Seismic Retrofitting Using Micropile Systems Centrifugal Model Studies

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ABSTRACT

A series of centrifuge tests were conducted on micropile group and network systems in order to [1]investigate the response to earthquake loading and soil-micropile interaction behavior. Model tests on group and network systems embedded in loose to medium dry sand are described. Micropile bending moment, deflection, and acceleration were measured during testing. Dynamic p-y curves were derived from the measurements for low and high levels of shaking and were compared with the backbone p-y curves for sand recommended by API and other published data. Group and network effects were investigated for different configurations and at different levels of loading. For the selected frequency of excitation, the results indicate a positive group effect increasing with the number of piles and the batter angle. This paper describes the experimental procedures used to carry out the centrifugal model tests and summarizes the main preliminary results.

KEY WORDS: centrifuge, cyclic, dynamics, frequency, fourier, group, micropiles, network, shaking, spectra.

INTRODUCTION

Micropiles are defined as small-diameter drilled and grouted piles, subset of cast-in-place replacement piles. With conventional cast-in-place replacement piles, in which most of the load is resisted by concrete as opposed to steel, small cross-sectional area is synonymous with low structural capacity. This is not the case with micropiles, however. Innovative drilling and grouting methods permit high

grout/ground bond values to be generated along the micropile's periphery. To exploit this benefit, high-capacity steel elements can be used as the principal load-bearing element, with the surrounding grout serving only to transfer.

grout/ground bond values to be generated along the micropile's periphery. To exploit this benefit, high-capacity steel elements can be used as the principal load-bearing element, with the surrounding grout serving only to transfer, by friction, the applied load between the soil and the steel. End-bearing is not relied on, and in any event is relatively insignificant given the pile geometry involved.

Micropiles have been subclassified according to diameter, construction process or the nature of the reinforcement. However, in the course of the FHWA study (Bruce and Juran, 1995), it has been concluded that a new, rigorous classification system for micropile design should be adopted based on two

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criteria: the philosophy of behavior, which dictates the basis of the overall design concept; and the method of grouting, which mainly determines grout/ground capacity.

Based on the philosophy of behavior, two basically different design concepts which are illustrated in figure 1 have been developed (Bruce and Juran, 1995) for engineering practice of micropiles, namely: Case 1 referring to micropiles which are designed to transfer structural loads through soft or weak soils to more competent strata. These micropiles are generally used as structural support to resist directly to the applied loads. Case 2 referring to Lizzi's (1978) original "root pile" design concept, relies primarily on using a three dimensional network of reticulated friction micropiles to create in-situ a coherent, composite, reinforced soil systems.

The behavior of micropile group systems under static loading has been investigated and design guidelines have been assessed in a state of practice review (Bruce and Juran, 1995). However, the application of micropiles to seismic retrofitting is facing the need for established and reliable design guidelines.

As reported by Lizzi, foundation with root piles in Italy have already survived earthquakes. It is well known that structures and installation which display a certain subtleness and flexibility perform in ground better than those are too rigid. Shock absorbing devices follow the same principle

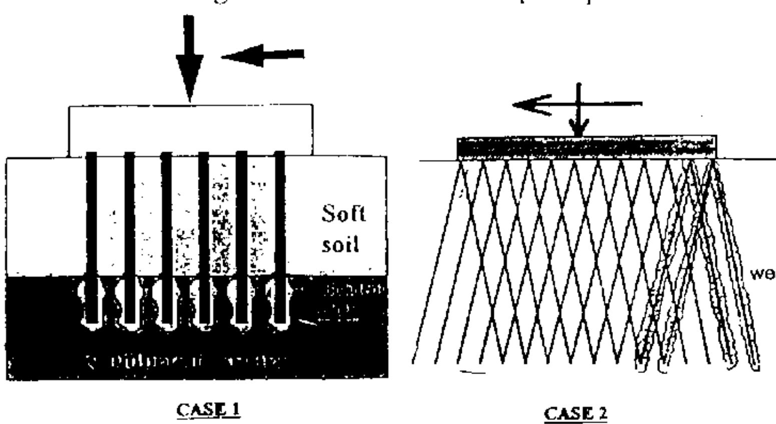


Figure 1. Design Concepts of Micropile Systems

Micropile is a very flexible pile. Due to its slenderness and its ductile steel core, it can be assumed that it follows best the shock induced displacements in the ground and that it remains integrated with the soil. With a group of micropiles being parallel or battered, a reinforced soil body is created which performs also in a flexible way. However, many issues needs to be addressed at this level and for proper design, still considerable research work needs to be done in order to establish and assess seismic design guidelines for micropile systems.

