Engineering Analysis of the Dynamic Behavior of Micropile Systems

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ABSTRACT:

A series of centrifugal tests were conducted on micropile groups and network systems, to investigate the system response to earthquake loading and the superstructure soil-micropile-interaction behavior. Model tests on vertical and batter micropile systems embedded in loose to medium dry sand under different levels of shaking are described. Group and network effects were investigated for different configurations and at different levels of loading. The experimental test results were analyzed parametrically to study the effect of the main controlling parameters. For the model testing conditions, the experimental and parametric results indicated: (i) a positive group effect increasing with the number of piles and the batter angle; (ii) dynamic p-y curves were found to be non-linear with low damping and much softer than the published data (API); (iii) for inclination of 10° and 30° a substantial improvement is observed in the superstructure response with acceleration reduction to 40% of the values obtained in case of vertical piles. The computer programs LPILE & GROUP were used in a pseudo-static analysis approach to simulate the representative centrifugal model tests in order to evaluate the available analysis methods as engineering tools to assess the seismic behavior of micropile groups and networks.

KEY WORDS: centrifuge, dynamics, group, micropiles, and network.

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INTRODUCTION

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, 1997). However, the application of micropiles to seismic retrofitting is facing the need for established and reliable design guidelines.

As reported by Herbst (1994), foundations with root piles in Italy have already survived earthquakes. 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 French FOREVER program (Juran et al, 1998). 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 Lille – France, 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.

Centrifuge model studies on instrumented micropile group and network systems (Juran et al, 1998; Benslimane et al, 1998) were conducted to experimentally assess: (i) the effect of the group and network system parameters on the load transfer mechanism, displacement, and resisting forces in the micropiles; such paramters include; pile inclination, spacing to diameter ratio, relative stiffness between micropile and the surrounding soil, and micropile configuration; (ii) current approach to derive soil-micropile interface parameters under seismic loading including lateral p-y curves and axial forces for inclined piles; (iii) characterization of lateral loads imposed on micropile systems due to base shaking; (iv) available pseudo-static methods for engineering analysis of the seismic behavior of micropile groups and network systems.

The detailed design of the model tests and their instrumentation was described by Benslimane et al, (1998). This paper briefly outlines the testing program and the experimental procedures used to carry out the centrifuge model tests, summarizes the effect of the main design parameters on the observed seismic performance, and evaluates current analysis methods that can be adopted for the seismic design of micropile systems.

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 due to boundary conditions.

PILE PROPERTIES AND MODEL LAYOUT

The structural characteristics of the model piles 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 bond and confining effects (Juran et al, 1998 ; Benslimane et al., 1998, 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). Pile slenderness ratios of 22 and 32 were selected to ensure that pile tip reaction did not significantly influence the pile head response. Based on these considerations, a scaling factor of 20 was adopted for the centrifuge model tests. The model piles were constructed of polystyrene with roughened shaft and a Young's modulus of Em = 2700 MPa and a length of L=21.3 cm. The Young's modulus of the model piles has been determined after computing the pile flexural rigidity EmIm from measurement of pile deflection versus applied transverse load using static beam deflection theory. 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.

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 was adopted for pile group tests.

INSTRUMENTATION AND DATA ACQUISITION

As shown on Figure 1, 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).

SOIL PROPERTIES

A Nevada sand 120 was used at a relative density of 57 %. Tests conducted on the sand (Arulmoli et al., 1992) yielded maximum and minimum void ratios of 0.51 and 0.88, specific gravity of 2.67, an average particle size D50 of 0.13 mm and a coefficient of uniformity (D60/D10) 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 tested relative density of 57%.

The influence of average grain size (D50) 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/D50<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.

EXPERIMENTAL TESTING PROCEDURE

The dynamic tests consisted of horizontally shaking the models while in flight at 20 g. For each configuration, the horizontal shaking included sequences of 100 cycles of sinusoidal accelerations at 2Hz (Prototype). 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 exciting frequency was constant for all micropile configurations to insure that micropile systems behavior is far from resonance. This procedure is found to be reliable (Gohl, 1991) in investigating the interaction parameters for pile groups. 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 advantage of this testing procedure is that the effect of the loading level on the seismic behavior of the model micropile configuration being tested can be established. Furthermore, according to the records of pile cap settlement and ground surface, the overall volume change of soil during shaking was relatively small suggesting that the change in void ratio was insignificant.

The procedure for model preparation was as follows: first a latex membrane 0.02 cm in thickness was used to line the inside of the laminar box, prevent leakage of the contents, and reduce the side friction. During model construction, the external side of the container was sealed and connected to a vacuum pump to remove air from the outer chamber. Dry sand with relative density of 57% was poured in layers of 50 mm thickness or less. The amount of sand needed for each layer to achieve required density was weighed. At appropriate stages, transducers (accelerometers and lvdt's) were placed in the soil. The main parameters of the experimental program are summarized in Table 2.

ANALYSIS OF THE CENTRIFUGE TEST RESULTS

The interpretation procedure adopted to analyze the centrifuge test 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 (Gohl, 1991).
- Investigation of the group and network effect on the bending moments, displacement, and axial forces developed in the micropiles under seismic loading.
- Analytical simulations of the test results using the finite difference computer program LPILE (Reese and Wang, 1989) and evaluation of the computer code GROUP (Reese and Wang, 1994) through a pseudo-static analysis of the test results.

