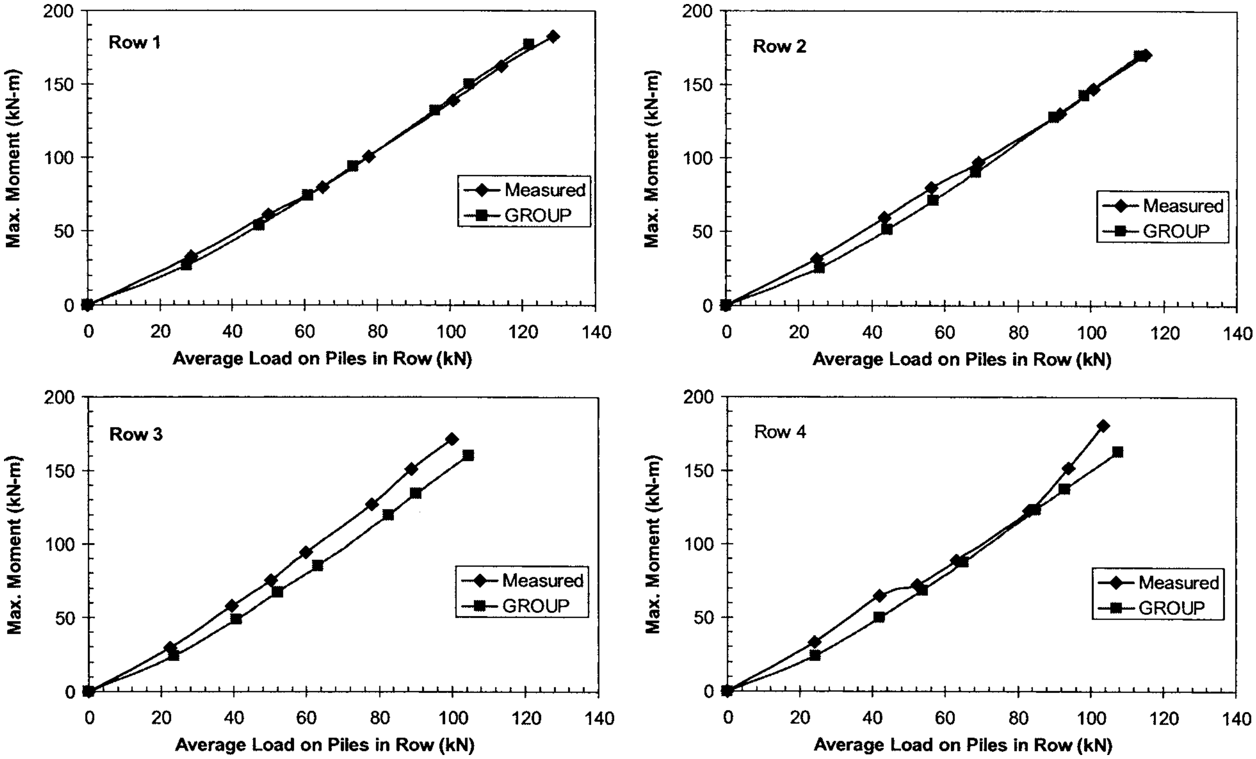

Fig. 13. Comparison of measured maximum bending moment versus pile head load curves for each row of 3×4 pile group at 4.4 pile diameter spacing relative to curve computed using GROUP with p multipliers developed in this study

pliers for the second row of piles are also noticeably higher than those for the third and subsequent rows. However, the p multipliers tend to remain about the same for the third and subsequent rows when present.

The back-calculated p multipliers for the leading row (row 1) piles in each group are plotted versus pile spacing in Fig. 8(a) while the p multipliers for the trailing row piles are shown in Fig. 8(b). p multipliers obtained from previous full-scale load testing (Brown et al. 1987, 1988; Meimon et al. 1986; Ruesta and Townsend 1997; and Rollins et al. 1998) are also shown in Fig. 8 for comparison. The p multipliers from this study are within the middle of the range of values from previous tests at the closest spacings.

Proposed design curves, which show p multiplier values as a function of pile spacing, have been developed based on the results from this study and the curves for leading and trailing row piles are presented in Figs. 8(a and b), respectively. Two curves are provided for trailing row piles in Fig. 8(b) to account for the variation in lateral resistance exhibited between the second row and other trailing row piles. The upper curve gives p multipliers for the second row (or first trailing row) in the group, while the lower curve gives the p multiplier for all other trailing rows in the group. For both leading and trailing row piles, there is a clear trend for the p multipliers to increase as the spacing increases; however, the relationship does not appear to be linear. The p multipliers tend to increase more rapidly from 3D to 5D, but then more gradually for spacings greater than 5D. Extrapolation of the curves suggests that the p multipliers will become one at a spacing of about 6.5 diameters for the leading row piles and between 7 and 8 diameters for the trailing rows.

