Research Report
Agreement T2696, Task 02
Micropiles

FHWA SUPPORTED STRUCTURES RESEARCH SEISMIC BEHAVIOR OF MICROPILES

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Prepared for

Washington State Transportation Commission
Department of Transportation
and in cooperation with
U.S. Department of Transportation
Federal Highway Administration

January 2005

1. REPORT NO.	2. GOVERNMENT ACCESSION NO.	3. RECIPIENT'S CATALOG NO.
WA-RD 604-1		

4. TITLE AND SUBTITLE	5. REPORT DATE
SEISMIC BEHAVIOR OF MICROPILES	January 2005
	6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S)	8. PERFORMING ORGANIZATION REPORT NO.
Adrian Rodriguez-Marek and Balasingam Muhunthan	
9. PERFORMING ORGANIZATION NAME AND ADDRESS	10. WORK UNIT NO.
Washington State Transportation Center (TRAC) Civil and Environmental Engineering; Sloan Hall, Room 101 Washington State University Pullman, Washington 99164-2910	11. CONTRACT OR GRANT NO. T2696-02
12. SPONSORING AGENCY NAME AND ADDRESS	13. TYPE OF REPORT AND PERIOD COVERED
Research Office Washington State Department of Transportation Transportation Building, MS 7370 Olympia, Washington 98504-7370	Final Research Report
	14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

This study was conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration

16. ABSTRACT

Micropiles are grouted and small diameter piles that are traditionally used in foundation retrofit. Experimental evidence has indicated that micropiles behave well under seismic loading due to their high flexibility. Moreover, observations in the 1995 Kobe Earthquake indicate a good performance of friction piles under seismic loading. However, the seismic behavior of micropiles is not fully understood due to the limited number of full- and model-scale tests, as well as the limited amount of numerical modeling studies for micropiles.

This project focuses on Finite Element modeling (FEM) of single micropile and micropile groups under both static and dynamic loading. Initially, dynamic FE soil models were developed to conduct site response analyses. The lateral vertical boundaries of the soil were set up in such a way that the reflection of the arrival waves at the boundaries was avoided. The results of the site response analyses were verified against the well-validated code, SHAKE.

Subsequently, FE models for micropiles were developed with two constitutive soil models, i.e. a linear elastic and a bounding surface plasticity model. The micropile/soil interface was modeled either with perfect bonding or with frictional interface elements. For dynamic loading cases, a SDOF (single degree-of-freedom) superstructure was placed on top of the micropiles. Parametric studies were performed for various independent variables including load intensity, non-linearity of soil, and soil stiffness for the static case; and soil non-linearity, input motion intensity, frequency contents of input motion, and the natural period of the superstructure for the dynamic case. The static and dynamic behavior of micropiles was studied via the effects of aforementioned independent variables on the deflections and bending moments along the micropile length.

17. KEY WORDS	18. DISTRIBUTION STATEMENT
Micropiles, soil-structure interaction, site response, non-linear soil behavior, finite elements	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22616

19. SECURITY CLASSIF. (of this report)	20. SECURITY CLASSIF. (of this page)	21. NO. OF PAGES	22. PRICE
None	None	50	

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The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Washington State Transportation Commission, Department of Transportation, or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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Micropiles are grouted and small diameter piles that are traditionally used in foundation retrofit. Experimental evidence has indicated that micropiles behave well under seismic loading due to their high flexibility. However, the seismic behavior of micropiles is not fully understood due to the limited number of full- and model-scale tests, as well as the limited amount of numerical modeling studies for micropiles.

This project focuses on Finite Element modeling (FEM) of single micropile and micropile groups under both static and dynamic loading. Two constitutive soil models (a linear elastic and a bounding surface plasticity model) were used to represent a wide range of soil behavior. The micropile/soil interface was modeled either with perfect bonding or with frictional interface elements. For dynamic loading cases, a SDOF (single degree-of-freedom) superstructure was placed on top of the micropiles. Parametric studies were performed for various independent variables including soil non-linearity, input motion intensity, and the frequency content of input motion. The FE element models were used to obtain prescriptive *p-y* curves for use in simpler design-type software.

Several observations on micropile behavior were gleamed from an exhaustive parametric study. The use of interface elements that capture soil-pile friction and separation (gapping) is important to capture adequately soil-structure interaction. Gapping results in an increase in pile deflection. For a linear elastic soil, the increase in deflection due to gapping is linearly related to the applied horizontal load. This implies that the gapping elements do not introduce non-linearity in the pile-soil systems. The increase in deflection when gapping elements are used compared to deflections in a system with perfect bonding between soil and pile is significant. Most of the deformation occurs near the top of the micropile. Hence, it is important to incorporate interface elements between the micropile and the soil at least within six diameter lengths from the micropile head. Gapping also causes higher moments near the micropile head because a

lesser amount of load will be transferred to the neighboring soils. This, in turn, is due to the lower contact area between the pile and the soil.

An increase in soil's non-linearity causes an increase in deflection. Even though this conclusion is self-evident, it points to the importance of using appropriate nonlinear models of soil behavior. The mobilized pile moments in piles on inelastic soils are higher than those inserted in elastic soil. This occurs because of the lesser degree of load transfer from the pile to the soil in the more non-linear material.

The non-linear behavior of the soil has a significant influence on the response of the micropile to seismic excitation. Two extremes of nonlinear behavior were studied: a soil with a large elastic range and a soil with strong non-linear behavior (e.g., large damping values and strong modulus degradation at low strains). Both moment envelopes and pile deflections are influenced by soil nonlinearity.

The FE analyses were also applied to groups of vertical and inclined micropiles under dynamic loading. The inclination of micropiles provides larger lateral stiffness and results in smaller displacements and accelerations at the micropile head as compared to groups of vertical micropiles. The inclination of the micropiles does not affect the strain levels in the soil, implying that no additional stresses are being transmitted to the soil. The inclination of micropiles also decreases the bending moment at the micropile head. This is due to the fact that the axial capacity of inclined micropiles is also mobilized (in addition to their bending capacity).

The FE analyses were used successfully to obtain *p-y* curves that can be used in finite difference (e.g., LPILE and GROUP software families) analyses of piles. This implies that complex pile geometries and complex soil behavior can be simplified into *p-y* curves that can be used by the design professional.

CHAPTER 1
INTRODUCTION

INTRODUCTION AND PROBLEM STATEMENT

Micropiles are drilled and grouted small diameter replacement piles that are commonly used in foundation retrofit. Micropiles are reinforced and typically have diameters less than 300 mm. In most applications, micropiles behave as friction (e.g. floating) flexible piles. The advantages of using micropile systems include (a) their high flexibility during seismic conditions, (b) micropiles can be installed in low overhead clearance (less than 3.5 m), in all types of soils and ground conditions, (c) minimal disturbance is caused during construction, (d) inclined micropiles can be easily constructed, (e) they are able to resist axial and lateral loads, (f) only small volumes of earth to be excavated due to small diameter, (g) little disturbance is caused during drilling through an existing structure due to their small diameters, and (h) they can be drilled with boring machines that do not cause much noise. Despite the increased use of micropiles, the seismic behavior of a single micropile and a micropile group is not fully understood due to the limited number of full- and model-scale tests, as well as the limited amount of numerical modeling studies of micropiles.

The Finite Element (FE) method provides a tool to understand the seismic behavior of micropiles. FE analyses can be used to systematically alter the parameters that affect the seismic response of micropiles. However, the dynamic analysis of soil-micropile-structure interaction is a very complex problem. The problem includes soil non-linearity (e.g. variation of soil shear modulus and damping with strains), gapping and slippage between the micropile and the soil, complex boundary conditions (especially at the vertical lateral boundaries), and possible pile non-linearity.

The research presented herein was conducted as a partial requirement for the master's degree of Mr. Joo Chai Wong (Wong 2004). His contribution to the research is hereby acknowledged.

OBJECTIVE AND SCOPE

The scope of the research project focuses on the FE modeling of micropiles. The objectives of the project are to study:

(a) the development of a dynamic FE model for site response analyses where the lateral boundaries and soil behavior are modeled appropriately,

(b) the static behavior of a single micropile,

(c) the seismic behavior of a single micropile,

(d) the seismic behavior of a micropile group which includes vertical and inclined micropiles, and

(e) the back calculation of p-y curves from the FE models.

Deviations from original proposal

The research reported herein covers the scope outlined in the original proposal with the following exceptions:

(a) Full-scale and physical model tests were not performed due to budget limitations. Industry partnerships were sought unsuccessfully. In lieu of testing, previously reported results were used in validation studies.

(b) The study of the static behavior of micropiles is continued in a current FHWA project (DTFH61-03-C-00104) under the direction of Dr. Muhunthan. Hence the partial results on the study of static behavior of micropiles are not reported herein

CHAPTER 2
REVIEW OF PREVIOUS WORK

INTRODUCTION

The response of a structure subjected to seismic or dynamic loading primarily depends on the characteristic of the site response, the external loading, the mechanical properties of the surrounding soils, and the structure itself. An extensive review of the current literature on site response and soil-structure interaction (SSI) was conducted. This reports presents only a summary of observations related to dynamic behavior of micropiles. For a more extensive review, including issues related to site response analyses, the reader is referred to Wong (2004). Before dwelling on these problems, post earthquake observations were reviewed. Past earthquakes have indicated contradictory observations of the influence of battered piles on the response of a structure.

POST EARTHQUAKE OBSERVATIONS

The October 17, 1989 Loma Prieta earthquake (moment magnitude, M_w, of 7.1) yielded important observations on pile performance. SEAOC (1991) reports that the 7th Street Terminal Complex suffered extensive damage as the 16 in. square pre-stressed concrete battered piles supporting the Public Container Wharf failed in tension at their connection to the deck. Similar damage was observed at the Matson Terminal Wharf on 7 Street with additional damage to the back row of the vertical piles. Failure of the 16 in. square pre-stressed concrete battered piles at or near the pile cap connection was noticed at the Oakland Outer Harbor Pier 7. In San Francisco, the Ferry Plaza Pier suffered tensile failure at the connection of the deck to the pre-stressed concrete battered piles. These structural damages led several codes, such as the seismic Eurocode EC8 (1994) and the French Seismic Code (AFPS 1990), to discourage or avoid the use of battered piles in a seismic region. It is worth noting that in the abovementioned Loma Prieta earthquake observations, many failures occurred at the connection between the structure and the battered piles. These failures most probably were due to inadequate detailing at the connection and also improper connection of piles to pile caps (Mitchell et al. 1991). This implies that failures resulted not from the poor performance of battered piles, but from poor connection design. Later research such as that of Gazetas and Mylonakis (1998) presented theoretical and field evidence demonstrating that battered piles are of benefit rather than detrimental to pile-supported structures.

Field evidence from the 1995 M_w 6.9 Kobe earthquake reveals that one of the few quay-walls that survived in a Kobe harbor was a composite wall supported by battered piles, while nearby walls built on vertical piles suffered very severe damage. Berrill et al. (1997) investigated the near-failure response of the foundation of the Loading Road Bridge after the Edgecumbe earthquake (1997) in New Zealand. The foundation was embedded in liquefied sands. The authors state, “The motion towards the river was impeded by the buried raked-pile foundations which resisted the lateral spreading of the upper 6 m of soil toward the river channel.”

These post earthquake observations indicate that the seismic role of battered piles should receive much greater attention. These observations are relevant to micropile design because micropiles must be inclined to resist high seismic loads.

ANALYSIS METHODS

The response of pile-supported structures during dynamic loadings can be significantly influenced by the behavior of the interface between the structure and the foundation soil or so called soil-structure interaction (SSI). At these interfaces, the bonding is not perfect. In reality, relative motions, such as sliding and gapping, occur at the interfaces between the pile and the soil when the pile-supported structure system is subject to static and dynamic loadings. These relative motions plus the resulting mechanisms of load transfer from the structure to the soil and vice versa result in strong nonlinear SSI. Consequently, analytical closed-form solutions become very difficult and numerical techniques, such as the boundary element method, the finite difference method, the finite element method (FEM), and the Beam-on-Nonlinear-Winkler approach are used. Since this research project focuses on the FE modeling of micropiles, the literature review concentrates on the FE modeling for laterally loaded piles and micropiles.

The FEM provides a rigorous and flexible approach for modeling SSI problems. It can model almost any geometric configuration, soil and pile materials, load application, and boundary conditions. In addition, the soil continuity and the soil nonlinearity can be taken into account using FEM. However, the accuracy of the FEM results primarily depends on both the accuracy of the constitutive models and the use of appropriate input soil properties. Another drawback is the long computation time, especially for a three-dimensional (3-D) model. The literature on FE models of Soil-Pile-Structure interaction is large. In this work only research directly relevant to micropiles is presented. For a more complete review refer to Wong (2004). The observations and conclusions of the numerical studies are presented in subsequent sections.