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 into the frequency content and response characteristics. In the case of these tests, spectra were calculated for each of the accelerometers (input, ac5, 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) in the investigation of the seismic behavior of piles and soil nailed retaining structures using centrifuge modeling.

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 2 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). For all these cases, the micropile system was subjected to the cap loading only.

The results clearly show that, for a specified level of shaking (a/g=0.3) and input frequency (2 Hz): (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 parameters 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:

$$M = EI\frac{d^2y}{dz^2} = EI\frac{\varepsilon}{h} \quad (1)$$

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 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.

As 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, a good agreement was generally observed.

Cyclic p-y curves were computed for the same test at low level vibrations during the shaking cycle where maximum pile deflection occurred and are shown for a range of depths in figure 3. 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.

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 depths. Figure 4 compares the computed p-y curves under strong and low level shaking 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 $EI = 172$ MN.m^2. 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 for low level shaking the secant lateral stiffness corresponds fairly well to the results obtained by Gohl (1991). For strong shaking, the API and the Gohl (1991) p-y curves are considerably stiffer compared to the experimental p-y curves derived from the centrifuge tests on model micropiles.

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 nearly follows the motion of the pile (Juran et al., 1998). The inertial loading at the pile head was estimated based on the recorded pile cap accelerations and the input base motion.

Considering test (I-203), several iterations were done to define the fixity conditions at the pile head (moment and shear force) and full fixity 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 5a and 5b illustrate that the LPILE simulations using the experimentally derived p-y curves agree fairly well with the pile bending and displacement profiles obtained using Ting's procedure. This comparison also indicates that for micropiles, the API p-y curves lead to much stiffer soil response as compared with the experimental results.

PARAMETRIC STUDY:

Bending moment profiles for the instrumented micropile in the various configurations for each event were derived to investigate the group effect. The maximum relative pile displacement with respect to the free field and the associated distributions of the bending moment profiles were derived for (i) single pile (D=0.13m), (ii) 2x1 and 2x(2x1) vertical pile groups with D=0.13m, (iii) (2x1) inclined pile group with a 30 degrees pile inclination and D= 0.13 m, (iv) 3x(2x1) group with a pile inclination of 10 degrees and D=0.13m; (v) 3x(2x1) network system with a 30 degrees pile inclination and D= 0.13 m, (vi) (2x1) vertical pile group with D=0.19m. Test configurations were subjected to a vertical loading equivalent to 50 and 90 percent of the static failure load (0.5Fl and 0.9Fl). Computed lateral pile cap displacements were compared with measured pile cap displacements Based on the experimental results, a parametric study was conducted in order to assess the effect of the main parameters on the dynamic performance of micropile systems.

Effect of Slenderness Ratio (L/d)

The effect of slenderness ratio (L/D) is illustrated on figures 6a and 6b which display the experimentally derived pile bending and displacement profiles for single pile (Test configuration 1 and 2) and 2x1 vertical pile groups with a spacing to diameter ratio of 3 (configurations 7 & 9) where the pile diameter varied from 0.13 m to 0.19 m which for the selected pile length, is equivalent to pile slenderness ratios of 32 and 22 respectively. These test results were obtained for a superstructure loading of 0.9 Fl, and an amplitude of input base shaking of 0.3 g.

The experimental results illustrate that i) bending moment and displacement profiles for single piles and 2x1 vertical pile group configurations with s/D = 3, under the same equivalent loading conditions are quite different, illustrating the occurrence of a group effect for the selected spacing to diameter ratio. ii) The bending stiffness significantly affects the bending moment and displacement magnitude of the pile group. The bending moment obtained for the pile diameter of 0.19 is shown in figure 6a as a normalized product of measured bending moment times the ratio of I_1/I_2 where I_1 and I_2 are respectively, the pile inertia modulus corresponding to pile diameter of 0.13m and 0.19m. The normalized bending moment profile corresponds fairly well to the measured bending moment in 0.13m diameter piles, indicating that bending moment is practically proportional.

Effect of Spacing to Diameter Ratio

Similarly, the effect of spacing to diameter ratio (s/D) is illustrated on figure 7a and b which displays the experimentally derived pile bending and displacement profiles for (i) single pile, (ii) 2x1 pile group configuration with s/D =3 (test 7) and s/D =5 (test 11), and 2x2 pile group (test 10). All the selected configurations are tested under 0.3 g, and subjected to 0.9 Fl superstructure loading.

The experimental results illustrate that: i) bending moment and displacement profiles for single piles and 2x1 vertical pile group configurations with s/D = 5, under the same equivalent loading conditions are fairly close, illustrating a negligible interaction effect. (ii) For the selected frequency of excitation, the experimental data illustrate a ''positive'' group effect, which results in smaller bending moments and displacements for the spacing to diameter ratio of 3 as compared with data measured for identical single pile and 2x1 pile group with s/D=5. Due to a "Frame action" the 2x2 vertical pile group configuration appears to present a significantly higher seismic resistance with smaller bending moments and cap displacement as compared with 2x1 pile group configuration.