In order to investigate the seismic behavior of micropile group systems (isolated piles, groups and networks of reticulated micropiles) under axial, lateral and combined loading in selected types of engineering applications, a proposed Workplan for Laboratory Centrifugal and Numerical Model Studies on the Seismic Behavior of Micropile Systems has been adopted by the FHWA in conjunction with the FOREVER French program. The prime objective of these model studies, accomplished by the Polytechnic University at New York in cooperation with the Ecole Nationale des Ponts et Chaussees Geotechnical Research Center - CERMES at Paris and the University of Canterburry at New Zealand, is to provide the experimental data base to develop and evaluate seismic design methods for selected engineering applications, including new construction in earthquake zones and seismic retrofitting of bridge foundations, retaining systems and slope stabilization

It is anticipated that this cooperative research program will provide the necessary database for the development and evaluation of seismic design methods for micropile groups and networks for infrastructure applications. A series of centrifuge tests on instrumented micropile group systems were performed at the Rensselaer Polytechnic Institute (RPI) Geotechnical Centrifuge Research Center and several configurations of model micropile group systems were tested in loose to medium dry sand. The main objectives of this phase were to:

(i) Study the seismic response of micropiles subjected to a vertical loading at different acceleration levels, (ii) Investigate the group and two-dimensional network effect in the case of different configurations of micropile groups (cf. table1) under a seismic loading subjected to a static axial loading simulating the effect of a superstructure, (iii) Investigate the effect of micropile inclination on the response of micropile groups to seismic loading.

This paper describes the experimental procedures used to carry out the centrifugal model tests and summarizes the main preliminary results

CENTRIFUGE TESTING

The centrifuge tests were performed at the Rensselaer Polytechnic Institute (RPI) Geotechnical Centrifuge Research Center. A detailed description of the RPI centrifuge facility is presented by Elgamal et al (1991). In the present series of tests, use was made of the rectangular, flexible-wall laminar container built at Rensselaer Polytechnic Institute to closely approximate a continuous shear strain field in the soil during shaking and accommodate possible shear strain concentrations.

Pile Properties and Model Layout

Based on the principles of dimensionless analysis, similitude requirements have been used to define geometrical and structural model piles used in the centrifuge tests. The model scale factor is defined as $n = L_p / L_m$, where subscript p refers to the prototype and subscript m refers to the model and L is the length of the pile.

The geometrical parameters have been defined according to 2 main considerations: (i) Similitude requirements in order to model full scale experiments carried out by Plumelle (1993), and 1g shaking table tests at University of Canterbury (L=5 m, D = 10cm, reinforcing steel: tubing steel 50.3 - 40.3 mm, New Zealand (ii) Size of the laminar box used in the centrifuge tests.

The structural characteristics of the model piles to be used in the centrifuge tests were chosen to maximize flexure of the pile during vibration. Interface properties were taken into account by gluing sand particles along the entire pile length, and local compaction around the pile to simulate high grout/ground bound and confining effects (Juran et al. 1997 ; Abdoun, 1996, Vucetic et al., 1993). It was also desirable to have the end of the pile with a sufficient distance from the base of the box to ensure that end bearing of the piles will not be significantly influenced by the laminar box base. For this purpose, the pile tips were about 5 diameters above the container base (Abghari et al., 1995). It was also desirable to use a pile of sufficient length to ensure that pile tip reactions did not significantly influence the pile head response. Based on this, a scaling factor of 20 has been adopted.

The model piles were constructed of polystyrene with roughened shaft and a Young's modulus of E_m = 2700 Mpa and a length of L=21.3 cm. The Young modulus of the model piles has been determined after computing the pile flexural rigidity E_mI_m from measurement of pile deflection versus applied transverse load using static beam deflection formula. An average value of 2179 Mpa has been adopted for the elastic Young modulus of the model micropiles. The selected diameters are 6.5 mm and 9.5 mm achieving slenderness ratios of 33, and 22 respectively. Table 1 summarizes pile model properties.

Table 1. Model micropile properties

Outside Diameter D (mm)	Inertia Modulus I_m (mm^4)	Flexural Rigidity E_mI_m ($N\,mm^2$)	Slenderness Ratio-L/D
9.5	3.99.10[illegible]	10.78 10[illegible]	22.4
6.5	0.87 10[illegible]	2.36 10[illegible]	32.8

The pile was strain gauged with 6 pairs of foil type strain gauges mounted on the outside of the pile to measure peak bending and axial strains. A mass is generally screwed at the pile head applying an axial loading of 50% and 90% of the failure load determined from 1g static tests in order to simulate the influence of a superstructure. The same procedure has been adopted for pile group tests.

Instrumentation and Data Acquisition

As shown on figure 2, the instrumentation consist of:

- LVDT's :Transducers at different locations, lv1 and lv2 to record any lateral soil profile displacement (not expected), lv3 for the pile cap settlements, lv4 for the lateral pile cap displacements, and lv5 to record surface settlements.
- Accelerometers Model 303A03 from PCB Piezotronics were used at different locations for acceleration -time history measurements. The labeled input accelerometer is used to monitor the input base motion, acc1 to acc5 to record the wave propagation along the soil profile and the free field accelerations, and acc6 to measure the pile head accelerations in order to characterize the structural response of soil-micropile system.
- Pairs of half bridge circuited strain gauges were installed on the surface along the model piles to monitor bending (sg2, sg4, and sg6) and axial (sg1, sg3, and sg5) strains. The strain gauges were type CEA- 13- 125 UN-120).