FE analysis of micropiles

Kishishita et al. (2000) performed a 2-D FEM analysis of micropiles subject to earthquake input motions. The soil was modeled with linear and nonlinear analyses. In the linear analysis, three soil models with different shear wave velocities were used in the upper layer. Four different types of piles were used in each of these linear soil models, such as precast piles, cast-in-situ piles, high-capacity micropiles, and high-capacity raking micropiles. Two earthquake input motions were used in the analyses, the 1940 El Centro Earthquake and the 1995 Kobe Earthquake. In the nonlinear analysis, only the soil model with the lowest shear wave velocity (the softest soil) was used. But, the nonlinear analysis was still conducted with the aforementioned four types of piles used in the linear case. A modified Ramberg-Osgood model was used for the soil, a tri-linear model for the cast-in-situ piles, a modified Takeda model for the pre-cast piles, and a bilinear model for high-capacity micropiles.

Shahrour et al. (2001) conducted a 3-D FEM analysis of micropiles using a finite element program, PECPLAS. A single micropile and a micropile group supporting a superstructure were simulated in the analyses. The micropile group includes 1 x 3 micropiles, 3 x 3 micropiles, and 3 x 5 micropiles. These micropiles were modeled as embedded in a homogeneous soil layer overlaying rigid bedrock. The soil-micropile-structure system was assumed to be elastic with Rayleigh material damping. The cross-section of the micropile was assumed to be square. The superstructure was modeled as a single degree-of-freedom system (e.g., a concentrated mass on a column). The base of the soil mass was assumed to be rigid. Periodic conditions were imposed at lateral boundaries for the displacement field. A harmonic acceleration was applied at the base of the soil mass with its frequency equal to the fundamental frequency of the soil.

Ousta and Shahrour (2001) performed similar analyses on saturated soils. The analyses were carried out using the (u-p) approximation for the fluid-soil coupling (Zienkiewicz et al. 1980) and a cyclic elastoplastic constitutive relation that was developed within the framework of the bounding surface concept for representing nonlinear soil behavior. Single micropile, 2 x 2 micropile group, and 3 x 3 micropile group were modeled in the analyses. The micropiles were assumed to be linear elastic. The base of the soil layer was assumed to be rigid and impervious. Water table was assumed to exist at the ground surface. Sadek and Shahrour (2003) used a similar model to investigate the influence of pile inclination on the seismic behavior of a micropile group. A 2 x 2 vertical micropile group and a 2 x 2 inclined micropile group with a 20° inclination to the vertical axis were used.

FIELD AND MODEL TESTS OF MICROPILES

This section reviews experimental work on micropiles in recent years. Conclusions from the study are summarized in the subsequent section.

Yamane et al. (2000) conducted lateral and vertical load tests on micropiles. The study was focused on the vertical behavior of micropiles. However, they performed lateral load tests on seven single micropiles to study the bending capacity. Five micropiles were composite micropiles, consisting of steel pipes, grout, and thread-lugged bars; another micropile is identical to the previous five but with coupling joints for the steel pipes. Another micropile is a plain steel pipe only.

Yang et al. (2000) carried out a series of shaking table tests to study the behavior of a single micropile under seismic loading. A hollow aluminium model micropile was inserted in a level dry sand deposit that was prepared in the laminar container bolted to the shake table. Sinusoidal vibrations were applied in the horizontal direction. Three SSI models were used to compute the pile response, i.e. a) the standard dynamic beam-on-Winkler-foundation model, b) the simplified beam-on-Winkler-foundation, and c) the ‘Pilate’ model.

Juran et al. (2001) performed a series of centrifuge tests on single micropile, micropile groups, and micropile network. Various micropile configurations, inclinations, number of micropiles, and loading levels were conducted. Finite difference programs, LPILE and GROUP, were used to simulate the representative centrifuge model tests. These tests were used to study the structure-soil-micropile behavior and also to investigate the response of the micropile systems subject to earthquake loading.

Geosystems, L.P. (2002) carried out lateral load tests on micropile groups and micropile networks at field to study their lateral performance. Different micropile numbers and configurations were installed and tested with different directions of lateral loading. Most of the micropiles installed were of the Ischebeck Titan type.

SUMMARY OF OBSERVATIONS

The observations based on the numerical and experimental work described in the previous paragraphs are summarized in this section.

Relative rigidity of pile and soil Ep/Es. The linear and nonlinear numerical analyses done by Kishishita (2000) show that the relative rigidity Ep/Es influenced the horizontal displacements of the top structure and micropile cap; Ep and Es are the

Young's modulus of pile and soil, respectively. The displacement increased when the relative rigidity Ep/Es increased (the soil becomes softer).

Pile inclination. The numerical results by Kishishita (2000), and the centrifuge tests by Juran et al. (2001) show that the horizontal displacement of the raked micropiles was smaller than that of the vertical micropiles. The results by Juran et al. (2001) reveal that when the inclination of the micropiles increased, the fundamental frequency of the micropile system increased. Sadek and Shahrour (2003) show that in a seismic analysis, when the inclination of the micropiles increased, the lateral stiffness, the bending moment, and the axial force increased, but, the shear force, and the lateral acceleration at the micropile cap and superstructure decreased.

Properties of the superstructure. Shahrour et al. (2001) state that the mass and the frequency of the superstructure affect the inertial interaction in SSI problems. Their results illustrate that in a single micropile analysis, as the mass of the superstructure increased, the lateral displacement, the bending moment, and the shearing force at the pile head increased. It was also observed that when the frequency of the superstructure became close to the loading frequency, the horizontal displacement of the superstructure, the bending moment and the shear force increased significantly. This observation shows the important role of the frequency of the superstructure in the design of micropile foundation systems.

Pile spacing. Shahrour et al. (2001) and Ousta and Shahrour (2001) show that the bending moment increased with increasing micropile spacing. This increase is attributed to frame action. However, Shahrour et al. (2001) show that the influence of the micropile spacing on the distribution of shearing forces is negligible.

Number of piles. Similar to the case of pile spacing, the results from the FE analyses by Shahrour et al. (2001) and Ousta and Shahrour (2001) show that when the number of piles increased, the bending moment decreased. However, unlike the case in pile spacing, the shear force increased with the increase in the number of piles.

Shaking intensity. The shake table test results by Yang et al. (2000) show that with weak base shaking (< 0.25g), the micropile follows the motion of the soil and the maximum bending moments occur near the sand surface. This indicates that inertial effect plays an important role in micropile bending during shaking.

However, during strong base shaking (0.25g), the micropile did not follow the motion of the soil and the effects of the nonlinear soil behavior clearly affected the micropile behavior. Moreover, under strong base shaking, the maximum bending moments occurred near the pile bottom, which indicated that the micropile bending was dominated by the deformation of surrounding soil and the inertial effect from the pile head could be ignored. Yang et al. (2000) also commented that the frequency domain method might not be suitable and a time history analysis is needed for strong shaking or high excitation frequencies.

Pile type. The numerical analyses by Kishishita (2000) reveal that the horizontal displacements of the top structure and micropile cap were not affected by the pile type. The horizontal response at these two places was almost the same even though four different pile types were used in his analyses, i.e. precast piles, cast-in-situ piles, high-capacity micropiles and raked high-capacity micropiles. He claims that this phenomenon occurs because the micropile cap follows the response of the soil.

Stress-strain behavior of pile. Usually the design of a conventional pile is controlled by the external (i.e. ground-related) carrying capacity. Meanwhile, the design of a micropile is normally governed by the internal design, i.e. the selection of pile components (Bruce and Juran 1997). Due to sophisticated micropile installation methods, high grout/ground bond capacities with relatively small cross-section can be achieved. This highlights the fact that micropiles are designed to transfer the load to the ground through skin friction only.

A trilinear model was used for cast-in-situ pile, a modified Takeda model for pre-cast pile and a bilinear model for high-capacity micropile in the numerical analyses performed by Kishishita (2000). The numerical results show that during a real earthquake (e.g. the Kobe Earthquake input), the high-capacity micropiles maintained linearity while the precast and cast-in-situ piles yielded. Therefore, high-capacity micropiles provide high ductility and resistance against earthquakes.

Group effect. The numerical analyses by Shahrour et al. (2001) and the centrifuge test data by Juran et al. (2001) illustrate that a positive group effect was observed in micropile group. The numerical results by Shahrour et al. (2001) show that the positive group effect was observed for the kinematic interaction because the maximum bending moment at the central part (around mid-height of micropile) decreased when the number of micropiles increased from 9 (3 X 3) to 15 (3 X 5). Meanwhile, the experimental data (in cohesionless soil) by Juran et al. (2001) illustrate the positive group effect for selected frequency of excitation (a=0.3g) which caused a reduction in bending moments and displacements of micropile groups with s/D=3 as compared to the data from s/D=5.

Load distribution in micropile group. Internal forces are influenced by the position of the micropile in a micropile group. In other words, seismic loading is not distributed equally in the micropile group. The experimental data of Juran et al. (2001) and the numerical analyses of Shahrour et al. (2001) clearly show that the loads taken by the corner micropiles are higher than the one taken by the center micropile.

Coupling joints. The field test results by Yamane et al. (2000) reveals that the micropile (steel pipes, grout, and thread-lugged bars) with coupling joints provided higher strength and stiffness as compared to the ones of an identical micropile without coupling joints.

Pile diameter. The full-scale test results by (Teerawut 2002) illustrate that the effect of the pile diameter on the stiffness of p-y curves for piles embedded in sands is affected by the relative density of the sand. In the case of dense weakly cemented sand, the effect of the pile diameter on the p-y curves was insignificant before the soil reaches its ultimate resistance. However, in the case of loose sand, the stiffness of the p-y curves increased with an increase in pile diameter. In other words, increasing the relative density will decrease the pile diameter effect on the p-y curves.

It was also observed that as the pile diameter increased, the natural frequency of the soil-pile system increased due to an increase in soil-pile system stiffness. Besides, as the pile diameter increased, the damping ratio increased due to the fact that the damping of the soil primarily comes from the radiation damping which is dependent on the contact area and the excitation frequency. The radiation damping increases with an increase in the contact area between the pile and soil, and also with an increase in the excitation frequency.

DESIGN GUIDELINES

Up to the date of preparation of this report, there are two complete design guideline documents on micropiles published in the United States. The first one was published by the U.S. Department of Commerce of National Technical Information Service in 1997. It has four volumes and the second volume (named "Drilled and Grouted Micropiles – State-of-Practice Review: Volume 2: Design") reviews the state-of-practice of micropile design. The second design guideline document (Micropile Design and Construction Guidelines: Implementation Manual) was published by the Federal Highway Administration (FHWA) in 2000. In this section, a very brief summary of the design guidelines of micropiles on the geotechnical aspects from these two documents will be presented below.

Drilled and Grouted Micropiles - State-of-Practice Review: Volume 2: Design

In this document, a new and rigorous classification criteria for micropiles was developed. The classification system is based on two criteria, (1) philosophy of behavior (design), and (2) method of grouting (construction). Using the first criteria, micropiles are classified into two types, i.e. CASE 1 and CASE 2 micropiles. CASE 1 micropiles refers to the micropile elements (single or group) that are loaded directly. The load is primarily resisted structurally by the steel reinforcement and geotechnically by the grout/ground bond zones of the individual piles. CASE 2 micropiles are the elements that circumscribe and internally reinforce the soil behaving like a reinforced soil composite (mass), as opposed to individual piles, to resist the applied loads. Thus, they are usually more lightly reinforced as compared to CASE 1 micropiles. Micropile usage in the United States is almost exclusively CASE 1, which are also the focus of this study.

To evaluate the geotechnical capacity of micropile subject to axial, lateral, or combined loading, appropriate determination of grout/ground interface parameters and the initial stress state in the ground after micropile installation (mainly because of the grouting pressure) are required. The geotechnical design guidelines for single micropile subject to axial loading are based on the criteria of ultimate load capacity and vertical displacement control. Similarly, ultimate load capacity and horizontal displacement control is used for micropiles subject to lateral loading. The design methods can be selected from (a) empirical methods for ultimate load prediction, (b) load-transfer interface models for vertical displacement estimation, and (c) site-specific loading tests.

For a single micropile design, there are no specific design codes for types A, B, C, and D micropiles in the United States (please refer to the document for the definition of these four types of micropiles). For type A micropiles, the design usually requires compliance with specifications that have been established for large-diameter drilled shafts (e.g. AASHTO 1992, Caltrans 1994). Meanwhile, the British Standard BS 8081 (1989), referring to the work of Littlejohn and Bruce (1977), and the French code (CCTG 1993), following the field correlations by Bustamante and Doix (1985), would apply to types B, C, and D micropiles.