Effect of Pile Inclination:

The effect of pile inclination on the seismic behavior of selected micropile configurations is illustrated on figure 8 for the case of a superstructure loading of 0.5 Fl. The results clearly illustrate that pile inclination results in smaller bending moment and displacement magnitude of the selected 2x1 and 3x(2x1) pile systems as compared to vertical single and 2x(2x1) pile group systems. It is worth noting that for a pile inclination of 30 degrees, the bending moment is mainly due to pile cap restraint rather than soil pile interaction.

GROUP EFFECT

The experimental data illustrated a "positive" group effect for the selected frequency of excitation, which results in smaller bending moments and displacements of pile groups with s/d of 3 compared with data measured for s/d of 5. Similar observations were reported by Ousta (1998) who conducted three dimensional finite element simulations on different configurations of vertical micropile group systems within the French FOREVER research program. These studies were limited to the kinematical loading and were conducted for different vibration modes. It is worth noting that dynamic pile group analysis for conventional piles in elastic and visco-elastic media are generally associated with a negative group effect. (i.e. Dobry and Gazetas,. 1988 ; Velez et al., 1984). However, the centrifuge test results 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.

The observed positive group effect for the studied micropile configurations may be related to the following: (i) radial compression of sand that occurs between individual micropiles as a result of the increase in lateral stresses induced by pressure grouting during installation, that was simulated through the centrifuge model studies as described elsewhere (Juran et al, 1998); (ii) For flexible small diameter micropiles, the relative soil-micropile movement required to mobilize the ultimate lateral pressure is sufficiently large, in relation to the diameter of the micropile, to allow for its bending resistance to be mobilized, the lateral capacity for single micropiles is primarily dependent upon their moment capacity, however, moment capacity of the micropile group system is significantly dependent upon the cap and superstructure displacement and rotation which, through a "frame action", affect the distribution of lateral and axial forces mobilized in the individual micropiles.

NETWORK EFFECT

Bending and Displacements

The experimental data for 2x1 and 3x2x1 networks with 10 and 30 degrees batter piles displayed on figure 8 illustrate that the bi-dimensional network effect results in a decrease of the bending moments as compared with vertical pile groups as well as in further reduction in lateral pile cap displacement as compared with 2x1 and 3x2x1 vertical groups. Figure 9 illustrates the effect of the loading and shaking level on the bending moment profile during Test 7- p3 (2x1 - s/D=3 - alpha=30 degrees). For the case of battered pile with pile inclination of 30 degrees, the measured bending moment for different levels of acceleration are relatively small as compared with vertical pile systems, except at the pile head. These results indicate that bending moment are due mainly to boundary conditions/pile cap restraint rather than lateral soil/micropile interaction.

Axial Load transfer

Figure 10 illustrates the effect of superstructure loading and shaking level on the axial load transfer for 2x1 groups of battered micropiles inclined at 30 degrees and 3x(2x1) inclined at 10 degrees. The experimental results indicate that, for the case of cap loading, the maximum axial forces develop at the pile head illustrating the effect of the amplification of the input base motion towards the ground surface. Measured pile cap acceleration records indicate that for this case, the amplification of the input motion is about 100 %. However, no amplification of the input base motion was observed in the case of a superstructure loading of 0.5 Fl with the measured pile cap accelerations being practically equal to the input base acceleration. Comparing the recorded axial forces for 0.5 Fl and cap loading only, it appears that for the case of the battered piles at 30 degrees, the superstructure loading of 50% Fl results in a decrease of the maximum dynamic axial forces. This reduction can be related to (i) increase of the natural frequency due to pile inclination as illustrated in figure 2 using the spectral Fourier analysis of the recorded acceleration time histories. (ii) Stiffening of the micropile system which is clearly illustrated by the significant decrease of pile cap lateral displacement observed for the case of 0.5 Fl. These results are consistent with the experimental results reported by Prevost and Scanlan (1983), indicating that the superstructure loading effect results in a stiffening of the system, increase in natural frequency of the soil micropile system and therefore, in a significant decrease of the amplification of the input base motion towards the ground surface. The results obtained for the 3x(2x1) micropile network system with a pile inclination of 10 degrees subjected to 0.3 g are quite similar to those observed for the 2x1 battered micropile system under same acceleration levels.

A preliminary estimate of the dynamic axial forces developed in the piles during simulated seismic motion can be obtained from a pseudo static analysis of the cap–superstructure motion through the seismic events. Figure 11 illustrates the free body diagram of forces acting on (i) 2x1 vertical pile group (Case A), (ii) 2x1 inclined pile system at α=30° (Case B), and (iii) Intermediate Case of 2x1 pile system with α=10° (Case C) under in-line base shaking. For Case A, the equation of moment of equilibrium of the freestanding portion of the pile group is given as:

$$m.\,(a/g).\,z - (P_1.s/2 + P_2.\,s/2) = M_1 + M_2 - I_{cg}.\,\theta_c \quad (2)$$