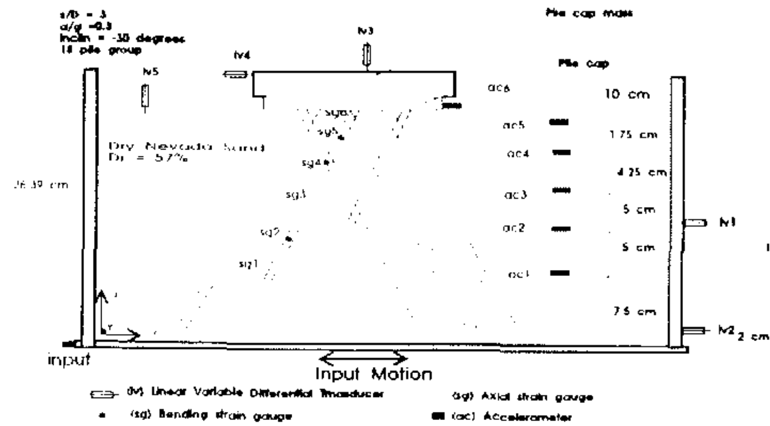


Figure 2. The micropile model system tested by Polytechnic University (From Juran et al., 1997)

Soil Properties

A Nevada sand 120 was used at a relative density of 57 %. Tests (Arulmoli et al., 1992) conducted on the sand yielded maximum and minimum void ratios of 0.51 and 0.88, specific gravity of 2.67, an average particle size D_{50} of 0.13 mm and a coefficient of uniformity (D_{60}/D_{10}) of 1.6.

The values of a peak friction angle of ϕ=33° and 36° were obtained from laboratory triaxial tests on Nevada Sand for relative densities of 40% and 60%, respectively. A value of ϕ=35° was therefore selected for the testing at a relative density of 57%.

The influence of average grain size (D_{50}) relative to the pile diameter in centrifuge tests has been studied by Oveson (1975) and summarized by Cheney (1985). It has been concluded that there is no significant influence of the grain size on the load settlement behavior for $30 < D/D_{50} < 180$. The choice of 6.5 and 9.5 mm pile diameter in the Nevada sand corresponds to a ratio of 50-73, which is adequate to minimize the effect of particle size on pile behavior.

In order to study the influence of the amplitude of the base shaking on the relative density, a series of tests were conducted where 2 sand samples were prepared in rigid boxes at 40 and 57 % initial relative density. 3 LVDT's were placed on the soil surface to monitor the settlements which were used to compute the variation of the relative density with the amplitude of shaking. The results are displayed on figure 3. It can be seen that, due to soil densification, there is practically no effect of the initial relative density on the relative density of the sand sample achieved after 100 cycles. A relative density of 57% was selected for the tests consistently with the

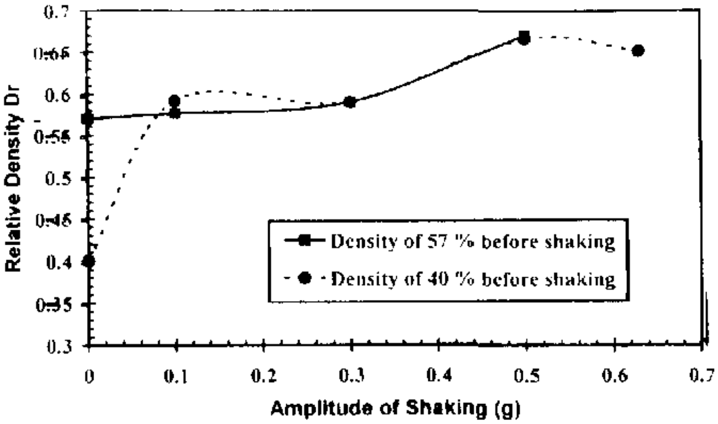


Figure 3. Influence of amplitude of shaking on the initial relative density of Nevada Sand 120

relative density used in the 1g shaking table model tests conducted by Canterburry University in New Zealand.

Experimental Testing Procedure

The dynamic tests consisted of horizontally shaking the models in flight at 20 g. For each configuration, the horizontal shaking included sequences of 100 uniform cycles of sinusoidal accelerations at 40 Hz. Sinusoidal ground motions were used in order to enable dynamic analysis of basic patterns of model behavior, which are more difficult to perform with more complex input motions. The models were first subjected to a prototype acceleration time history with amplitudes of 0.3 g with cap only, and then under 50% and 90 % of the estimated failure load. The failure load (FL) is defined as the vertical load which causes failure under static loading by occurrence of large deformation and loss of friction. As recommended by Weltman (1980), a limiting displacement of 10 percent of the pile diameter was chosen to define failure in compression .

The main parameters of the experimental program are summarized in table 2.

CENTRIFUGE TEST RESULTS

Figure 4 illustrates typical input base shaking (input), free field (a3) and pile head accelerations (a6) recorded in test 7 (2 pile group- cap alone - s/D =3). The actual input acceleration consists primarily of 100 cycles of a 2 Hz sinusoidal signal, having 0.3 g average amplitude in prototype units. Comparison between the acceleration time histories indicates an amplification of the base input acceleration toward the ground surface, which is similar to the behavior observed by other centrifuge investigators for cohesionless soils (e.g. Steedman and Zang, 1990; Takemura et al., 1991, Gohl, 1991).