Due to the absence of design codes relating to lateral performance of micropiles, the current design practice will usually require lateral load tests that follow the present codes for drilled shafts (e.g. UBC 1994, BCNYC 1991, AASHTO 1992). For preliminary design, the design codes, like API (1988), CCTG (1993), and Caltrans (1994), referring to research works by Matlock (1970) and Reese et al. (1994) will be followed.

Similarly, there are no design codes developed for micropile groups and networks in the United States. As in the case of single micropiles, the design criteria used for micropile groups and networks is the ultimate load capacity and the displacement control.

The ultimate load capacity and displacement are influenced by the pile spacing, soil and site conditions, and types of micropiles and pile cap. It is highlighted that the group efficiency factor is significantly dependent on the pile installation technique.

There is no good reference of design codes can be used for estimating the ultimate axial loading capacity of micropile groups and networks since the laboratory and full-scale test results from various investigators (Lizzi 1978, Plumelle 1984, Maleki 1995) exhibit contradictory group effects. However, one of the design codes “mentioned” in the report is AASHTO (1992), following Terzaghi and Peck (1948), and this method has been used for conventional piles. It estimates the axial group capacity as the lesser of (a) the sum of the ultimate capacities of the individual piles in the group, or (b) the axial load capacity for the block failure of the group (a rectangular block). The French CCTG (1993) recommendations can be adapted for a preliminary conservative calculation of the group efficiency factor as its suggested Converse-Labarre group efficiency equation gives conservative results.

To estimate micropile group vertical displacement, several approaches have been adopted:

(a) empirical correlations relating the vertical displacement of pile groups to that of a single pile (e.g. Skempton 1953, Vesic 1969, Meyerhof 1976, Fleming et al. 1985),

(b) continuum elastic methods using Mindlin’s equations (1936) (e.g. Butterfield and Banerjee 1971, Randolph and Wroth 1979, Poulos and Davis 1980, Yamashita et al. 1987),

(c) load-transfer models and hybrid methods (e.g. O’Neill et al. 1977, Chow 1986, Lee 1993, Maleki and Frank 1994), and

(d) a pure shear interface model assuming no radial movement developed by Randolph and Wroth (1979).

To estimate the ultimate lateral capacity of micropile groups, similar to the case of axial group capacity, one of the ways mentioned is the lesser of a) number of micropiles times the lateral load capacity of a single pile in the group, or b) lateral load capacity of an rectangular block containing the micropiles and the soils between them. To account for the group effect on the lateral load capacity and pile deflections, different design codes (e.g. AASHTO 1992, CCTG 1993, BOCA 1990) specify different minimum spacing between the piles. However, when the piles are close to each other, the interaction between them has to be considered. Group efficiency factors for side-by-side piles and line-by-line piles are discussed in the state of the practice report.

To estimate the lateral load-deflection of a pile group, one of the common approaches is the usage of p-y curves (e.g. Reese et al. 1994, Brown et al. 1987, Bogard and Matlock 1983). Reese et al. (1994) suggest that the most rational way of analyzing the lateral load-displacement response of pile groups is the use of p-y curves for a single pile modified with the use of "softening" factors to allow for group interaction effects.

There are no recommendations in design codes for design guidelines for micropile networks (Case 2 micropiles). The development of high capacity micropiles has rendered the reticulated micropile network concept (Lizzi 1982) less price competitive and hence its lack of use in the United States.

Micropile Design and Construction Guidelines: Implementation Manual

In this manual, the Service Load Design Method (SLD) and the Load Factor Design Method (LFD) are used for micropiles in accordance with the 1996 AASHTO Standard Specifications for Highway Bridges, 16th edition. Micropiles are usually assumed to transfer their load to the ground through grout-to-ground skin friction without the contribution from the end bearing, except when the micropile is embedded on rock. The dependence on skin friction is geotechnically equivalent in tension and compression. There are no step-by-step design procedures for micropiles outlined in this manual. However, the manual presents several geotechnical micropile design guidelines and considerations.

The guidelines include the estimation of load transfer (grout-to-ground bond) parameters for different soil layers, the determination of the micropile bond length to support the loading, and the evaluation of the group effect for axially loaded micropiles. The implementation manual emphasizes that the geotechnical load capacity of a micropile is highly sensitive to the processes used during pile construction, especially the techniques used for drilling the pile shafts, flushing the drill cuttings, and grouting the pile. Table 5-2 in this manual tabulates the estimated unit values for grout-to-ground bond nominal (ultimate) strengths for various installation methods and ground conditions. These values are estimated based on the experience of the local Contractors or Geotechnical Engineers. Based on the estimated grout-to-ground bond strength, the bond length is determined to support the structural loading. Usually, the group effects in micropiles are beneficial, especially in granular soils, due to the compaction of the soil from pressure grouting.

The geotechnical considerations include: (a) prediction of anticipated structural axial displacements, (b) long term ground creep displacement, (c) settlement of pile groups, (d) lateral load capacity, (e) lateral stability (buckling), and (f) downdrag and uplift considerations.

When the micropile designs require strict displacement criteria, it may be necessary to predict pile stiffness and deflection limits during design and confirm the predictions through field load tests. Large creep deformation can occur in fine-grained clayey soils. Therefore, extended load testing should be performed to verify performance within acceptable limits. Micropiles in a group can cause additional displacement due to the consolidation of the soil layer, especially the cohesive ones below the micropile group. This is because when a single pile transfers its load to the soil in the immediate vicinity of the pile, a pile group can distribute its load to the soil layer below the group. The behavior of a laterally loaded micropile depends on the properties of the micropile such as diameter, depth, bending stiffness, fixity conditions of the pile in the footing, and on the properties of the surrounding soils. Considerations must be made to the combined stresses due to the bending induced by the lateral displacement and axial loading. The lateral capacity can be increased by inclining the micropiles and installing an oversized upper casing. Buckling of micropiles is only of concern in soils with the poorest mechanical properties, like loose silts, peat, and soft unconsolidated clays. The micropiles that extends above the ground or those that are subject to scour should be checked for buckling reduction. The small surface area of a micropile reduces the ability of the settling or expansive soils to transfer loads to micropiles. However, the use of battered micropiles should be avoided in settling or expansive soils because the settlement or expansion will induce excessive lateral loading on the micropiles.

CHAPTER 3
RESEARCH APPROACH

OVERVIEW OF RESEARCH APPROACH

The finite element code ABAQUS is used as the basic framework for the analysis of the seismic behavior of micropiles. The research is divided into two parts. First, a parametric study was conducted on single micropile and group arrangements under static and dynamic loads. The objective of the parametric study was to evaluate the influence of various parameters on the behavior of micropiles. The second step was to utilize the finite element to back-calculate *p*-*y* curves for micropiles which can be used in design.

A bounding surface plasticity model was used to represent the nonlinear behavior of soils (Borja and Amies 1994, Borja et al. 1999). Details on the implementation of the model are reported in Wong (2004). The model accurately represents modulus reduction and the increase of damping with increasing shear strain. Boundary conditions are represented by transmitting boundaries (Lysmer and Kuhlmeyer 1969, see also Rodriguez-Marek 2000). The seismic loads are entered as inertial forces proportional to the acceleration time history. This has the same effect as entering the seismic loading through displacement time histories at the base of the model.

The finite element model was validated for various conditions including: pure site response (e.g. the response of a soil column without the presence of piles), the response of single piles under lateral load, and the response of micropile groups under static loading. The later validation was done by comparing predictions of the model with field tests reported in Geosystem (2002). Due to space limitations, the reader is referred to Wong (2004) for the validation exercise.

FINITE ELEMENT MODELS FOR SOIL-PILE-STRUCTURE INTERACTION

Several micropile configurations and loading conditions were considered in this study, including single micropile under static loadings, single micropile under dynamic loading, and micropile groups under static and dynamic loading. Parametric analyses of static loading for single micropiles were used to test the validity of the model, and are reported in Wong (2004). This report focuses on the dynamic behavior of single and group micropiles. The model used for static analysis of micropile groups is also presented to illustrate the 3-D capabilities of the finite element model.

FE Model for Single Micropile under Dynamic Loading

Figure 3.1 illustrates the FE model for a single micropile under dynamic loading. Transmitting boundary conditions were implemented at the base and at the sides to prevent wave reflection from the boundaries. Two-dimensional (2-D) FE models were used to reduce computational time. Anandarajah (2000) has shown that the results from his 2-D FE models of SSI problem agree well with centrifuge data. A superstructure system with a single DOF was built on top of the micropile. The superstructure system consists of a single mass being linked to the micropile top with a solid element. Interface elements between the micropile and the clay were initially incorporated; unfortunately, divergence in the numerical solution was encountered. Therefore, perfect bonding between the micropile and the clay was used instead. Additional studies are currently being performed to incorporate interface elements in the analysis.

Three different soil models were used for the clay, i.e. linear elastic model, plasticity model with strong non-linearity, and plasticity model with weak non-linearity. The difference in dynamic behavior of highly and mildly nonlinear soils can be seen in Figures 3.2. Observe that the highly nonlinear soil has a larger modulus reduction and larger damping ratio increase than the mildly nonlinear soil. Moreover, the highly-nonlinear soil also has a lower undrained strength (see Wong 2004). Linear elastic materials were used for the micropile and the superstructure system. A Ricker Wavelet was used as the input motion in place of a real earthquake motion in order to save computational time. Gazetas (2001) successfully used a similar input to study topographic amplification effects in the 1999 Athens Earthquake. The wavelet is defined by the following formula (Mavroeidis and Papageorgiou 2003):

$$a(t) = A\sum_{i=1}^{3}(1-2\pi^2 f_p^2 t^2)e^{-\left(\frac{\pi}{f_p}\right)^2 t^2} \tag{3.12}$$

where *a(t)* is the acceleration time history, *t* is the time, *A* is the maximum acceleration, and f_p is the prevailing frequency. Three different input motion intensities were used (A = 0.1 g, 0.3 g, and 0.5 g, respectively). Three prevailing frequencies were used to obtain a broadband motion ($1/f_{p1}$ = 0.1 s, $1/f_{p2}$ = 0.16 s, and $1/f_{p3}$ = 0.22 s). These frequencies were chosen to closely match the natural site period in order to study resonance effects due to site amplification. The displacement time histories obtained from double integration of the acceleration time history are shown in Figure 3.3a. The spectral accelerations of the input motions are shown in Figure 3.3b. The duration of the wavelet pulse is 0.6 s, but analyses were executed for a total duration of 4.0s.

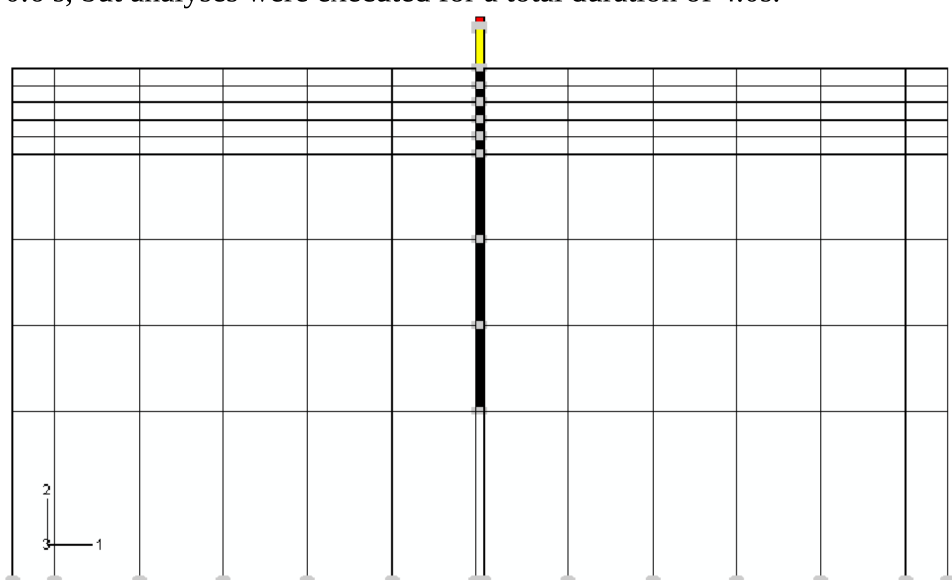


Figure 3.1 Two-dimensional FE model for single micropile analysis under dynamic loading

FE Models for Micropile Groups under Static Loading

Two FE models for micropile groups under static loading were developed. Both of them were constructed to perform a validation study against field test results reported by Geosystem, L.P. (2002). Therefore, the geometry and loading of the FE models were built as close as possible to the field load tests. One of them was a micropile group consisting of four vertical members whereas the other comprised four inclined micropiles raked at 25° to the vertical. Only half of the symmetrical geometry of the micropile group under load test was modeled in order to save computational time. Figure 3.4 illustrate the FE models of the vertical and the inclined micropiles.