For Case B and Case C, The equation of moment equilibrium is given as:

$$m.\,(a/g)\,z - [P_1.(\,s/2\text{-}l\tan\alpha) + P_2.(\,s/2 - l.\tan\alpha)]\cos\alpha = M_1 + M_2 - I_{cg}.\,\theta_c + [V_1.(\,s/2 - l.\tan\alpha) + V_2.(\,s/2 - l.\tan\alpha)]\sin\alpha \quad (3)$$

Where m is the pile cap and/or structural mass, a is the recorded acceleration at the center of gravity of the mass, g the gravitational acceleration, z is the distance of the center of gravity to the ground surface, *l* is pile cap stick up above ground surface, s is the pile spacing measured at the level of the pile cap connection, α is the pile inclination with respect to the vertical, I_{cg} the mass moment of inertia with respect to the center of gravity of the structural mass for the direction of shaking, M_1 or M_2 is the average bending moment per pile at the soil surface, P_1 or P_2 is the dynamic axial load in each pile, V_1 or V_2 is the dynamic shear forces, and θ_c is the angular acceleration of the mass.

Table 3 illustrates that, for the considered basic reference cases, the estimated axial forces agree reasonably well with the experimental data. This simple procedure, which has been adopted for preliminary test result interpretation, seems to illustrate that the experimental scheme yields a consistent description of the observed centrifugal model behavior. However, this analysis presents limitations which are related to the simplifying assumptions with regard to the dynamic soil-pile interaction, the effect of the kinematical forces induced by the free field motion, and the assumed distributions of moments an axial forces. Further experimental and numerical studies are presently being conducted for development and evaluation of an analytical approach for the seismic performance evaluation of micropile groups and reticulated network systems.

EVALUATION FOR SIMPLIFIED PSEUDO-STATIC ANALYSIS APPROACH

The main objective of this research project was to establish an experimental data base for the assessment of the dynamic performance of micropile systems, investigate the effect of the main system parameters on the observed seismic performance, and evaluate current analysis methods that could be used for the seismic design of micropile systems. For this purpose, the computer program GROUP was used to simulate the experimental results from a quantitative point of view.

Analysis Procedure:

The GROUP program has been well accepted as a useful tool for analyzing the behavior of piles in a group subjected to both axial and lateral loading. The program was developed (Matlock and Reese, 1966; Reese and Wang, 1994) to compute the distribution of loads (vertical, lateral, and bending moment) from the pile cap to piles in a symmetrical group. The piles may be installed vertically or on a batter and their head may be fixed, pinned or elastically restrained by the pile cap. The cap may settle, translate, and rotate and is assumed to act as a rigid body.

It is recognized that with regard to earthquakes a rational solution should proceed from the definition of the free field motions of the near surface due to the earthquake. As mentioned early, the kinematical loading can be neglected in the case of the tests presented and an approximate pseudo-static solution has therefore been used by applying a horizontal inertial load to the superstructure to represent the effect of the earthquake. For this analysis, the experimental p-y curves derived using the numerical scheme of Ting's were used in order to take into account the dynamic soil-pile interaction effect. The maximum unit skin friction derived from the axial load transfer was used to characterize the axial interface, and the recorded amplified cap acceleration was used to define the pseudo-static loading. Table (4) summarizes the values of the soil lateral stiffness k modulus experimentally derived. Based on these values a limiting upper and lower boundaries were defined in the range of $k = 1170 - 5128$ kN/m^3.

Figure (12) compares the experimentally derived bending moment profile obtained for test configuration P3 (2x1 group; α=30°) under cap loading only and 0.3 g base acceleration with the GROUP simulations. The experimental results agree fairly well with the theoretical predictions using GROUP in the range of the experimentally derived k values. Similarly, the case of a superstructure loading of 0.5 Fl is displayed on figure (13). A reasonable agreement between the experimental results and the GROUP predictions of the distribution of maximum bending moments is observed

Using the group code, the parametric effect of the pile inclination on the bending moment at the pile head and the lateral pile cap displacement of the 2x1 pile group system is shown on figure 14 and 15 respectively. The results obtained for the bending moment agree fairly well with the experimental data, while the results obtained for cap displacement appear to underestimate the measured data for vertical pile groups.

Both the experimental results and GROUP predictions indicate that pile inclination will result in a significant increase of bending moments at the pile cap connection. This comparison between experimental results and GROUP predictions also indicate that simplified pseudo-static analysis approach could be used for parametric performance assessment and preliminary seismic design provided the soil/pile interface properties defined from using experimentally derived p-y curves. However, as shown in figures 17a & 17b, for vertical micropiles systems while the experimental bending moment profiles illustrate a positive group effect, GROUP predicts a negative group effect, this is mainly related to the assumptions of an elastic media.

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 bending 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 significantly affected by the micropile system configuration. Based on the spectral Fourier analysis conducted on the selected configurations under cap loading and subjected to a prototype input base motion of 0.3 g amplitude, a value of 1.1 Hz was 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 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 low level shaking were found to be non-linear with low damping. Similar results were found for strong shaking with signs of gapping at shallow depths.

(iv) The experimental data illustrate a ''positive'' group effect for the selected frequency of excitation, 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.

(v) Network configuration tends to resist earthquake loading resulting in higher axial stresses compared with vertical groups, while there is a considerable reduction in pile bending moments and cap displacements. The superstructure loading effect results in a stiffening of the system, increasing in natural frequency of the soil micropile system and therefore, in a significant decrease of the amplification of the input base motion towards the ground surface.