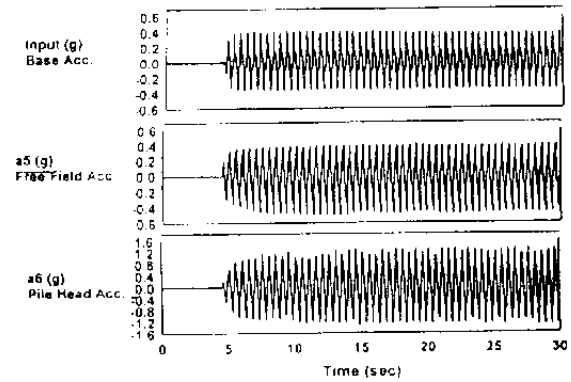


Figure 4. Typical Recorded Acceleration Time Histories

The effect of the number of cycles on the variation of the relative density is displayed on figure 5 for test 7 during all the events (cap only, 50% Fl, 90 % Fl). A steady state is generally reached after the first 50 cycles.

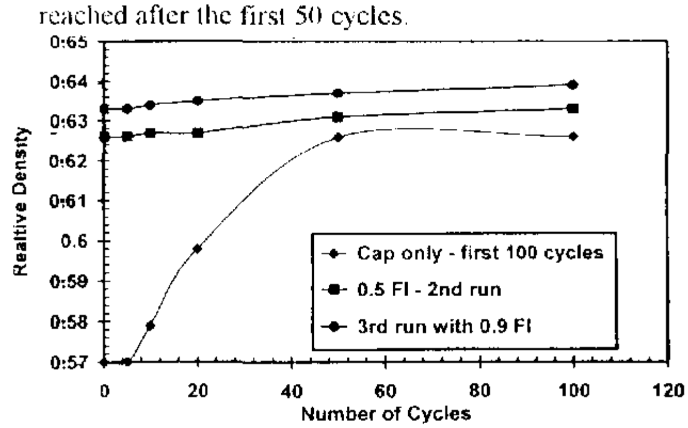


Figure 5. Variation of the relative density with the number of cycles during Test 7 (all events)

Table 2. Main parameters of the achieved experimental program.

Micropile Configuration	*Diameter (Model) D (mm)*	*Slenderness Ratio L/D*	*Micropile Inclination α (deg)*	*Spacing to Diameter Ratio s/D*	*Ampl. of Acc. (Prototype a/g)*	*Loading Level*
Single Micropile	9.5 6.5	32 22	0 0	- -	0.03 to 0.5	100 cycles- cap only 100 cyc. - 0.5 Fl 100 cyc. - 0.9 Fl
Group of Micropiles 2x1	6.5	32	0	3 5	0.3 until failure	100 cycles- cap only 100 cyc - 0.5 Fl 100 cyc - 0.9 Fl
	9.5	22	0-30	3 5	Same	Same
Group of Micropiles (2x2)	6.5	32	0	3 5	Same	Same
3x(2x1)	6.5	32	10 - 30	3	Same	Same
3x3x(2x1)	6.5	32	10 - 30	3	Same	Same

ANALYSIS OF THE CENTRIFUGE TEST RESULTS

The interpretation procedure adopted to analyze the centrifuge tests results involved the following

- Spectral Fourier Analysis of the input ground motion and the recorded accelerations in order to characterize and evaluate the soil-micropile system dynamic response.
- Characterization of the soil pile interaction during base motion excitation through the development and assessment of dynamic p-y curves
- Comparison of the derived p-y curves with the recommended p-y by the American Petroleum Institute (1983) and published data.
- Analytical simulations to check the accuracy of the test results
- Investigation of the group and network effect

Natural Frequency of the Soil/Micropile System

It is generally agreed that the most meaningful engineering characteristics of ground motion are presented by Fourier and Response Spectra of the recorded ground accelerations. These representations exhibit the recorded acceleration time histories in the frequency domain in order to gain insight on the frequency content and response characteristics.

In the case of our tests, Fourrier spectra was calculated for each of the accelerometers (input, ac3, ac6) for each event of every test.

The determination of the natural frequency for each investigated configuration is an important parameter for analytical modeling. The general procedure requires to excite the model at different frequencies and from the plot of peak responses to identify the natural frequency of the system as well as to detect any non linearity from the lack of symmetry of the plot of the peak responses. However, due to the experimental constraints, this approach could not be used. Therefore, The following procedure was adopted. Fourier amplitudes computed from the measured pile head accelerations (Aph) have been normalized with respect to the amplitudes computed from the free field accelerations (Aff). The frequency at which a peak amplitude ratio occurs can be used to characterize the natural frequency of the micropile system. The same procedure is used to obtain the natural frequency of the free field by normalizing the free field accelerations with respect to the input base motion. This procedure has been extensively used by Gohl (1991) and Tufenkjian and Vucetic (1993)

The frequency range over which the spectra were plotted was narrowed in order to examine the most meaningful response. This range encompassed frequencies between 1 to 5 Hz. Frequencies less than 1 Hz and greater than 5 Hz were discarded. The selected frequency range is consistent with results obtained by Tse et al. (1964) for a single degree of freedom oscillator subjected to sinusoidal base excitation indicating that the acceleration ratio is only active over a small range of frequency ratio values (0.5 - 2.0). Response amplification outside this range could not be realistically detected. Therefore, in this analysis, the frequency ratio was conservatively chosen to vary from 0.4 to 2.0.