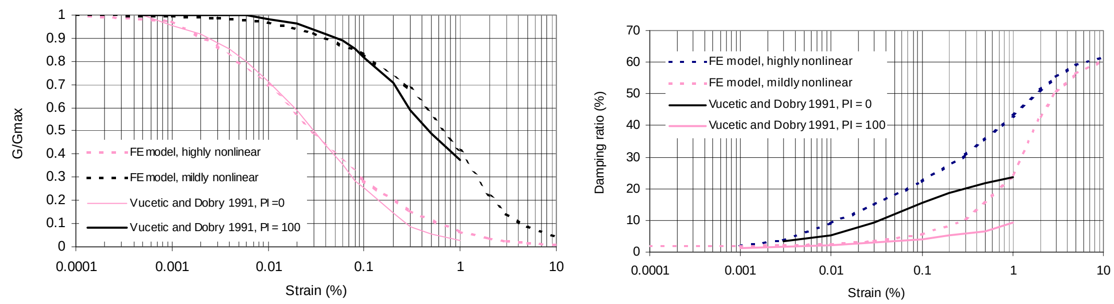


Figure 3.2 Modulus reduction and damping curves for soils used in this study. For comparison, the Vucetic and Dobry (1991) curves for PI=0 and PI=100 are also shown

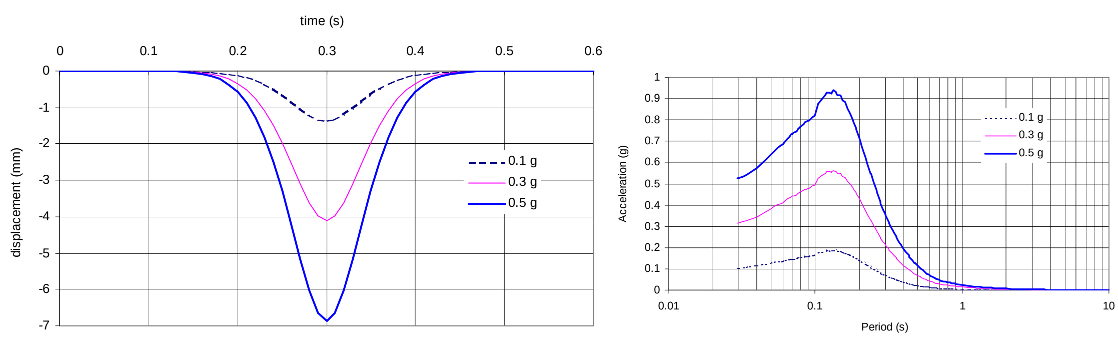


Figure 3.3 Wavelet with various intensities, a) Displacement time history, b) Acceleration response spectra

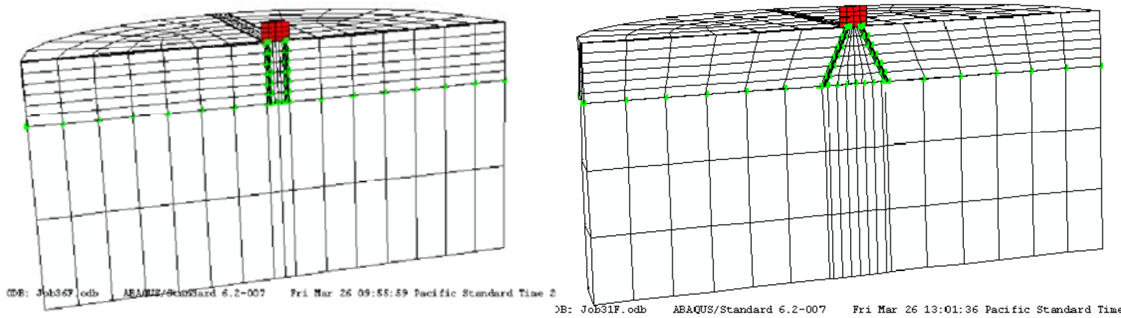


Figure 3.4 FE model for micropile groups under static loading

The pile cap was 3 ft in diameter and 2 ft in height. The micropiles were roughly 6.5 ft in length below the bottom of the pile cap. A horizontal load was applied at approximately 6 in above the bottom of the pile cap. The stiffness information for the soils at the field was not available. Thus, the Young's modulus of the soil, E, was estimated from the input data for GROUP analyses done by Weinstein (2003), a member of a group in-charged of the field load tests. A k value of 100 lbf/in^3 was used for the entire soil layer in the GROUP analyses where $E_{py} = k\,x$; E_{py} is the secant modulus of the p-y curve (p = soil reaction per unit length, and y = lateral deflection of the pile at a point x along the pile length), k is a constant, and x is the depth of the pile below the pile head. Terzaghi (1943) approximated the relationship between E_{py} and E in sand as shown below:

$$E_{py} = \frac{E}{1.35} \tag{3.13}$$

A constant E_{py} was assumed for the entire soil layer and approximated as the value of E_{py} at the mid-depth of the micropile length. Thus, an approximated E of 5265 lbf/in^2 was used in the FE models.

Perfect bonding was used for the interaction between the micropiles and the soil. Interface elements with gapping and sliding were not used because divergence in the solution was encountered.

FE Models for Micropile Groups under Dynamic Loading

Two FE models were constructed for micropile groups under dynamic loading. The first one is a micropile group consisting of two vertical micropiles as shown in Figure 3.5a. A pile cap was built on top of the two micropiles and a superstructure system was constructed on top of the cap. This superstructure system was similar to the one used in the FE model for single micropile under dynamic loading. Meanwhile, Figure 3.5b illustrates the second FE model consisting of two micropiles inclined at 20° to the vertical. The pile cap and the superstructure system were constructed in a similar manner.

For these two micropile groups, a similar input motion with the same intensities as the one used in the single micropile under dynamic loading was applied at the base of the clay. The frequency content of the input motion was varied to evaluate the effect on the response of micropile groups.

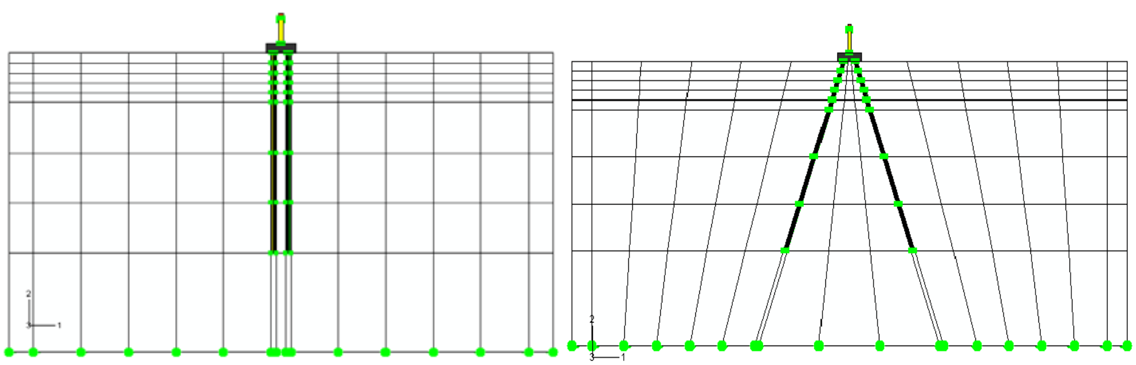


Figure 3.5 Two-dimensional FE model for two vertical micropiles under dynamic loading

CHAPTER 4
RESULTS AND DISCUSSION

PARAMETRIC STUDY

A parametric study was conducted in order to develop a better understanding of the seismic behavior of micropiles. The parametric study on a single micropile and micropile groups under dynamic loading is presented herein. Similar results for static lateral loading are presented in Wong (2004).

Single Micropile : Dynamic Loading

The parametric study conducted for single micropile under dynamic loading is presented in this section. The independent variables include the nonlinear stress-strain behavior of the soil and intensity of input motion. The dynamic behavior was studied via the dependent variables of deflection and moment along the pile.

Non-linearity of Soil

Figure 4.1 presents the time history of the deflections at the micropile head from the FE models with the linear elastic, mild nonlinear and highly nonlinear soils for an input motion of 0.3 g intensity (see Chapter 3). The effect of soil damping is clearly seen as the motion for soils with increasing damping (e.g., elastic, mildly nonlinear, and highly nonlinear, respectively) show both more attenuation with time, and a delayed phase arrival that also increases with damping.

Alternatively, the micropile head responses are presented in terms of acceleration response spectra (Figure 4.2). Distinctly different behavior is observed for the elastic and mildly nonlinear soil than for the highly nonlinear soil. Spectral peaks for the first two soils occur at the same spectral period (i.e. 0.18 s) which indicated no significant reduction in shear modulus in the soil or the soil behaved nearly elastically for this input motion level. The smaller spectral acceleration for the mildly nonlinear soil result mainly from the hysteretic damping of the inelastic material. Meanwhile, the highly nonlinear material shows a strong shift of peak spectral acceleration to longer periods. The shift of the peak to the right was attributed to the reduction of shear velocity due to a decrease in shear modulus. Meanwhile, the much lower peak from the material with strong non-linearity as compared to the one with weak non-linearity was resulted from the higher hysteretic damping in the highly nonlinear soil at a given strain (see Figure 3.2). It is worth repeating that the two nonlinear soils were selected to illustrate extremes in soil behavior, and do not necessarily represent realistic material properties. Furthermore, the soil with high nonlinearity has a low undrained strength.

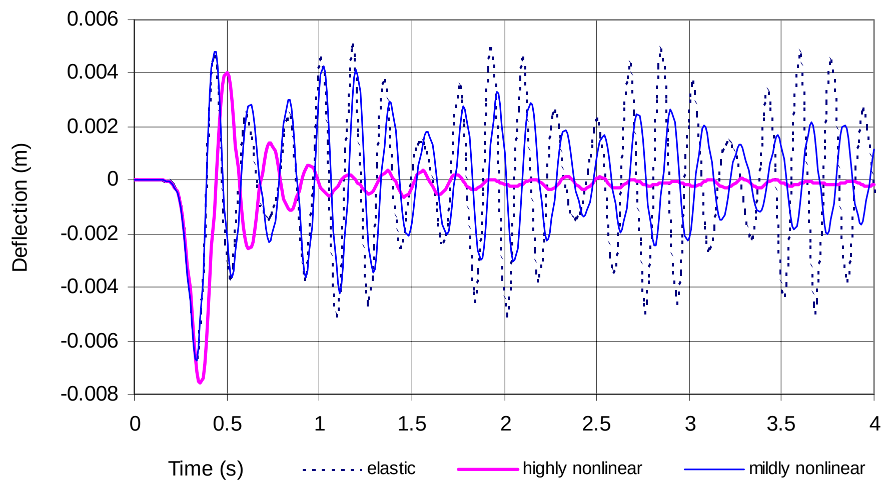


Figure 4.1 Time history of deflections at micropile head from various soil models

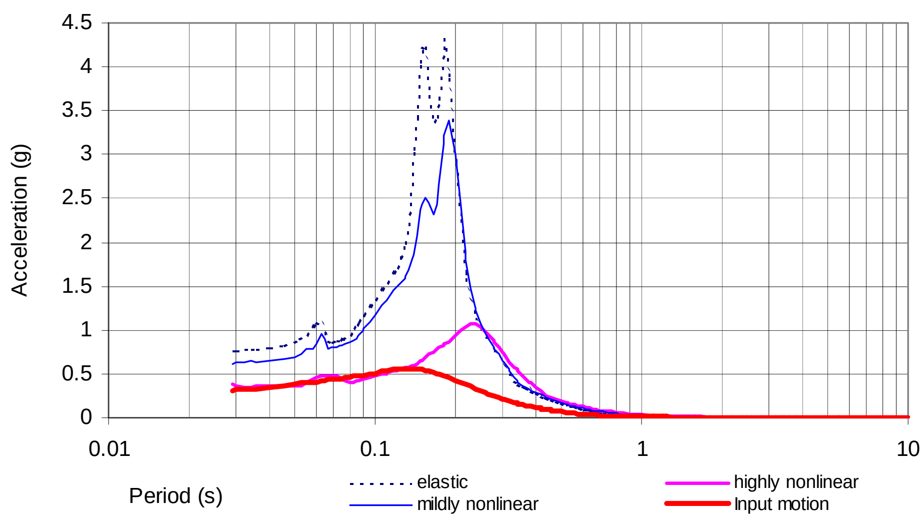


Figure 4.2 Acceleration response spectra from elastic and inelastic materials, and of input motion

Figure 4.3 presents the envelope of the bending moments along the micropile length for the 0.3 g input motion for both the elastic, mildly nonlinear and highly nonlinear materials. In all cases, the maximum bending moment happened at the micropile head due to the fixed head condition. The moment envelope from the inelastic material with weak non-linearity was smaller than the one from the elastic material. As mentioned in the above, this inelastic material behaved elastically with this input motion. Therefore, the smaller moment envelope was attributed to its hysteretic damping. Meanwhile, the much smaller moment envelope from the inelastic material with strong non-linearity as compared to the one with weak non-linearity resulted from the lower spectral accelerations transmitted through the highly nonlinear soil (Figure 4.2).