One of the major objectives of this study was to evaluate the applicability of current analysis methods to predict the seismic behavior of micropile groups and networks systems. For this purpose, the computer program LPILE & GROUP has been used in a pseudo-static analysis approach to simulate representative centrifuge model tests.
The comparison between the experimental results and the group prediction showed in particular the following:

(i) Typical to Winkler models in a continuum elastic media, the GROUP program predicts a negative group effect. Therefore, theoretical predictions are not consistent with the positive group effect observed in micropile systems in the present study as well as by other authors.

(ii) The Group program predicts fairly well the effect of pile inclination on the bending and pile displacement profiles during the dynamic event. It is also worth noting that both the experimental results and GROUP predictions demonstrate that pile inclination will result in: (i) decrease of the pile cap displacement; (ii) increase in axial forces; (iii) increase in bending moment at the pile cap connections which strongly suggest the use of flexible connections for micropile systems

The present study has resulted in the creation of a significant database relating to the response of single, groups, and one-dimensional strain plan networks of micropiles to simulated earthquake excitation. Testing methods and interpretation procedures have been presented. However, these preliminary results need to be further investigated in order to develop reliable seismic design guidelines for micropile systems, and parametrically assessed for the complex soil-pile-superstructure interaction under seismic loading for micropile groups and network systems.

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ɔle 1. **Model micropile properties**

ɿutside ameter ˈ(mm)	Inertia Modulus I_m (mm^4)	Flexural Rigidity E_mI_m ($N.mm^2$)	Slenderness Ratio-L/D
9.5	$3.99.10^2$	$10.78\ 10^5$	22.4
6.5	$0.87\ .\ 10^2$	$2.36\ 10^5$	32.8

ɔle 2. **Main parameters of the experimental program**

Ⅵicropile nfiguration	Diameter (Prototype) D (m)	Slenderness Ratio L/D	Micropile Inclination α (deg)	Spacing to Diameter Ratio s/D	Ampl. of Acc. (Prototype) (a/g)	Loading Level
Single Ⅵicropile	0.19 0.13	32 22	0 0	- -	0.03 to 0.5	100 cycles- cap only 100 cyc. - 0.5 Fl 100 cyc. - 0.9 Fl
Ͻroup of Ⅎicropiles 2x1	0.13 0.19	32 22	0 0-30	3 – 5 3 – 5	0.3 until failure Same	100 cycles- cap only 100 cyc. - 0.5 Fl 100 cyc. - 0.9 Fl Same
Ͻroup of Ⅎicropiles (2x2)	0.13	32	0	3 – 5	Same	Same
3x(2x1)	0.13	32	10 - 30	3	Same	Same
x3x(2x1)	0.13	32	10 - 30	3	Same	Same

Table 3. Comparison of recorded axial forces with the estimated axial forces using the mome equilibrium of the free body diagram for the free standing portion of 2x1 pile grou subjected to an in-line shaking for the considered basic reference cases.

Reference Case	a/g	Loading Type (Cap / Superstructure)							
		Cap Only				Cap +50 % Failure Load			
		m. a/g (kN)	M_0 (kN.m)	Measured Exp. P(kN)	Estimated Eq [2-3] P (kN)	m. a/g (kN)	M_0 (kN.m)	Measured Exp P(kN)	Estimated Eq[2-3] P (kN)
CaseA	0.3					7.2	60	11	8.2
Case B	0.1	0.8	22	4.3	4	0.25	25	4	4.8
Case B	0.3	2.4	45	17.2	12.5	0.75	75	7.5	10
Case C	0.3					10.4	40	10	8.5

Nota m : Pile cap (with or without superstructure) mass
M_0 : Recorded pile bending moment near the ground surface. $M_0 = (M_1+M_2\text{-}l_{cg}.\ \theta_c)/2$
P : Recorded or calculated axial effort in the pile.
* : For test details, refer to Juran et al., (1998) (FHWA Report)

ble (4). **Summary of adopted values of the lateral stiffness k Modulus.**

ıt afig.	System Configuration	D (m)	s/D	α (deg.)	Loading Level	a/g	k (kN/m³)
	Single Pile	0.13	-	0	Cap Only	03	1170
	2x1	0.13	5	0	0.5Fl	0.3	1846.15
	2x1	0.19	3	0	0.5 Fl	0.3	2631
	2x1	0.13	3	30	Cap	0.3	3076.92
	2x1	0.13	3	30	0.5 Fl	0.1	5128.15
	2x1	0.13	3	30	0.5 Fl	0.3	4273.46
	2x2	0.13	3	0	Cap	0.3	3076.92
	2x2	0.13	3	0	0.5 Fl	0.3	2051.3
	3x(2x1)	0.13	3	10	0.5 Fl	0.3	2254