Equations have also been developed to compute the p multiplier (f_m) for each of the curves shown in Fig. 8. The equations for each case are

first (lead) row piles

$$f_m = 0.26 \ln(S/D) + 0.5 \leq 1.0 \tag{1}$$

second row piles

$$f_m = 0.52 \ln(S/D) \leq 1.0 \tag{2}$$

third or higher row piles

$$f_m = 0.60 \ln(S/D) - 0.25 \leq 1.0 \tag{3}$$

where S=center-to-center spacing between piles in the direction of loading and D=width or outside diameter of the pile. The f_m values are limited to 1.0.

The p multiplier versus pile spacing curves suggested by Reese et al. (1996) and employed in GROUP are also presented in Figs. 8(a and b) for comparison along with the curve recommended by AASHTO (2000). The p multipliers based on the results from this and previous full-scale group load tests are significantly lower than the curves used in GROUP, particularly for the closest spacing. In addition, the curves used in GROUP assume that group interaction effects are eliminated at much smaller spacings than are indicated by this series of tests.

The AASHTO curve consistently underestimates the back-calculated p multipliers determined from this study. The error is greatest for the front row piles; however, there is still significant error for the trailing row piles. Therefore, use of the AASHTO curve will often lead to more conservative and more expensive foundation designs than the proposed curves.