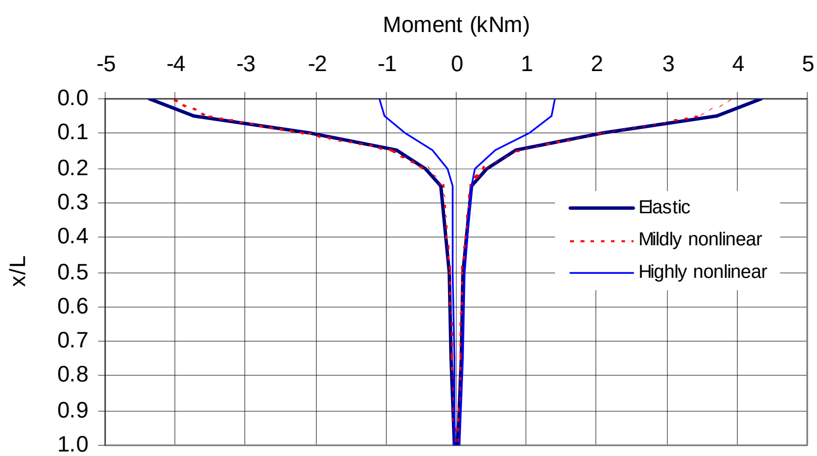


Figure 4.3 Bending moment envelopes from elastic and inelastic materials with 0.3 g input motion. Initial motion produces positive moment

Intensity of input motion

The bending moment envelopes at three different input motion intensities from the models with mildly and strongly nonlinear materials are presented in Figures 4.4 and 4.5, respectively. As expected, the bending moment envelope increases with increasing intensity. Observe that the moment envelope for the mildly nonlinear soil is nearly symmetric (Figure 4.4). On the other hand, the asymmetry of the highly nonlinear soil increases with increasing input motion intensity (Figure 4.5). This implies that soil yielding significantly dissipates the energy of the waves traveling through the pile. This observation is of significant for the design of piles in soft soils for earthquakes that generate large number of strong motion cycles.

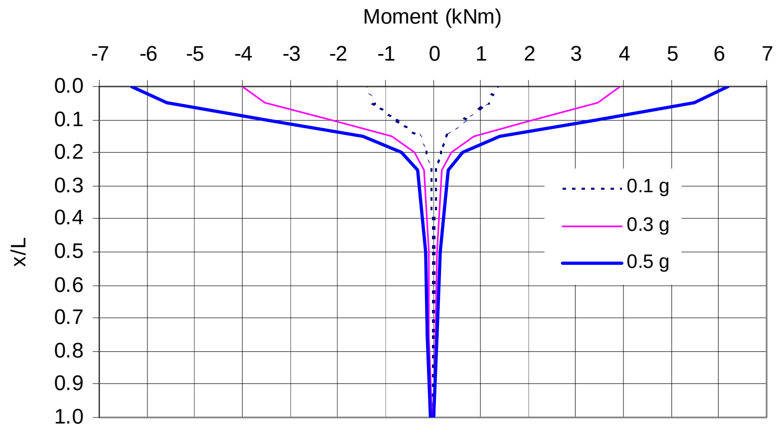


Figure 4.4 Bending moment envelope in inelastic soil with weak non-linearity at various input motion intensities

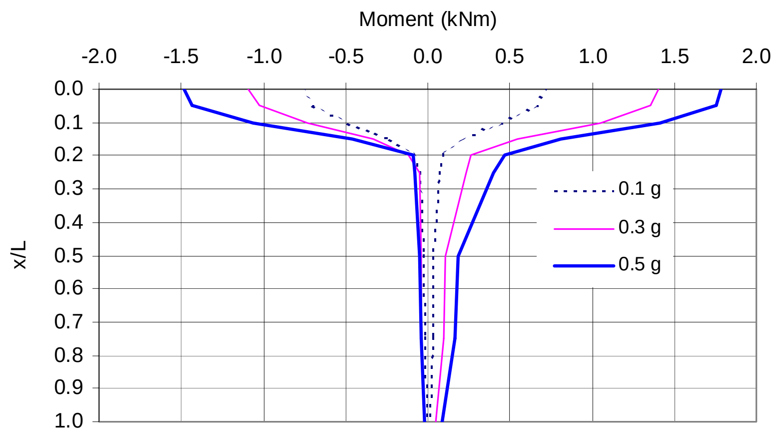


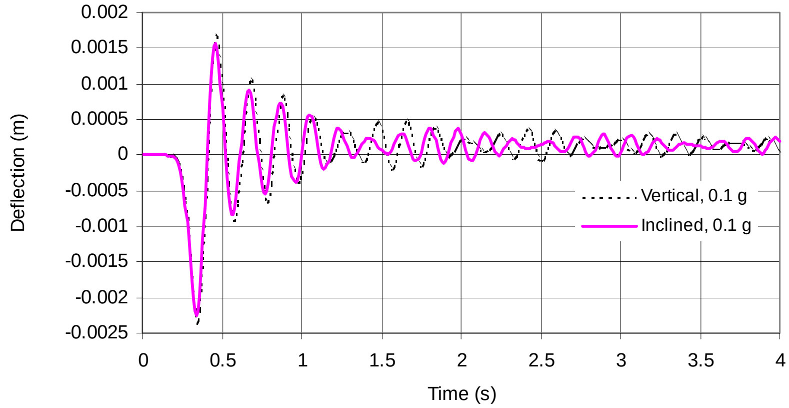
Figure 4.5 Bending moment envelope in inelastic soil with strong non-linearity at various input motion intensities. Initial motion produces positive moment

Micropile Groups : Dynamic Loading

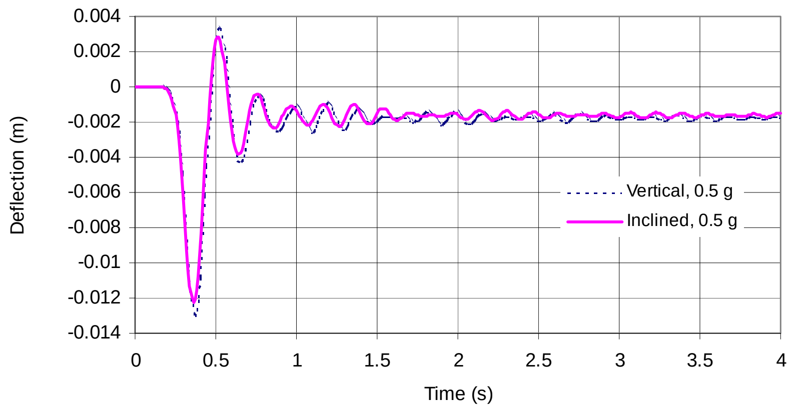
The parametric study conducted for micropile groups under dynamic loading includes the study of the effect of variations in the intensity of input motion, inclination of micropiles, and the frequency content of the input motion. In this section, the study was conducted only on the inelastic material with strong non-linearity.

Input Motion Intensity

Figures 4.6 (a) and (b) present the time history of deflection at the micropile head of the vertical and inclined micropile groups at the input motion levels of 0.1 g and 0.5 g, respectively. Generally, the maximum deflection from both the vertical and inclined micropile groups increase with input motion intensity. It was also observed that for both input motion levels, there was a residual displacement at the end of shaking. It appears that the residual displacement in the case with 0.5 g input motion, approximately 1.68 mm, was higher than that in the case with 0.1 g input motion, approximately 0.2 mm. Residual displacements imply plastic soil deformation.



(a) 0.1 g



(b) 0.5 g

Figure 4.6 Time history of deflections at micropile heads in both vertical and inclined micropiles at (a) 0.1 g, and (b) 0.5 g input motions

Figure 4.7 illustrates the acceleration response spectra of the micropile head in both vertical and inclined micropiles at acceleration levels of 0.1 g and 0.5 g. Observe that the arrangement of the micropiles does not change the frequency content of the micropile motions. This implies that kinematic effects are negligible, which is to be expected due to the relative flexibility of the micropile foundation.

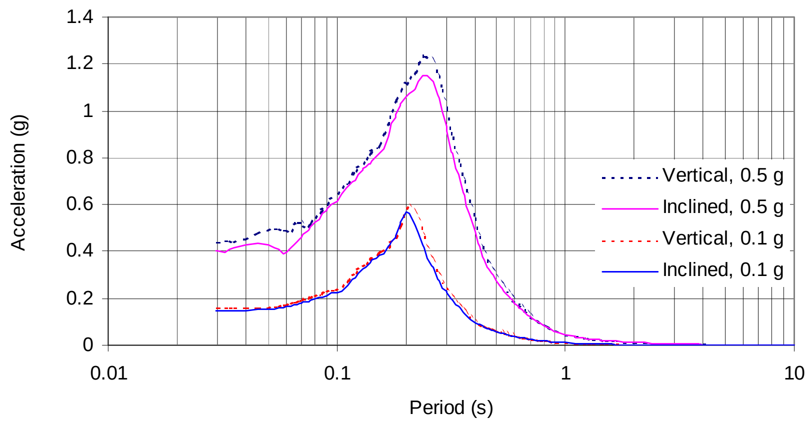


Figure 4.7 Acceleration response spectra of micropile head in vertical and inclined micropiles at 0.1 g and 0.5 g input motions

Figures 4.8 and 4.9 present the bending moment envelopes for vertical and inclined micropile groups, respectively, for varying input motion intensities. The bending moment envelope increases in both vertical and inclined micropiles with increasing intensity.

It was interesting to note that in the case of vertical micropile group, the moment envelope for the left and right vertical micropiles was similar at all input motion intensities. This indicates that there was an equal distribution of loading among the vertical micropile members under seismic loading. However, in the case of inclined micropiles, there was no equal distribution loading among the inclined micropiles. The left micropile appears to have carried higher loads. This indicates that the inclination of micropiles contributed to the unequal distribution of loads among the micropile group members.

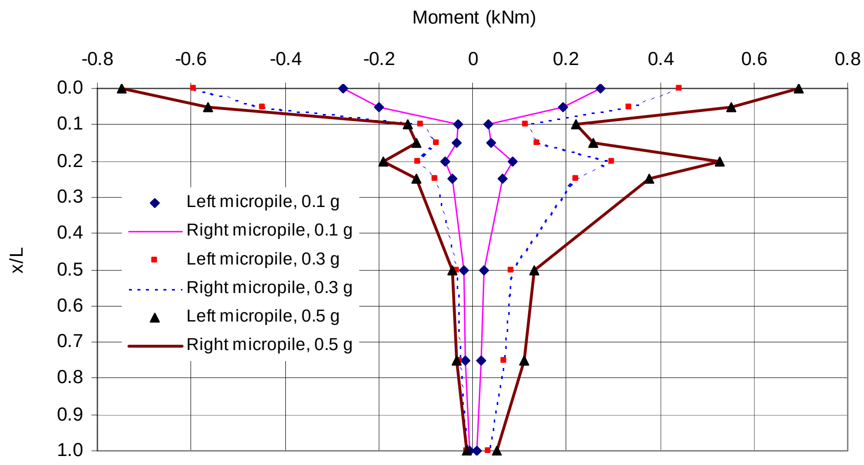


Figure 4.8 Bending moment envelope of left and right vertical micropiles at various intensities of input motion

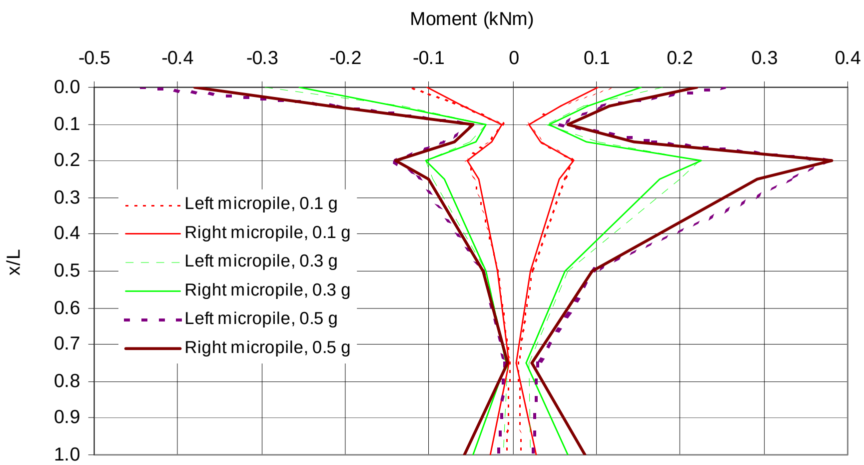


Figure 4.9 Bending moment envelope of left and right inclined micropiles at various intensities of input motion

Inclination

By referring to Figure 4.6, the maximum amplitude of deflection is slightly lower in the inclined micropile group than in the vertical micropile group. This illustrates the higher stiffness of the inclined micropile group. Moreover, the deflection response from the vertical micropiles lagged behind the one from the inclined micropiles, indicating larger damping, possibly due to differences in radiation damping.