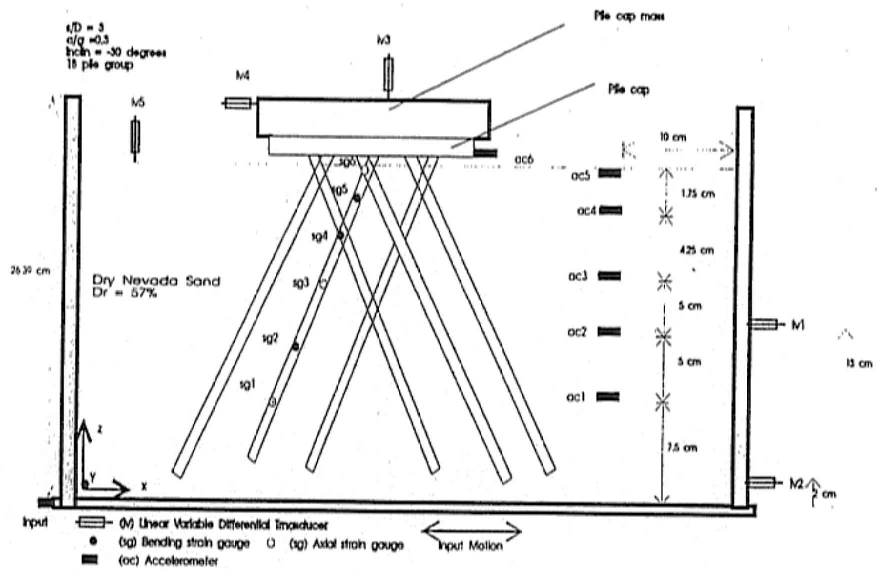
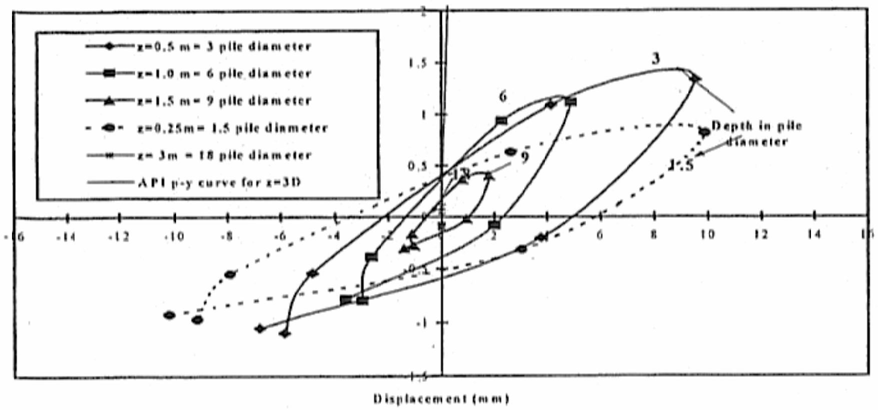
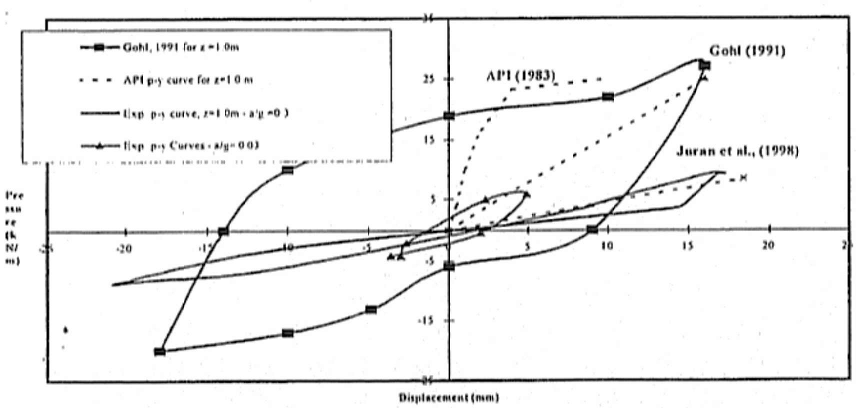


Figure 1. Typical micropile model system tested by Polytechnic University (Juran et al., 1998)



ure 3. Cyclic p-y curves at depths during steady state shaking cycle (t=30-30.5 sec) and comparison with API p-y curve – centrifuge test 1 (203) (a/g=0.03).



ure 4. Comparison of the experimentally derived cyclic p-y curves during steady state shaking cycle (t=30-30.5sec – centrifuge test 1-203 and P5) with API (1983) p-y curve, and Gohl, 1991