The procedure outlined above was applied to all the configurations during the main events in order to characterize the natural frequency of the micropile system. The results are displayed on figure 6 for the case of single pile (test 2), 2x1 pile group (test 7), 2x(2x1) pile group (test10), 6 pile group arranged in 3 elements of 3x(2x1) inclined piles at 10 degrees (test 16), 18 pile group system disposed inclined at 10 degrees 3x3x(2x1) (test 18) and 30 degrees (test 19). The results clearly show that, for a specified level of shaking (a/g=0.3) : (i) the estimated fundamental frequency of the micropile system is strongly affected by the micropile system configuration. A value of 1.1 Hz can be adopted for the natural frequency of the single micropile. This value increased to 2.2 Hz for the 18 micropile network system where the piles are inclined at 10 degrees.(Test 18). ii) a greater interaction occurs for higher pile inclination, as a value of 4 Hz was observed for an inclination of 30 degrees (Test 19).

Soil-Pile Interaction

To determine the nature of interaction between the soil and the pile during base motion excitation, cyclic p-y curves were derived from the single pile data using procedures described by Ting (1987) and briefly summarized herein.

From simple beam deflection theory, the moment in the pile M is proportional to the recorded flexural strain ε:

$$M = EI\frac{d^2y}{dz^2} = EI\frac{\varepsilon}{h}$$

where y is the lateral pile deflection, z is the vertical coordinate along the pile, h is the distance from the strain gauge to the neutral axis of bending, and EI is the flexural stiffness of the pile. The moment may be integrated twice for the deflection y or differentiated twice for the pressure p.

Since the strain data are known at discrete locations along the pile, a numerical scheme is necessary to obtain the needed pressures and deflections and dynamic p-y curves.. The method developed by Ting (1987) for the full scale data was used with a suitable assumption to take into account the rigidity of the pile cap connection This method involves the use of a least square fitting procedure for the moment profile with a seventh degree polynomial

Since the dynamic tests were conducted in sand, the polynomial should be subject to the constraint that the net soil pressure is zero at the ground surface. The other main boundary conditions assume zero moment and shear forces at the pile tip

The numerical procedure was evaluated by comparing the computed deflections at the top of the pile head mass with measured displacement. Figure 7 displays for a loadin