Figure 4.10 presents the bending moment envelope of two micropile members in the vertical and inclined micropile groups at the input motion intensity of 0.5 g. The bending moment envelopes from the inclined micropiles were smaller than those from the vertical micropile because the axial capacity of the inclined micropiles was mobilized.

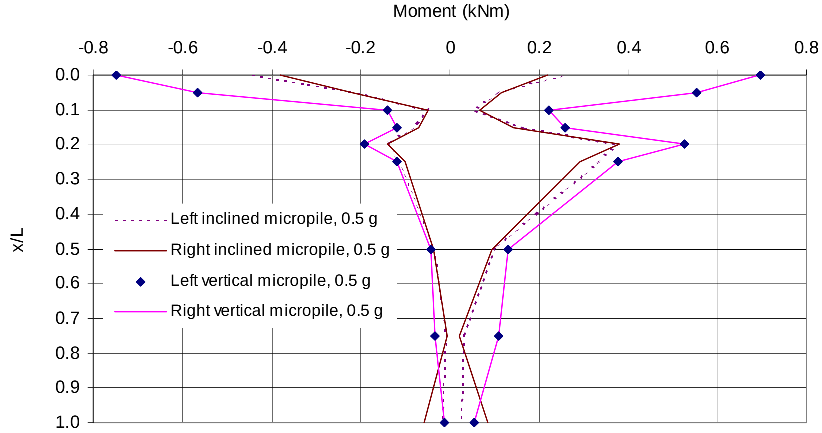


Figure 4.10 Bending moment envelope of vertical and inclined micropiles at 0.5 g input motion

Frequency Content of Input Motion

Figure 4.11 illustrates the bending moment envelopes of vertical micropiles at various frequency contents of input motion. The input motion with larger predominant period (*f2*) generally had a larger maximum moment than the one in the case with smaller predominant period (*f1*). This was attributed to the larger spectral accelerations due to site response effects for the input motion with a larger natural period. However, note that the opposite effect is observed at the micropile head. This illustrates the complexity of the soil-pile interaction problem.

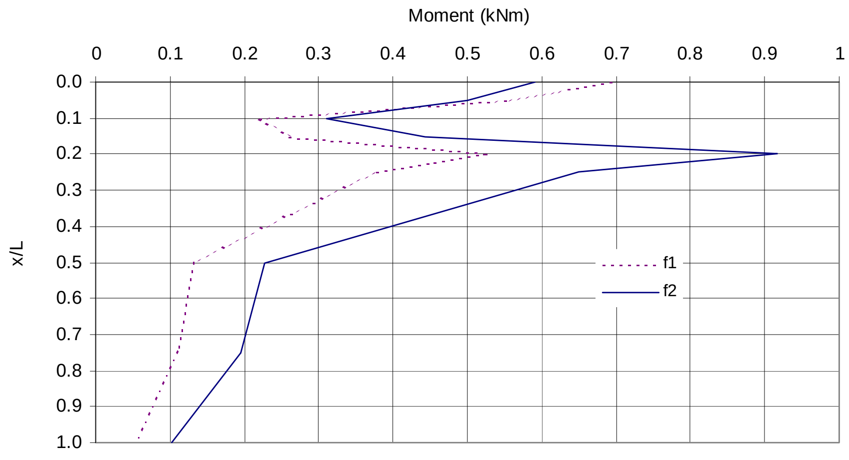


Figure 4.11 Bending moment envelopes of vertical micropile groups at different frequency contents of input motion

P-Y CURVES

The load transfer mechanism at the interface between the pile and the soil for a laterally loaded pile usually is represented by *p*-*y* curves. These *p*-*y* curves are commonly used in conjunction with computer programs, such as COM624, Florida Pier, and LPILE to estimate deflections and moment envelopes in the design of pile foundations. Therefore, *p*-*y* curves serve as a useful tool in the design of a laterally loaded pile. In this study, *p* is defined as the lateral soil resistance per unit length of the pile, and *y* is the lateral deflection. In this chapter, the back-calculation, validation, and behavior of *p*-*y* curves are presented.

Backcalculation of *p*-*y* curves

There are several ways to back-calculate *p*-*y* curves from FE analyses or full-scale load tests. One of the most commonly used methods is by making use of the bending moments along the pile. An analytical expression is fitted to the discrete moment data along the pile. Subsequently, the expression is differentiated twice to derive the soil resistance, *p*. Another method to obtain *p* is by summing the normal and shear stresses applied to the pile by the soil immediately surrounding it (Bransby, 1999). In this study, the former method was used.

Bending moment data were derived from the axial stresses in the micropile elements. These axial stresses were located at the two opposite nodes on the outermost diameter of the micropile at various depths. The bending moment *M* is thus defined as:

$$M = \frac{(\sigma_L - \sigma_R)}{d} I \tag{5.1}$$

where σ_L and σ_R are the axial stresses at the left and right outermost micropile diameter, respectively, *d* is the micropile diameter, and *I* is the moment inertia of the micropile.

Soil reaction, p was derived from the differential equation for a beam on a Winkler type of subgrade:

$$p = -\frac{d^2 M}{dx^2} \tag{5.2}$$

where x is the depth from ground surface. In this study, a 6^{th} degree polynomial was used to fit the moment data using least squares method to provide some degree of smoothing. The resulting polynomial should predict a shear force at the micropile head equal to the applied load, i.e.

$$V = \frac{dM}{dx}(x = 0) \tag{5.3}$$

In order to satisfy this additional constraint, a method suggested by Weaver (2004) was implemented. The method was to create an artificial moment data point above the micropile head and vary its value until the calculated shear force (Equation 5.3) equaled the applied horizontal force at the micropile head. The deflections y were obtained directly from the output of the FE analysis.

Validation of p-y curves

The *p-y* curves back-calculated from the FE models of a single micropile under static loading are validated herein. The validation was performed by using the *p-y* curves obtained from the FE analyses at various depths in a finite difference (FD) code, LPILE where the pile is treated as a beam-column and the soil is represented by non-linear Winkler-type springs.

Figure 4.12 shows the back-calculated *p-y* curves from a model with linear elastic soil properties. The *p-y* curves at various depths were used in the FD analysis with the exception of the *p-y* curve at the ground surface. This was done because in general the *p-y* curves at the ground surface obtained from this and other models were unreasonable.

Instead, the *p*-*y* curve obtained at the depth of 0.12 m was used for the depth of 0.00 m assuming that the springs at these two locations had the same properties.

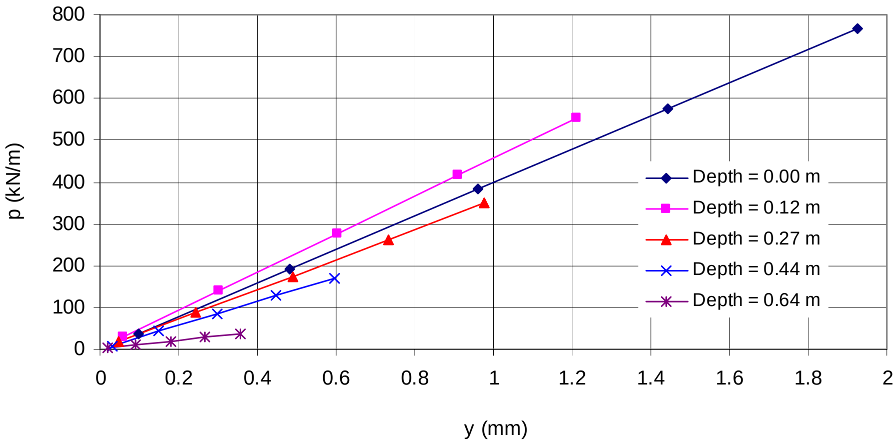


Figure 4.12 Load-transfer curves at various depths for a linear elastic soil

Figure 5.1 reveals that, with the expection of the curve at zero depth, *p*-*y* stiffness (modulus of subgrade reaction, in the units of force per area) reduces with depth even when the soil properties remain constant. A similar observation was reported by Bransby (1999).

Figure 4.13 presents the deflections and the bending moments along the micropile length under the load of 400 kN at the micropile head from the finite element (ABAQUS) and finite difference (LPILE) analyses. Except for the deflections at the depths of 0.05 to 0.40 micropile length, the results of deflections and moments from both analyses are similar. This indicates that the *p*-*y* curves obtained from the FE model are applicable in a FD code. The relevance of this is not to be underestimated. It implies that FE analysis can be use to render simple design curves (*p*-*y* curves) for complicated pile arrangements.

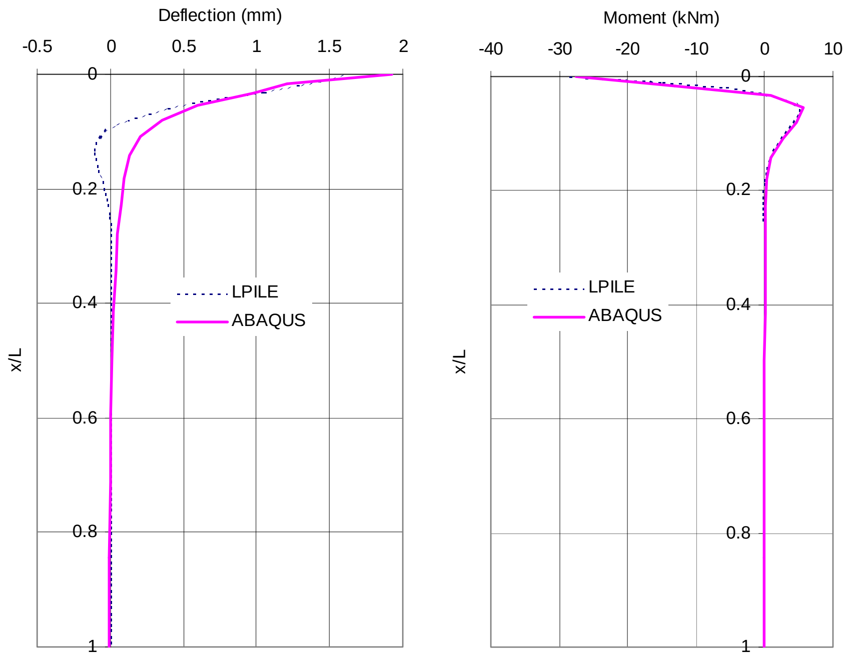


Figure 4.13 Deflection and bending moment profiles under the load of 400 kN at the micropile head from LPILE and ABAQUS

Figure 4.14 shows the derived *p*-*y* curves at various depths for a model with a linear elastic soil with interface elements that model soil-pile friction and soil-pile separaton (gapping). It is interesting to observe that the *p*-*y* stiffness generally increased with depth (except the one at the 0.64 m depth). This is opposite to the trend found in the linear elastic soil. The deflections and the bending moments along the micropile length under a load of 400 kN at the micropile head from FE and FD analyses are plotted in Figure 4.15. Again, the results from both analyses agree well with each other, thus validating the use of FE derived *p*-*y* curves in FD analyses.

The *p*-*y* curves obtained at several depths from the model with a highly non-linear soil with interface elements are presented in Figure 4.16. The values of the back-calculated *p* at the depths of 0.00 m and 0.12 m were negative. These values were considered erroneous since there were very unlikely to have a tensile reaction (negative *p*) in a soil at the front of the micropile when the soil was displaced (positive *y*). Therefore, the *p*-*y* curve obtained at the depth of 0.27 m was used at the depth of 0.00 m as well. The corresponding deflection and the bending moment profiles along the micropile length from FE and FD analyses are compared in Figure 4.17. The results from both analyses did not agree well with each other. Recall that this soil has a low undrained strength. This warns us about the reliability of using the *p-y* curves back-calculated from FE analyses when the soil has potential to have large plastic strains. The discrepancies between the FD and FE codes are due to the limitations of the Winkler model in representing the continuum model when the soil can reach shear failure.