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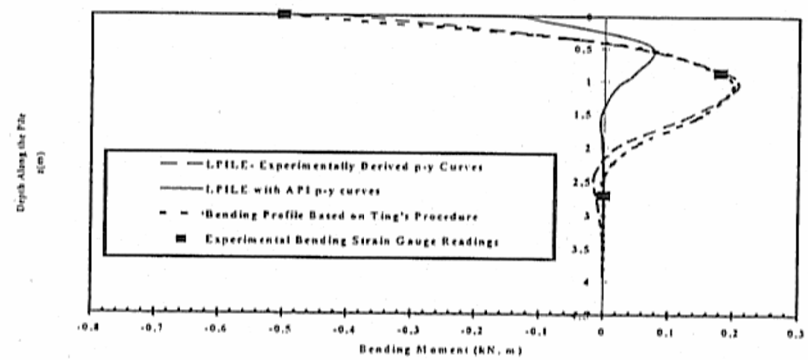
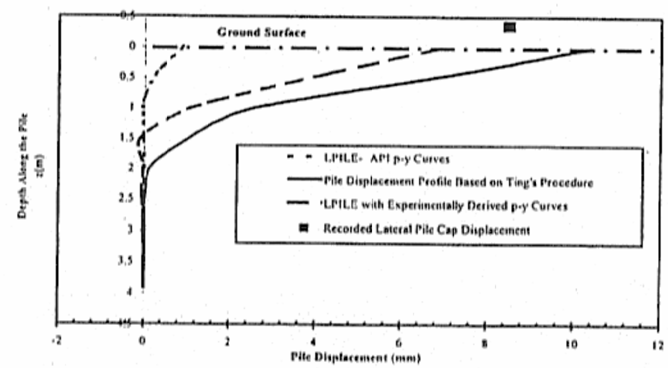
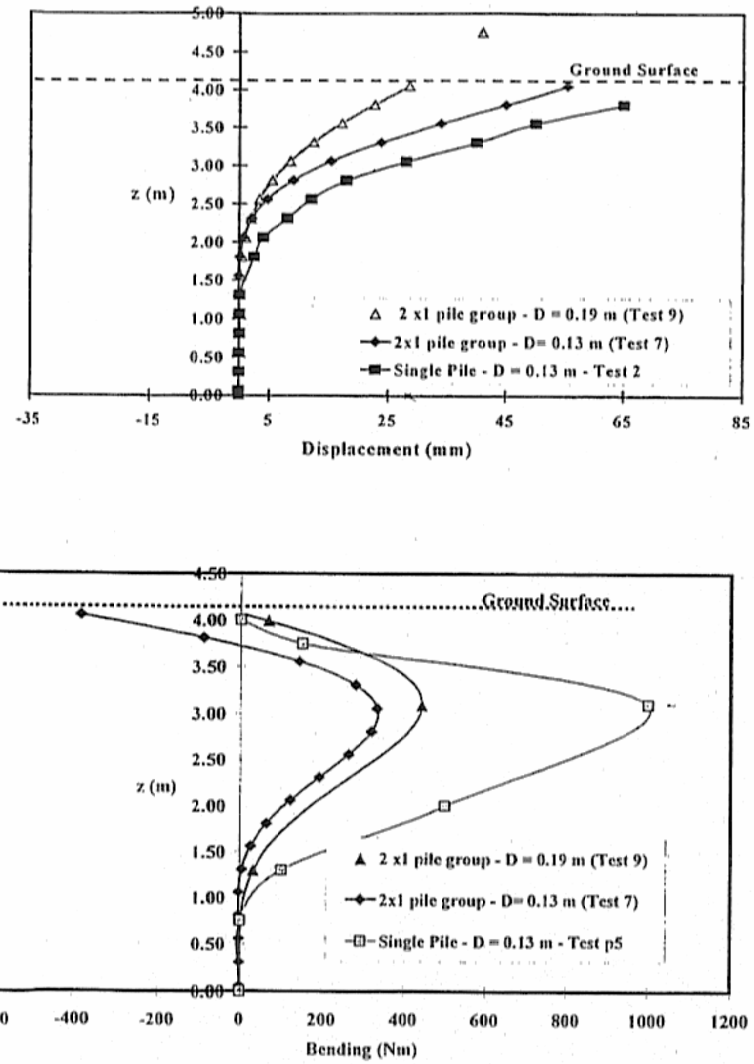


a)



b)

Figure 5. 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) correspondin displacement profiles Test 1 (203) - single pile- low level shaking (a/g=0.03)



·e 6. Parametric effect of the slenderness ratio (L/D) on the recorded a) displacement, and b) bending moment profiles of micropile systems (a/g = 0.3, s/D = 3, 0.9FL)