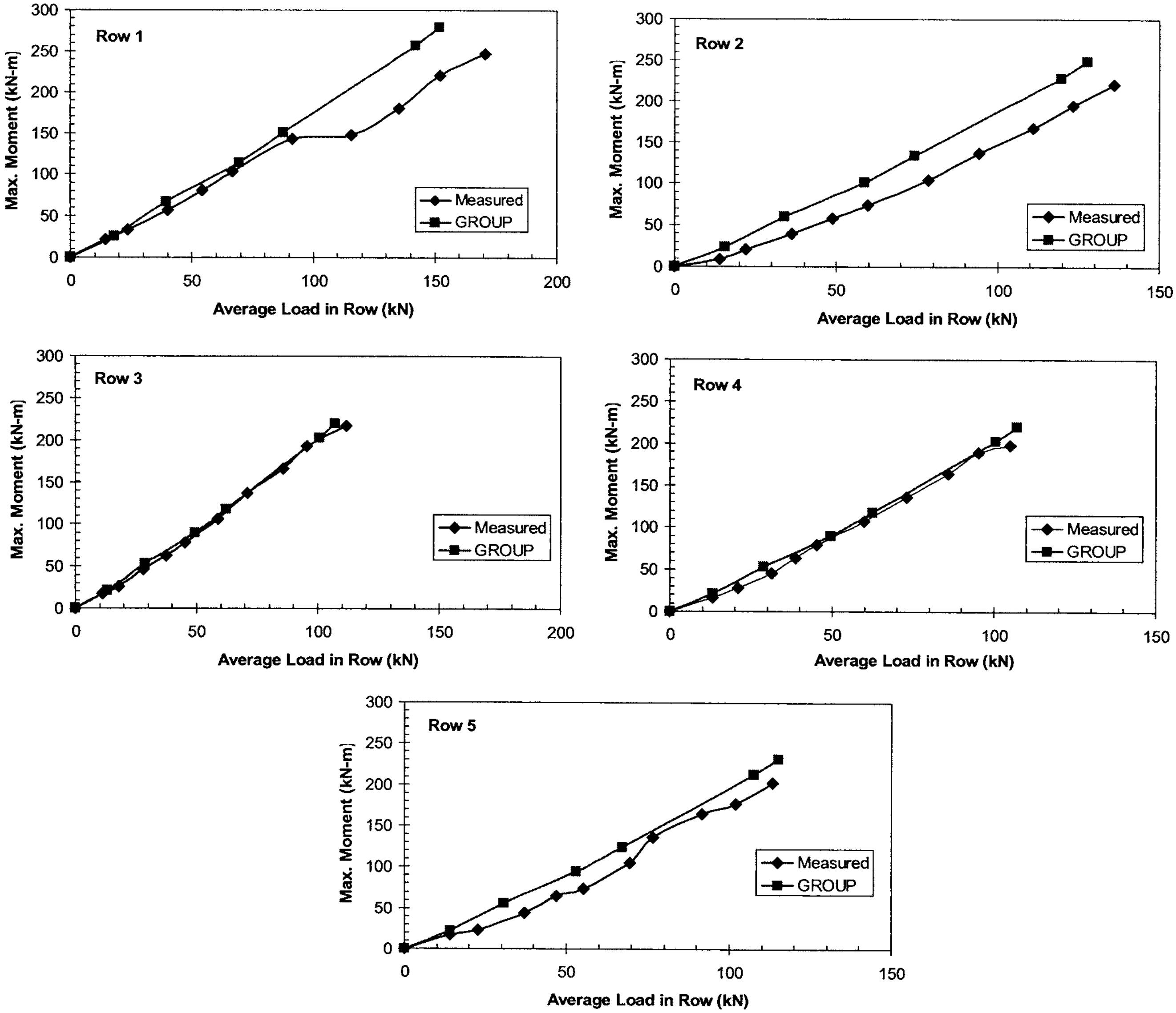

Fig. 14. Comparison of measured maximum bending moment versus pile head load curves for each row of 3×5 pile group at 3.3 pile diameter spacing relative to curve computed using GROUP with *p* multipliers developed in this study

Comparison of Measured and Computed Response

The back-calculated *p* multipliers were then used in computing load–deflection curves, bending moment–load curves, and bending moment–depth curves for each row without further adjustment of soil or pile properties.

Load–Deflection Curves

Figs. 9–11 provide plots of the measured load–deflection curves for each row in the 3×3, 3×4, and 3×5 pile groups, respectively. Load–deflection curves for each row computed using GROUP with the *p* multipliers developed during this study are also plotted in Figs. 9–11 for comparison. For the 3×4 pile, similar analyses with Florida Pier were also performed. The agreement is very good particularly considering the simplicity of the adjustment factor and the range of pile spacings involved. The curves computed using GROUP and Florida Pier in Fig. 10 are essentially the same and indicate that the adjustment factors are not sensitive to the numerical method employed.

Bending Moment–Load Curves

Figs. 12–14 provide plots of the measured maximum bending moment versus pile load curves for each row in the 3×3, 3×4, and 3×5 pile groups, respectively. The load is the average load carried by the piles in each row and the moment is the maximum along the length of a pile in that row. Maximum moment versus load curves computed using GROUP with the *p* multipliers developed during this study are also provided in Figs. 12–14 for comparison. Generally, the agreement between measured and computed moment is very good; however, in some cases the deviations are 15–20%. In some cases, some of this error may be a result of uncertainties in interpretation of bending moment from the strain gages as well as the inadequacy of the numerical models. Computed curves using Florida Pier in Fig. 13 provided similar results to GROUP.

Bending Moment–Depth Curves

Bending moment versus depth curves are plotted for each row of the 3×3, 3×4, and 3×5 pile groups at one deflection increment

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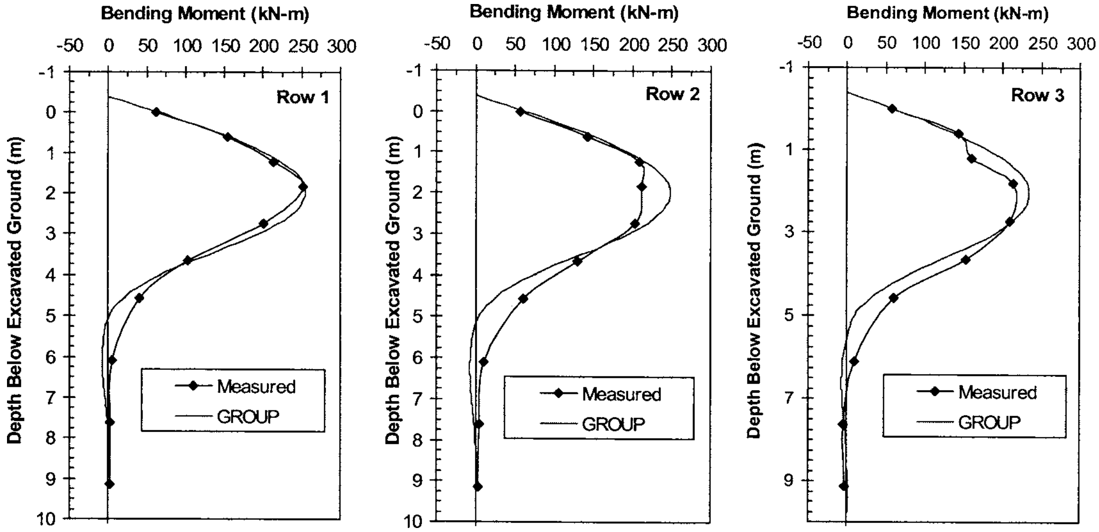