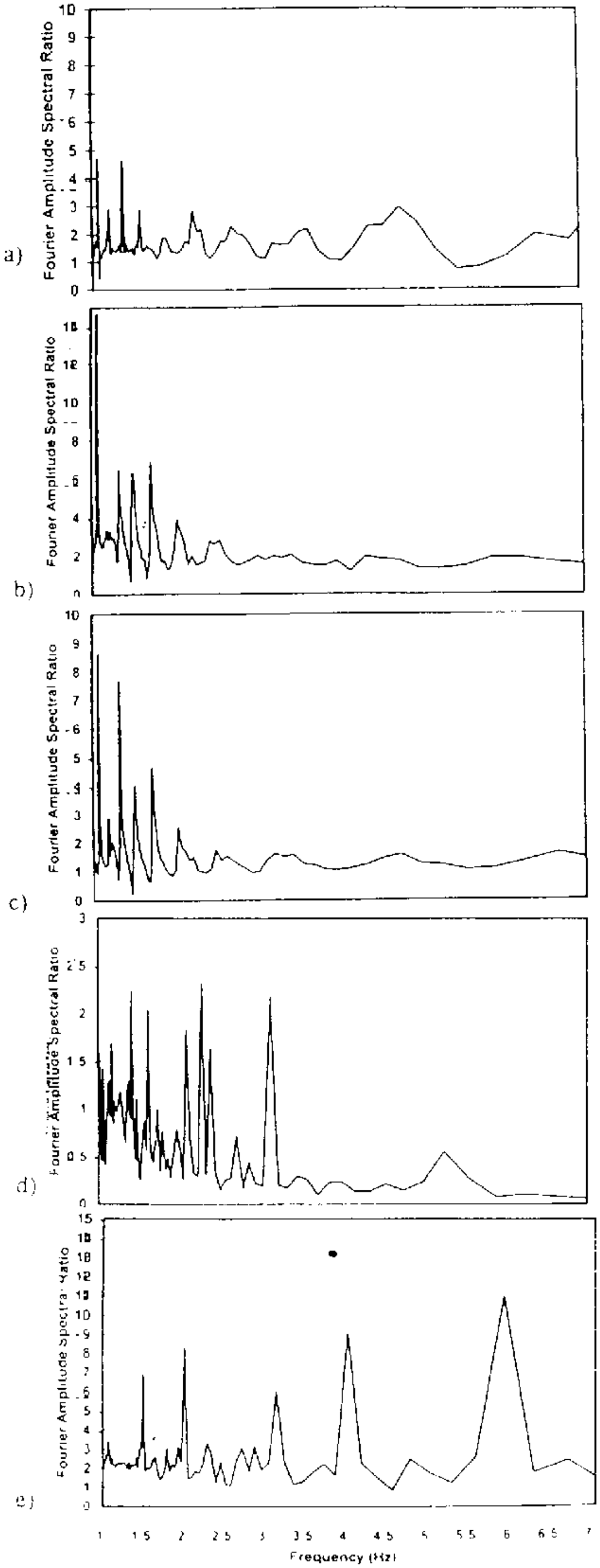


Figure 6. Variation of the Fourier amplitude spectral ratio of the pile head acceleration normalized with respect to the free field acceleration for : a)single pile- b) 2x1 vertical pile group. c) 3x(2x1) inclined network system (α=10°). d) 3x3x(2x1) network system (α=10°). e) 3x3x(2x1) network system (α=30°) - s/D= 3 - L/D =32 - a/g=0.3

measured displacement. Figure 7 displays for a loading cycle during steady state shaking the displacement profiles of the pile derived with Ting's procedure for the case of a single pile subjected to a seismic base motion of 0.03g prototype amplitude of acceleration.(test 1-203). It illustrates that computed displacements of the pile cap agree fairly well with measured data.

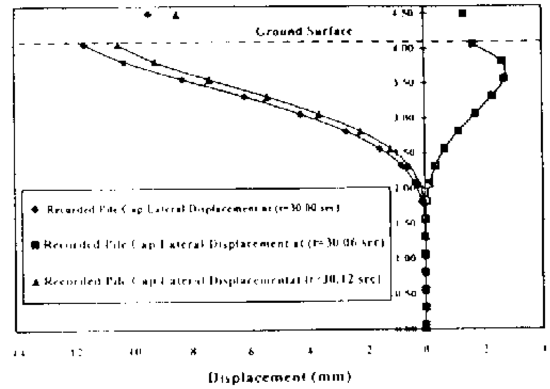


Figure 7 Displacement Profiles for test 1(203) during steady state shaking cycle derived using Ting's procedure and measured pile cap displacement.

Cyclic p-y curves were computed for the same test during the shaking cycle where maximum pile deflection occurred and are shown for a range of depths in figure 8. The p-y curves up to about 9 pile diameter are non linear and exhibit hysteresis loops. Up to 6 pile diameter, the secant lateral stiffness of the soil, defined as the slope of the line passing through the end point of the loop gradually increased with the depth. No signs of gapping between the sand and the piles are evident.

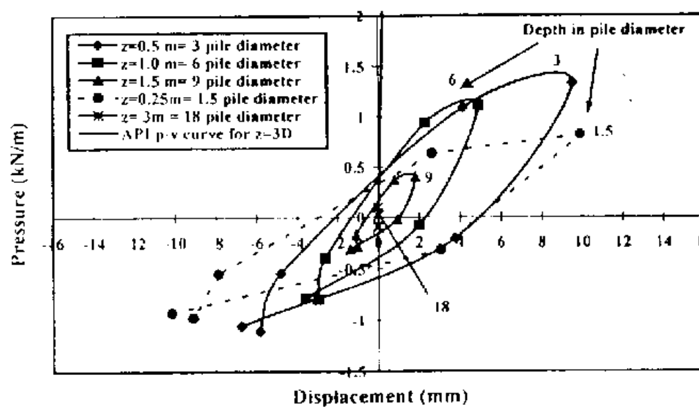


Figure 8. Cyclic p-y curves at various depths during steady state shaking cycle (t=30-30.5 sec) and Comparison with API p-y curve - Centrifuge test 1(203).

Cyclic p-y curves were derived from single pile analysis using the same procedure for the case of a strong shaking (a/g=0.3) Similar behavior was observed except that signs of gapping were observed at shallow depthes. Figure 9 compares the computed p-y curves under strong shaking (a/g=0.3) with the cyclic p-y curves recommended by the API (1983) and the p-y curves reported by Gohl (1991) based on centrifuge tests on model piles with prototype bending stiffness of EI = 172 Mpa. The API curves were computed using a peak friction angle of 32 degrees and an n_h value of 6750 kN/m^3. The latter defines the initial slope of p-y curve and was based on values recommended for loose dry sand. It can be seen that the API and the Gohl (1991) p-y curves are considerably stiffer compared to the experimentally derived p-y curves based on the centrifuge tests on model micropiles. This could be related to the lower bending stiffness of the pile which results in larger pile deformation and better strain compatibility between the soil and the pile leading to a significantly lower viscous damping of the soil-pile system.