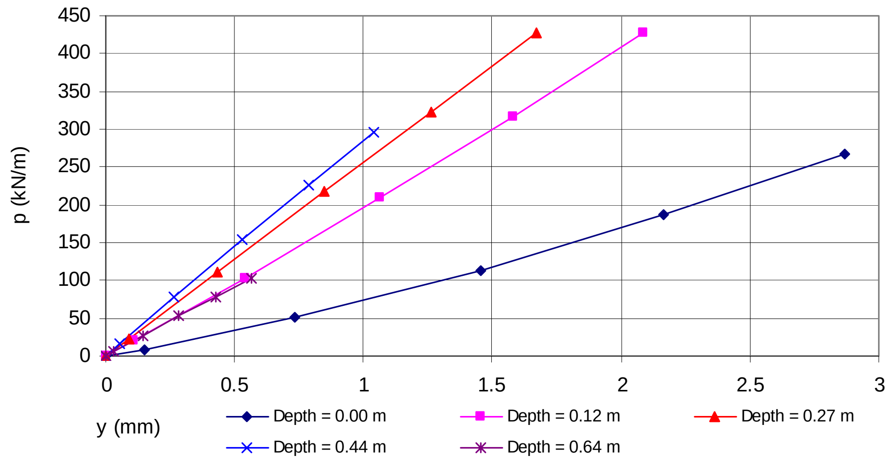


Figure 4.14 Load-transfer curves at various depths from a linear-elastic soil with interface elements

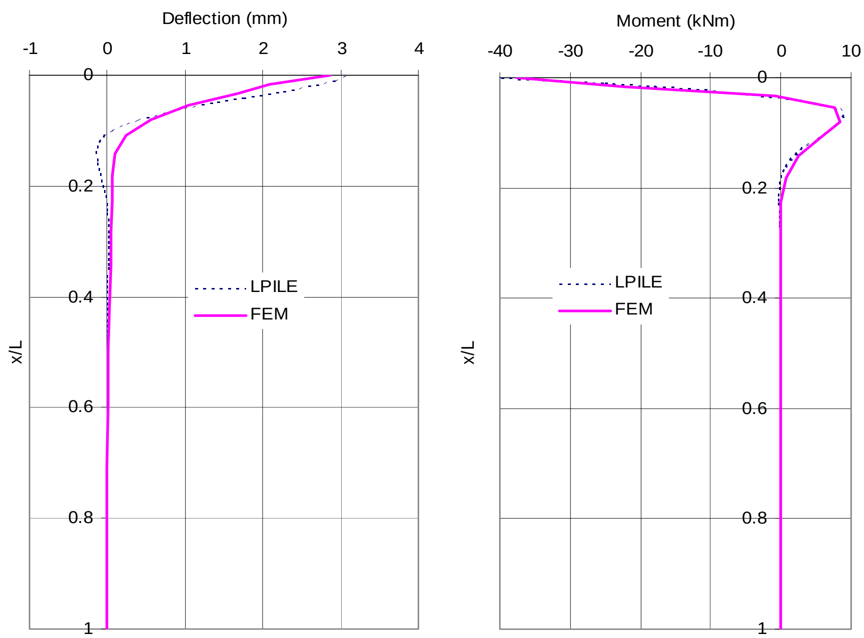


Figure 4.15 Deflection and bending moment profiles under the load of 400 kN at the micropile head from LPILE and ABAQUS

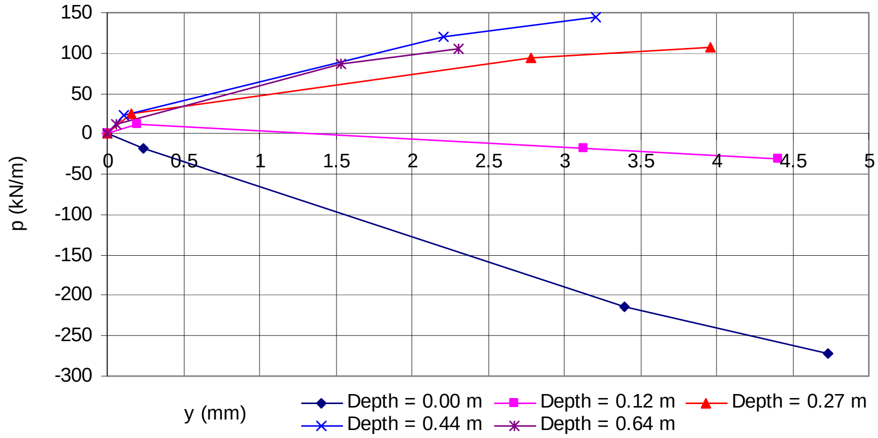


Figure 4.16 Load-transfer curves at various depths for a highly non-linear soil and interface elements

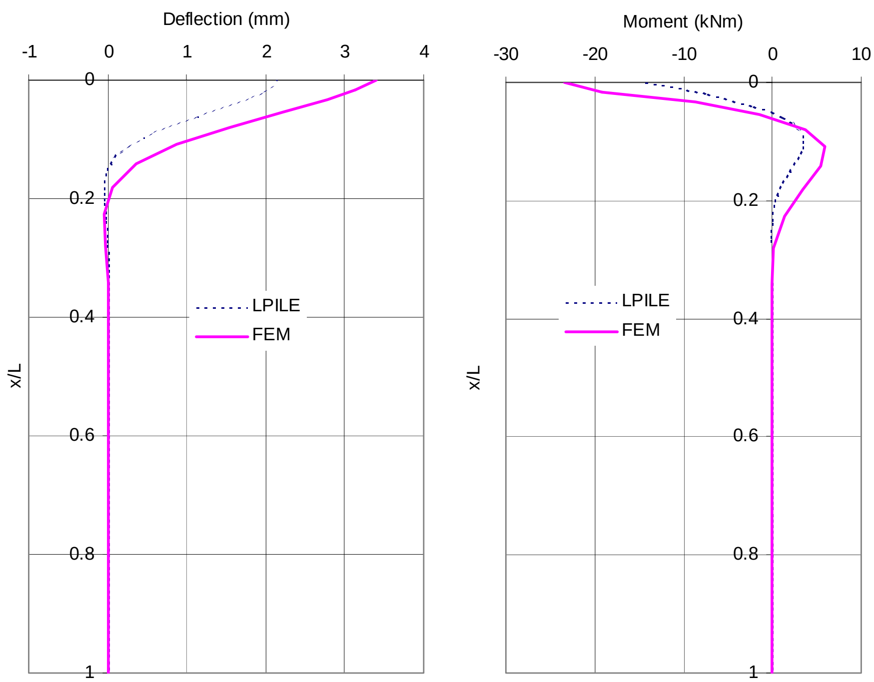


Figure 4.17 Deflection and bending moment profiles under the load of 400 kN at the micropile head from LPILE and ABAQUS for a highly non-linear soil with interface elements

Figure 4.18 illustrates the *p-y* curves derived at various depths for a mildly nonlinear soil with interface elements. As opposed to the soil used in the model represented in Figures 4.16 and 4.17, this soil does not reach shear failure. Generally, the graph shows that the *p-y* stiffness increased with depth except the one at the depth of 0.64 m. This is opposite to the trend found in the linear elastic soil without gapping (Figure

4.12) but similar to the one in the linear elastic soil with gapping (Figure 4.14). The *p-y* curves near the ground surface were more non-linear (*p-y* stiffness increased with depth). This is likely due to the higher strain levels near the surface. The softer soils near the surface resulting from soil non-linearity imply that stresses are transmitted to lower portions of the micropile. Figure 4.19 illustrates the deflection and bending moment profiles along the micropile length with the load of 400 kN at the micropile head from the FD and FE analyses. The results from LPILE and ABAQUS agreed well to each other.

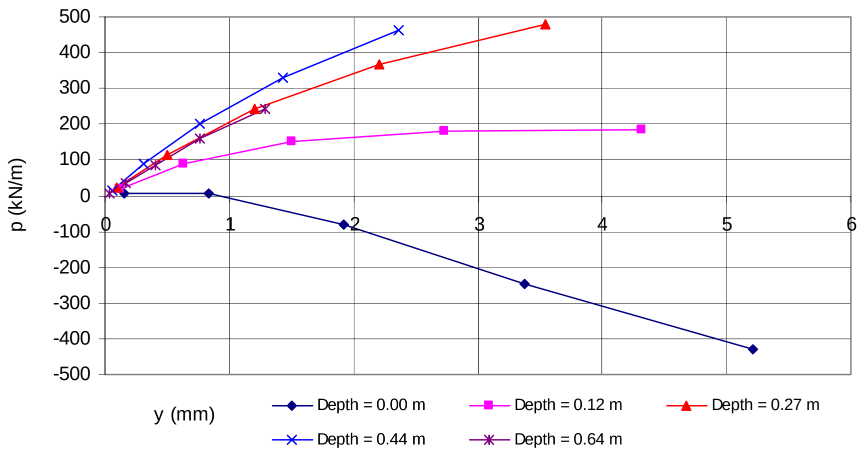


Figure 4.18 Load-transfer curves at various depths for a midly non-linear soil with interface elements

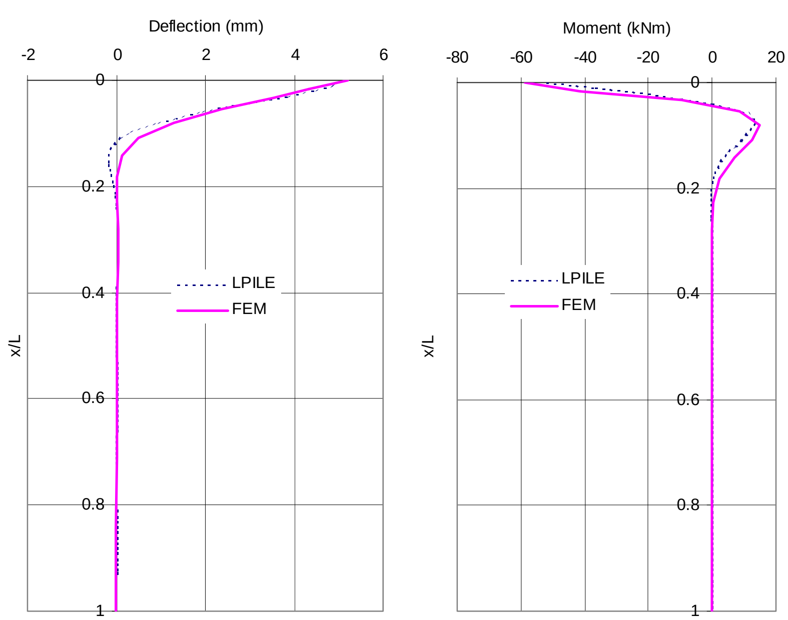


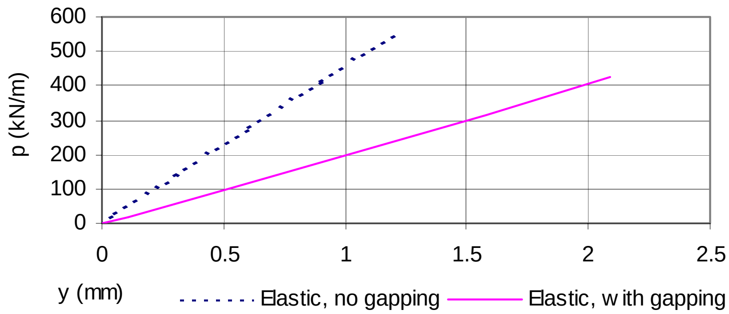
Figure 4.19 Deflection and bending moment profiles under the load of 400 kN at the micropile head from LPILE and ABAQUS

Effect of Gapping on p-y curves

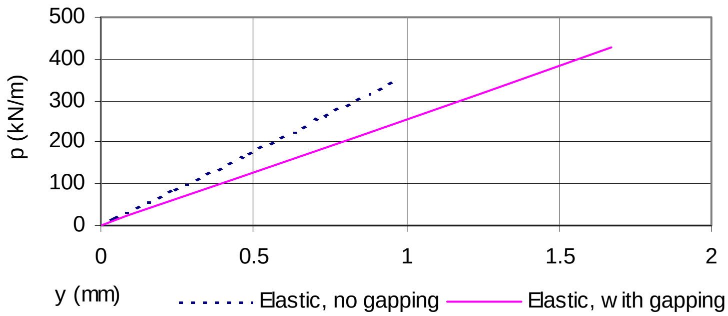
Figure 4.20 shows the *p-y* curves obtained at different depths from the linear elastic models with and without gapping. The *p-y* stiffness from the model with gapping was lower than the one without gapping at the depth of 0.12 and 0.27 m. This implies that larger deflections are needed when gapping is included to reach a given level of stress between the soil and the pile. The difference reduced with depth until the deflection from the model with gapping was larger than the one without gapping at a given *p* at the depth of 0.64 m. This can be explained by a change of curvature in the pile deflection curve at this depth. The problem should be looked at more carefully.