Fig. 15. Measured bending moment versus depth curves for each row of 3×3 pile group at deflection of 64 mm comparison to curves computed using GROUP with p multipliers developed during this study

in Figs. 15–17, respectively. Bending moment versus depth curves computed using the computer program GROUP with the p-multipliers developed in this study are also presented in Figs. 15–17 for comparison purposes. Curves computed using Florida Pier are also provided in Fig. 16. Generally, the computer programs were successful in predicting the depth to the maximum moment and the shape of the curve to this depth or somewhat deeper. However, at greater depths, the computed moments tended to be less than the measured curves. This discrepancy was common to both GROUP and Florida Pier as illustrated in Fig. 16. This discrepancy was not apparent during analysis of the single pile test.

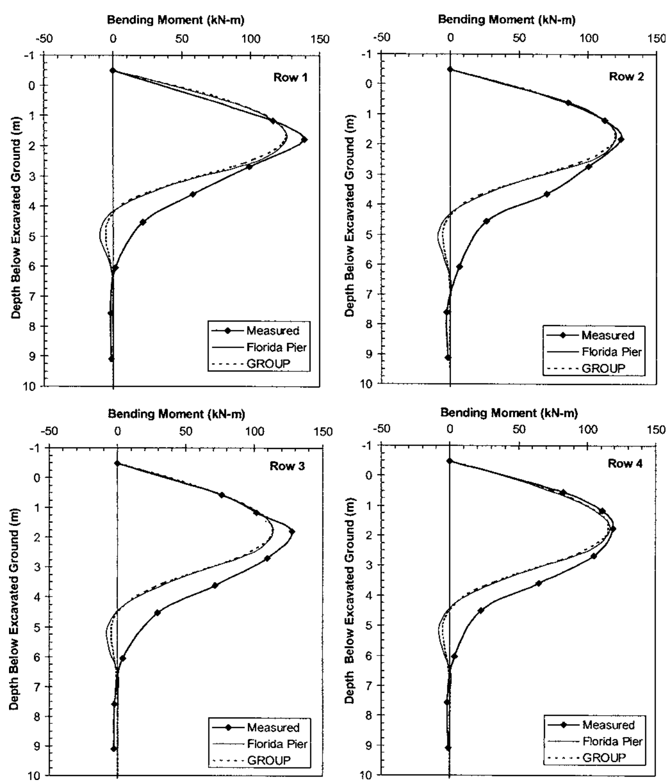

Fig. 16. Measured bending moment versus depth curves for each row of 3×4 pile group at deflection of 25 mm in comparison to curves computed using GROUP and Florida Pier with p multipliers developed during this study

Example Calculations

The total lateral load resistance of a group of 12 piles is to be determined. The piles are arranged in four rows of three piles each as shown in Fig. 18 with a spacing of 1,143 mm center to center in the direction of loading. Each pile is a 324 mm outside diameter steel pipe pile. Therefore, the S/D ratio is 1,143/324 or 3.53. The p-multiplier values for this spacing were determined using Eqs. (1)–(3) and the results are shown below

first (lead) row piles

$$f_m = 0.26 \ln(3.53) + 0.5 = 0.83$$

second row piles

$$f_m = 0.52 \ln(3.53) = 0.66$$

third and higher row piles

$$f_m = 0.60 \ln(3.53) - 0.25 = 0.51$$

Lateral load analyses for the pile groups can be performed directly using computer programs such as GROUP or FLPIER with the calculated p multipliers for each row. In the absence of these programs, analyses can also be performed using the computer program LPILE (Reese and Wang 1997) with these three f_m values to account for group effects as described below. The computed load vs. deflection curves for a single pile with f_m values of 1.0, 0.83, 0.66, and 0.51 are shown in Fig. 19. As the f_m value decreases, the computed deflection increases for a given load. To obtain the total load–deflection curve for the group, the resistance for each pile is summed at a given displacement using the appropriate single pile load–deflection curve in Fig. 19. An example calculation of the total group load for a deflection of 75 mm is shown below.