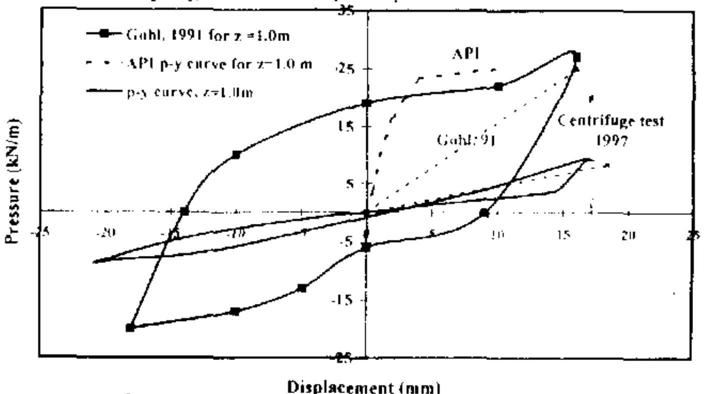
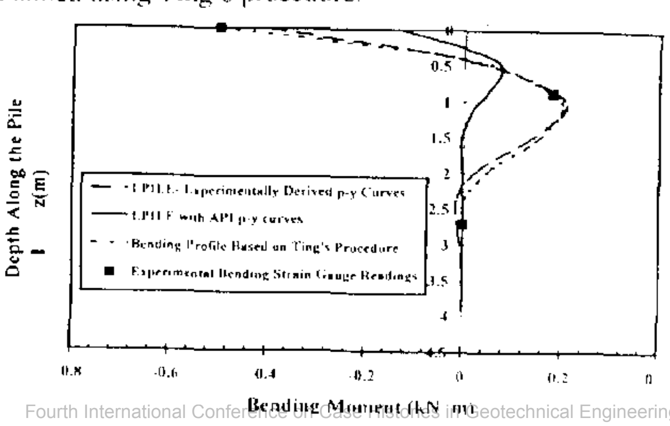


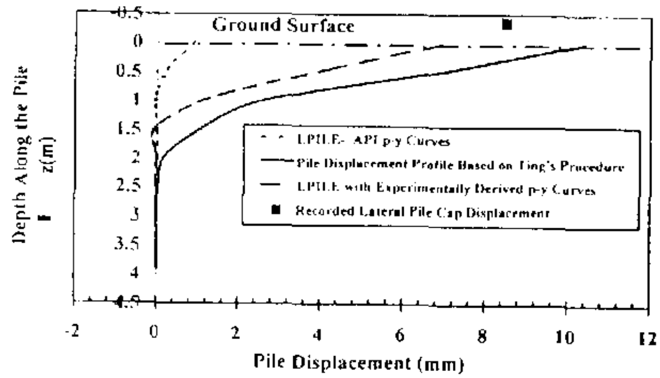
Figure 9. Comparison of the experimentally derived cyclic p-y curves during steady state shaking cycle (t=30-30.5 sec-centrifuge test 1-203) with API p-y curve, and (Gohl, 1991).

In order to evaluate the potential use of the LPILE finite difference computer program for the soil-pile interaction analysis, the program has been used with the experimentally derived p-y curves to compute the bending and pile displacement profiles. The numerical results were compared with the experimental measurements. For low level shaking, the flexural response of the model pile was assumed to be dominated by structural inertia rather than by the free field ground motions. The kinematical loading was neglected as the free field follows the motion of the pile. The inertial loading at the pile head was estimated based on the difference between the recorded pile cap accelerations and the input base motion

Considering test (1-203), several iterations were done to define the fixity conditions at the pile head (moment and shear force) and full fixation was assumed to yield the most appropriate matching between pile displacement computed with LPILE and the experimentally derived pile displacement profile obtained using Ting's procedure. LPILE was used considering both the experimentally derived p-y curves and API p-y curves. Figure 10a and 10b illustrate that the LPILE simulations with the experimentally derived p-y curves agree fairly well with the pile bending and displacement profiles obtained using Ting's procedure.



a)



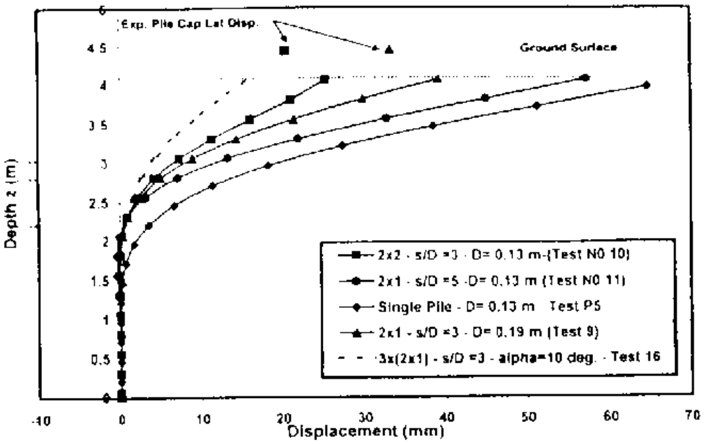
b)

Figure 10. Comparison of a) the bending profile obtained by Ting's procedure with bending profiles obtained through iterations using LPILE based on API and experimentally derived p-y curves, b) corresponding displacement profiles Test 1 (203) - single pile- Low level shaking (a/g=0.03)