Effect of soil nonlinearity on p-y curves

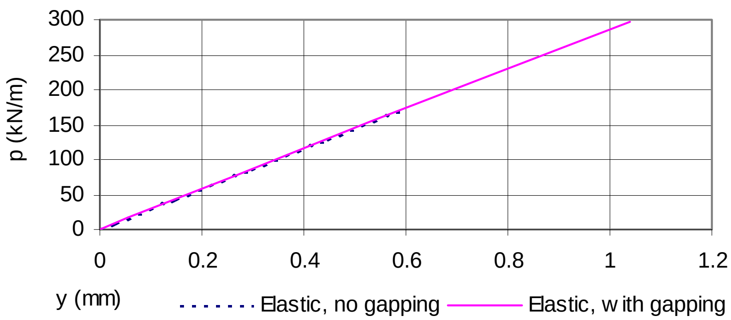
Figure 4.21 presents the *p-y* curves derived at various depths from the FE models with the clay made out from the linear elastic and the highly non-linear materials. Interface elements were incorporated in both models. Generally, the linearity of *p-y* curves demonstrates the linear elasticity of the soil material, and the non-linearity of *p-y* curves depicts the non-linearity of the soil as well. The *p-y* curve from the inelastic material at the ground surface shows high non-linearity due to high strain. It was interesting to note that the *p-y* curves from the inelastic material behaved closer to the ones from elastic material with increasing depth. This was said so because the gap between them became smaller with depth. This phenomenon is attributed to the decreasing strains with increasing depth. In other words, the inelastic soil behaved essentially more elastically with depth at smaller strains.



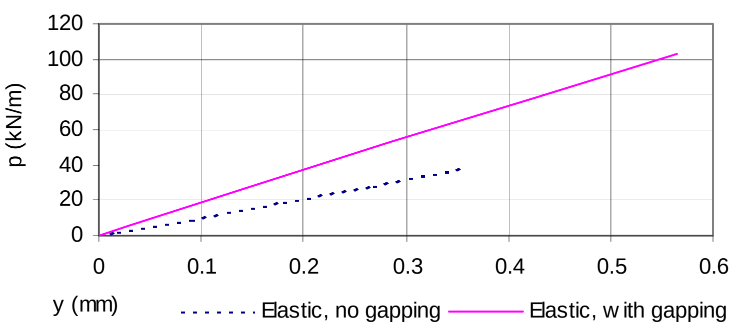
(a) 0.12 m



(b) 0.27 m

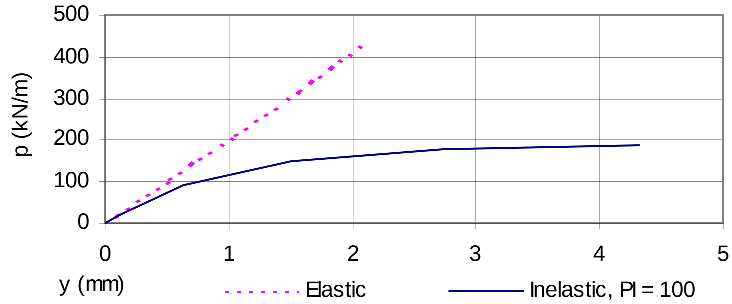


(c) 0.44 m

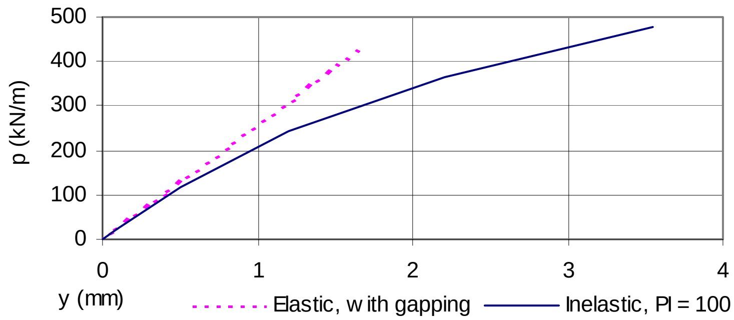


(d) 0.64 m

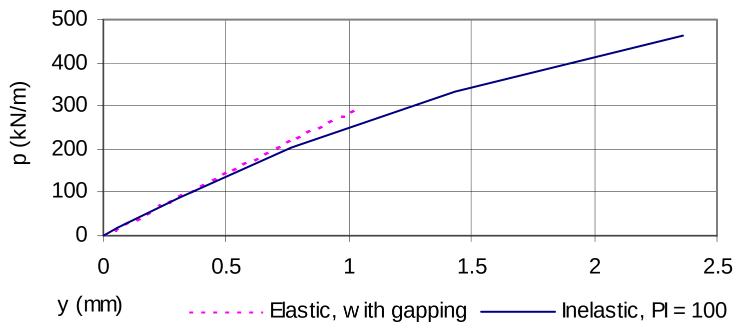
Figure 4.20 Effect of gapping on *p-y* curves at various depths



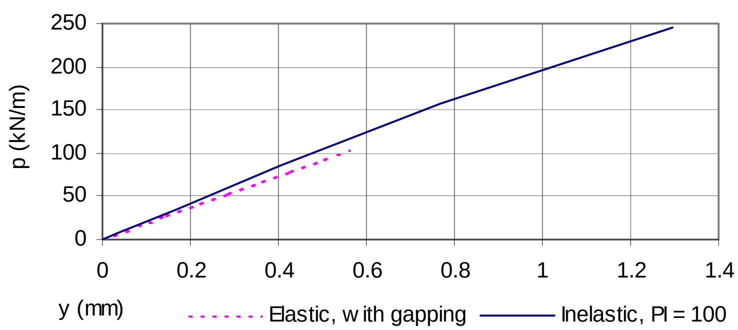
(a) 0.12 m



(b) 0.27 m



(c) 0.44 m



(d) 0.64 m

Figure 4.21 Effect of soil inelasticity on *p-y* curves at various depths

CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

The main conclusions from this study can be grouped into four categories:

(a) behavior of a single micropile under static loading,

(b) behavior of a single micropile under seismic loading,

(c) behavior of micropile groups under seismic loading, and

(d) behavior of *p-y* curves for a single micropile.

Conclusions from this study are presented in detail in the remainder of this section. For each of the cases listed above, the influence of various parameters on the response of the micropile systems is described. These parameters include the soil stiffness, the soil's nonlinear behavior, the use of gapping elements between the soil and the micropiles, static load intensity, and input motion characteristics.

Static Behavior of Single Micropile

Gapping results in an increase in deflection. For a linear elastic soil, the increase in deflection due to gapping is linearly related to the applied horizontal load. This implies that the gapping elements do not introduce non-linearity in the pile-soil systems. The increase in deflection when gapping elements are used compared to deflections in a system with perfect bonding between soil and pile is significant. Most of the deformation occurs near the top of the micropile. Hence, it is important to incorporate interface elements between the micropile and the soil at least within six diameter lengths from the micropile head. Gapping also causes higher moments near the micropile head because a lesser amount of load will be transferred to the neighboring soils. This, in turn, is due to the lower contact area between the pile and the soil.

An increase in soil's non-linearity causes an increase in deflection. In this context, an increase in non-linearity implies a more significant degradation in stiffness and strength with increasing strain levels. Even though this conclusion is self-evident, it points to the importance of using appropriate nonlinear models of soil behavior. A larger volume of the soil around the micropile head will yield in the case of more non-linear behavior. The moments from the inelastic materials, especially the one with more non-linearity, are higher than those with elastic material because of the lesser degree of load transfer from the pile to the soil in the more non-linear material.

The deflection increases with decreasing Young's modulus of the soil, E. This implies that the micropile, under certain circumstances, behaves indeed as a long flexible pile, as it is commonly assumed for the design of micropiles. There is no unique relationship between the change in soil stiffness, *E*, and the maximum moment at the micropile head. However, the higher the stiffness of the soil, the lower the length of the micropile (measured from the micropile head) that mobilizes moment resistance.

Dynamic Behavior of a Single Micropile

The non-linear behavior of the soil has a significant influence on the response of the micropile to seismic excitation. Two extremes of nonlinear behavior were studied: a soil with a large elastic range and a soil with strong non-linear behavior (e.g., large damping values and strong modulus degradation at low strains). Two material models were used to represent these two extremes, one model that matches the modulus degradation and damping characteristics of a soil with weak non-linearity, and another model matching the characteristics of a soil with strong non-linearity. The former represents a material with a large elastic range, the latter a material with strong non-linearity. The maximum bending moment at the micropile head from the material with weak non-linearity (e.g. a material with a large elastic range) is slightly smaller than the one from the elastic material. The maximum bending moment at the micropile head from the material with strong non-linearity is significantly smaller than the one from the material with weak non-linearity. The smaller moment is due to the larger hysteretic damping in the soil for the highly non-linear material.

Dynamic Behavior of Micropile Groups

At all input motion intensities, the moment envelopes of the left and right vertical micropiles were the same, implying equal distribution of loads among the vertical micropile members. However, the moment envelopes for the left and right inclined micropiles were different, implying unequal distribution of loads among the inclined micropile members. Therefore, inclination of micropiles results in unequal distribution among the micropile groups under dynamic loading. This is due to the fact that the axial resistance of the inclined micropiles also contributes to the load carrying capacity of the micropile group.

The inclination of micropiles provides larger lateral stiffness and results in smaller displacements and accelerations at the micropile head as compared to the case of vertical micropiles. The inclination of the micropiles does not affect the strain levels in the soil, implying that no additional stresses are being transmitted to the soil. The inclination of micropiles also decreases the bending moment at the micropile head. This, again, is due to the fact that the axial capacity of inclined micropiles is also mobilized (in addition to the bending capacity).

The response at the micropile head is a function of the frequency content of the input motion. The input motion with higher frequency content results in a smaller response at the micropile head. This is likely due to the fact that the higher frequency motion tends to introduce more damping than the input motion with lower frequency (longer period) content. With the exception of moments near the micropile head, an input motion with a larger predominant period results in larger maximum bending moments along the length of the micropile. This is attributed to the larger site-response effects leading to larger kinematic loading from the input motion with a larger natural period.

p-y Curves of a Single Micropile

The single most important conclusion regarding the computation of p-y curves from FE analysis is the fact that the FE analysis renders p-y curves that can be implemented into simpler, user friendly design codes. This implies that complex pile geometries and complex soil behavior can be simplified into p-y curves that can used by the design professional. Additional observations regarding the p-y curves obtained from the FE analyses are summarized as follows:

- Generally, the *p-y* stiffness increases with depth in all cases except for the model with a linear elastic soil without gapping, in which case the *p-y* stiffness decreases with depth.
- The contribution of gapping to *p-y* curves is inconclusive. While in some cases gapping results in stiffer *p-y* curves, in other cases it results in softer *p-y* curves. However, in the linear elastic soil, gapping was found to make the *p-y* stiffness increase with depth.
- The linearity and non-linearity of the soil is also reflected in the resulting *p-y* curves.
- It is important to incorporate the nonlinearity of soils into nonlinear *p-y* curves, especially at shallow depths. The *p-y* curves of the inelastic material behave more elastically at large depth due to smaller strain levels at depth.

RECOMMENDATIONS FOR FUTURE RESEARCH

The following are recommendations for further research in the subject of seismic response of micropiles. Most of the recommendations were not implemented in this study as a result of time constraints, but are the subject of current work by the PI.

- Analyses should be repeated for real earthquake time histories.
- A three-dimensional FE models for dynamic site response and SSI analyses should be implemented.

- Various degrees of inclination of micropiles should be attempted in order to investigate how the inclination affects the response.
- Many attempts were done to incorporate the interface elements in the dynamic analysis. However, divergence in the solution was encountered. Hopefully, this task could be performed with other commands in the program, ABAQUS or with other software.
- Probably a more realistic superstructure, such as a building with several stories, should be connected to the pile cap to investigate the effect of the number of degrees of freedom to the response of the micropiles.
- Full- or model- scale tests with sufficient material property information should be used for the validation of the FE models, especially for the dynamic cases.
- Other factors might have effects on the behavior of *p-y* curves should be investigated, such as the type of loading (cyclic and seismic loading), different soil type, and also the coefficient of friction.
- Buckling of micropiles has been an increasing focus and concern of engineers and researchers. Therefore, it is worth to investigate the problem using FEM by creating a void or soils with very poor strength properties such as peat, very loose sand, and soft clay around the micropile.
- FE models of micropiles used to retrofit existing foundations should be studied (e.g., the change in the dynamic characteristics of a foundation due to micropile retrofit should be quantified).

There is ongoing research at Washington State University on soil-pile-structure interaction using numerical models. The results presented herein constitute the basis upon which further research is taking place. The ability of performing “numerical tests” on FE models can greatly aid in the design of pile-supported structures. Current research is guided towards the development of practical guidelines based on the numerical platform presented in this report.

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