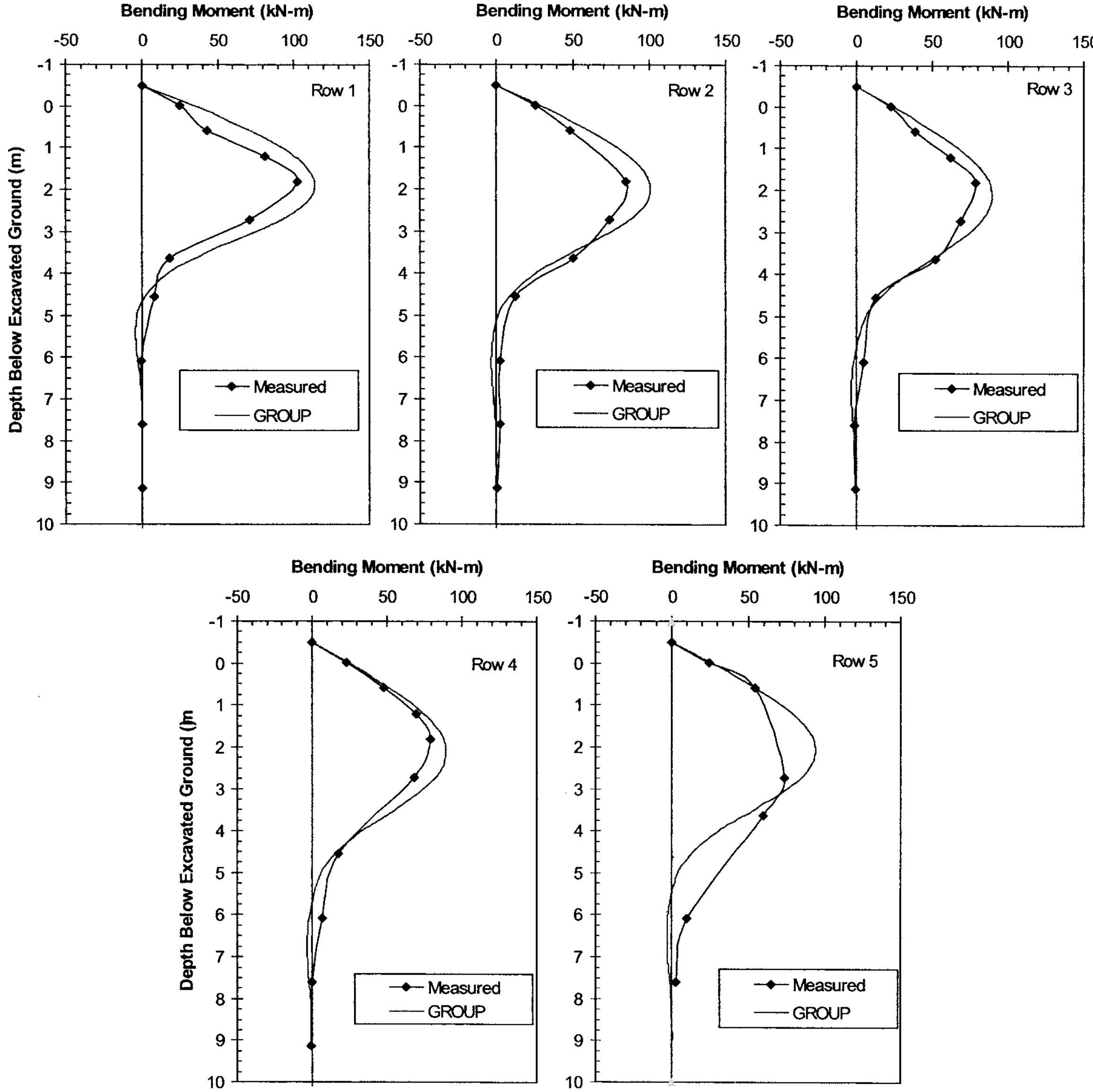

Fig. 17. Measured bending moment versus depth curves for each row of 3×5 pile group at deflection of 26 mm in comparison to curves computed using GROUP with *p* multipliers developed during this study

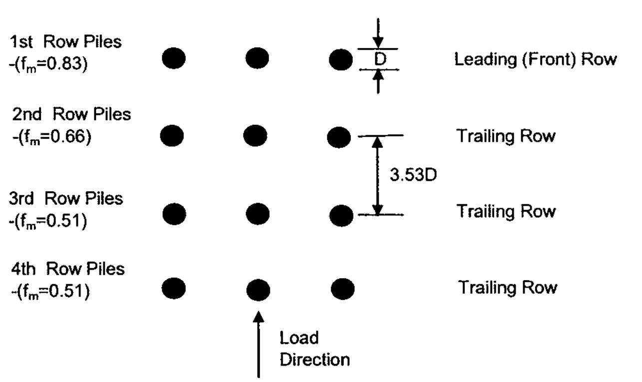

Fig. 18. Load direction, layout of piles, and appropriate *p* multipliers for each row in example pile group