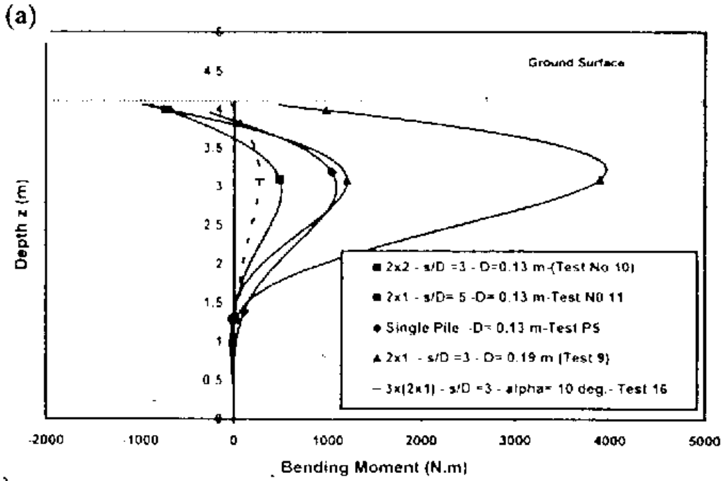
GROUP EFFECT

Based on the bending strain gauge recordings, bending moment profiles of the instrumented micropile in the various configurations for each event were derived to investigate the group effect. Figure 11(a) and 11(b) displays respectively the maximum relative pile displacement with respect to the free field and the associated distributions of the bending moment profiles for (i) single pile (D=0.13m), (ii) 2x1 and 2x(2x1) vertical pile groups with D=0.13m, (iii) 3x(2x1) group with a pile inclination of 10 degrees and D=0.13 ; (iv) 2x1 vertical pile group with D=0.19m. All tests configurations were subjected to a vertical loading equivalent to 50 percent of the static failure load (0.5 Fl). Computed displacements were compared with measured pile cap displacements.

The experimental results illustrate that : (i) Bending moment and displacement profiles for single pile and a group of (2x1) vertical piles with s/D=5, under the same equivalent loading conditions, are quite similar illustrating a negligible interaction effect for s/D=5; (ii) The bending stiffness significantly affect the bending moment and displacement magnitude of the pile group, the bending moment obtained for the pile diameter of D=0.19m is about four times greater than that measured for D=0.13m pile, while the displacement is about 150% smaller than the displacement of D=0.13m pile. The normalized bending moment of the two pile groups (M/EI) are of similar magnitude, while displacement increases according to pile flexibility. (iii) The experimental data illustrate a "positive" group effect, which results in smaller bending moments and displacements for the 2x(2x1) and 3x(2x1) pile groups with spacing to diameter ratio of 3 as compared with data measured for identical single pile and 2x1 pile group with S/D=5. It is worth noting that the normalized stiffness ratio defined as the ratio of the dynamic stiffness of the pile group to n times the static stiffness of the single pile is significantly larger than that cited in the litterature for conventional piles in elastic and visco-elastic media which are generally associated with a negative group effect (i.e. Dobry and Gazetas., 1988 ; Velez et al., 1984). These observations appear to be consistent with the positive group effect observed by several investigators (i. e. Lizzi, 1982 ; Bruce and Juran, 1997 ; O'Neill, 1983) for vertical pile groups subjected to static loading in cohesionless soils.



(a)



(b)

Figure 11.Comparison of: (a) Recorded displacement profiles for single pile, (2x1), (2x2) Vertical Pile Groups and 3x(2x1) network system (alpha =10deg) During peak pile Lateral Displacement (a/g =0.3 - s/D =3 -L/D =32) (b) Corresponding bending moment Profiles (From Juran et al., 1997)

CONCLUSIONS

The principal conclusions from the preliminary analysis of the centrifuge tests conducted on model micropile configurations can be summarized as follows:

(i) Micropile systems present a flexible behavior. For the selected normalized frequency simulating earthquake excitation, due to the relatively low micropile stiffness pile deformation follows closely free field motion except at shallow depths. This results in a composite seismic response as the micropile system transfers the inertial force of the accelerated superstructure to the soil through soil-structure interaction with relatively low dynamic sresses.

(ii) the estimated fundamental frequency of the micropile system is strongly affected by the micropile system configuration. A value of 1.1 Hz was adopted for the natural frequency of the single micropile based on the experimental results. This value increased to 2.2 Hz for the 18 micropile network system where the piles were inclined at 10 degrees. A greater interaction occurred for higher pile inclination, as a value of 4 Hz was observed for an inclination of 30 degrees.

(iii) Soil-pile interaction under lateral base shaking was evaluated in terms of dynamic p-y curves. The p-y curves obtained under strong shaking were found to be non-linear and low damping. This could be related to the lower bending stiffness of the pile which results in larger pile deformation and better strain compatibility between the soil and the pile

(iv). The experimental data illustrate a ``positive`` group effect, which results in smaller bending moments and displacements of the 2x(2x1) and 3x(2x1) pile group with spacing to diameter ratio of 3 as compared with data measured for identical single and (2x1) pile group with s/D=5. These observations appear to be consistent with the postive group effect observed by several investigators for vertical pile groups subjected to static loading in cohesionless soils.

However, the complex soil-pile-superstructure interaction under seismic loading for micropile groups and network systems raise challenging R & D needs for the research community in the years to come in order to develop reliable design guidelines for micropile systems in earthquake zones

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