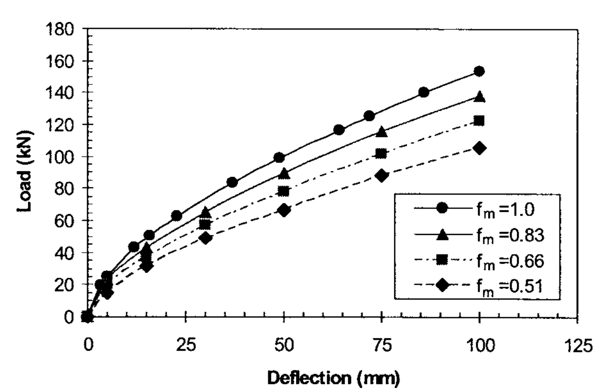

Fig. 19. Load–deflection curves computed using LPILE for each row in example pile group for f_m values of 1.0, 0.81, 0.66, and 0.51

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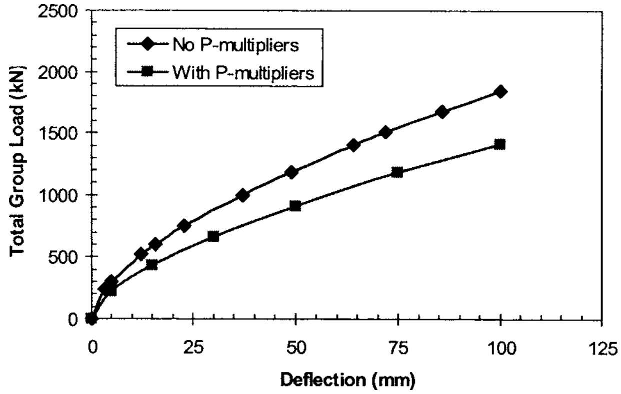

Fig. 20. Total computed load–deflection curves for example pile group with and without *p* multipliers

Example Calculation of Total Group Load at 75 mm Displacement

$$\text{1st (front) row pile load} = 116 \text{ kN}$$

$$\text{2nd row pile load} = 101.5 \text{ kN}$$

$$\text{3rd and 4th row pile load} = 88 \text{ kN}$$

$$\text{total load} = 3 \text{ piles} \times 116 \text{ kN} + 3 \text{ piles} \times 101.5 \text{ kN} + 6 \text{ piles} \times 88 \text{ kN} = 1{,}180.5 \text{ kN}$$

The total group load versus deflection curve computed using LPILE with consideration of appropriate *p* multipliers is shown in Fig. 20 along with a curve assuming no group interaction. In this case, failure to account for group interaction effects would lead to a 30% overestimation of lateral resistance.

The maximum bending moment versus load or bending moment versus depth curves can also be determined for piles in the group using the appropriate *p* multipliers. In general, the worst case curves should be used for all piles since the load direction may reverse, changing 1st row piles into 4th row piles, etc.

Conclusions

1. Back-calculated *p* multipliers based on the test results increased as the pile spacing increased from 3.3 to 5.65 diameters. Extrapolation of the test results suggests that group reduction effects can be neglected for spacings greater than about 6.5 for leading row piles and 7–8 diameters for trailing row piles;
2. Recommendations for *p* multipliers provided by Reese et al. (1996) and Reese and Van Impe (2001) overestimate the lateral resistance for closely spaced pile groups and could lead to somewhat unconservative results, while *p* multipliers recommended by AASHTO (2000), the US Army (1993), and the US Navy (1982) significantly underestimate lateral resistance and could lead to extra foundation costs for foundations in clay;
3. Based on analysis of the full-scale test results, improved design curves have been developed for three general cases: (1) first or front row piles; (2) second row piles; and (3) third and higher row piles; and
4. The response (load versus deflection, maximum moment versus load, and bending moment versus depth) for each row of the pile groups computed using GROUP (Reese et al. 1996) and Florida Pier (Hoit et al. 1997) generally correlated very well with measurements from the full-scale tests when the *p* multipliers developed in this test program were employed. However, both programs tended to underestimate the measured bending moment at depths below the maximum value.

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