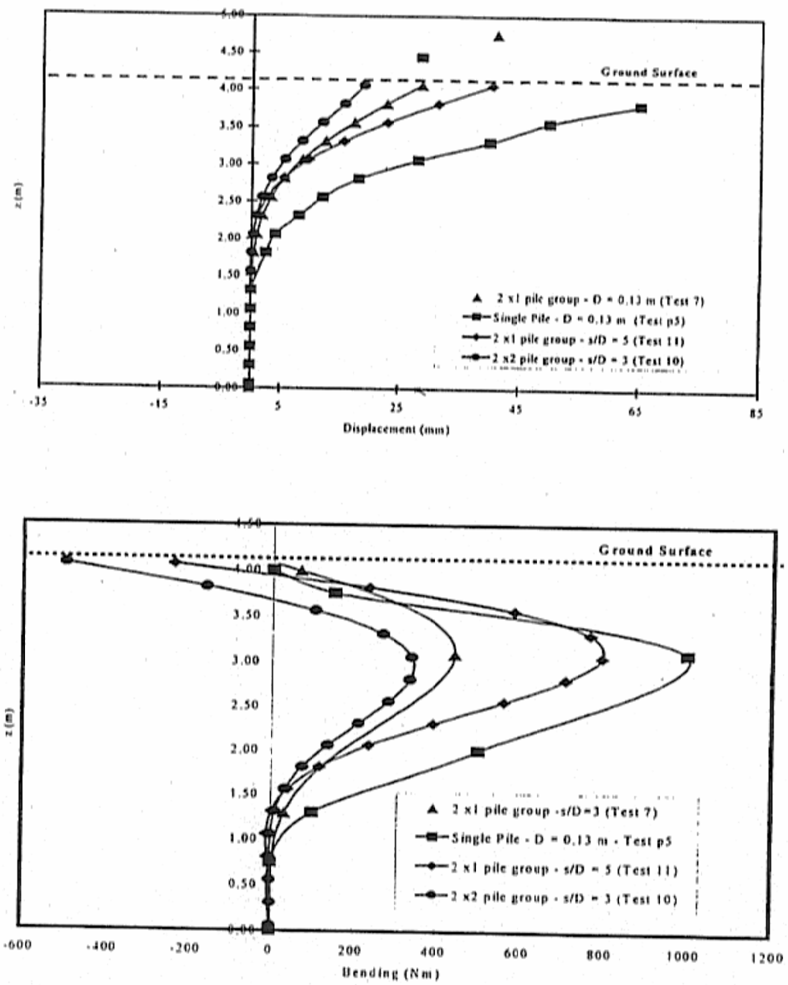
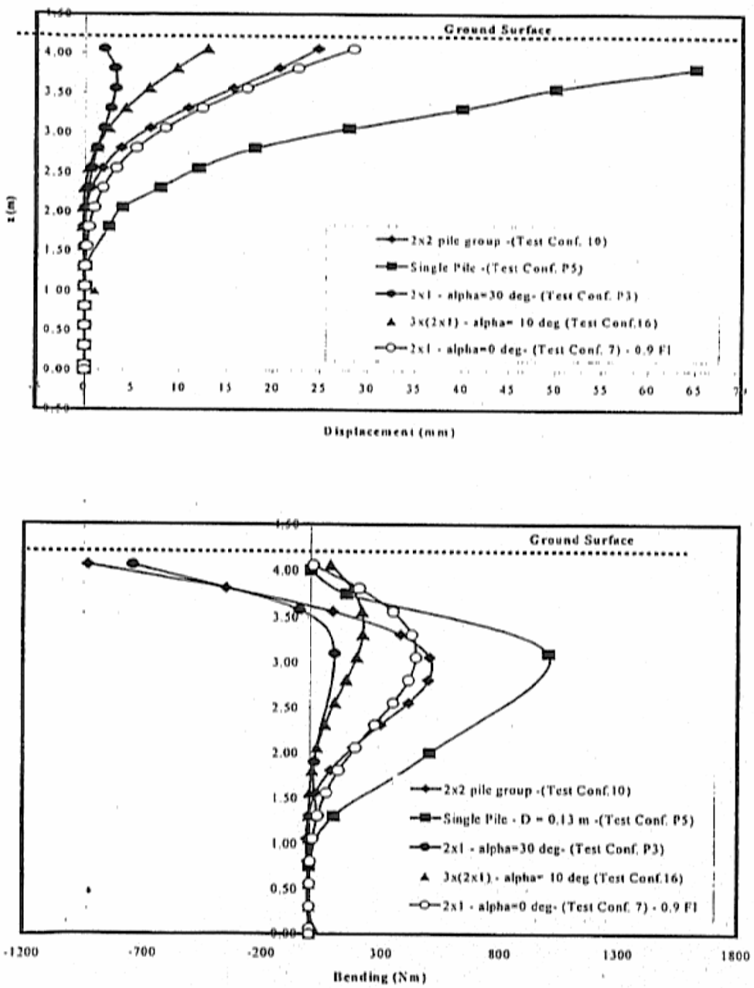


Figure 7. Parametric effect of the spacing to diameter ratio (s/D) on the recorded a) displacement and b bending moment profiles of micropile systems. (a/g = 0.3, D = 0.13m, 0.9FL)



ıre 8. Parametric effect of the pile inclination (α) on the recorded a) displacement and b) bending moment profiles of micropile systems. (a/g = 0.3, D = 0.13m, s/D = 3, 0.5FL)

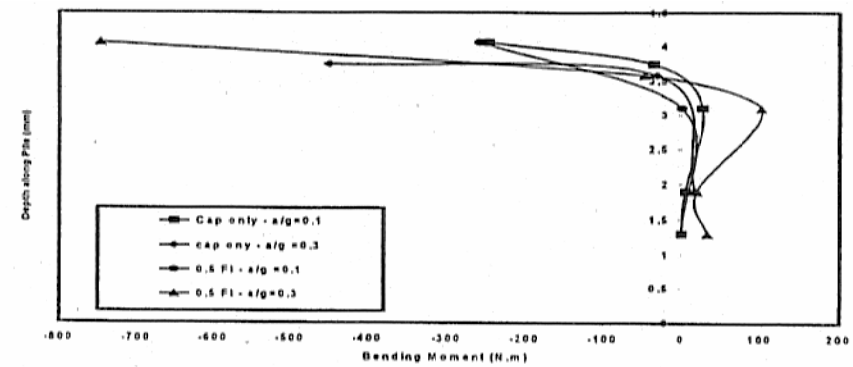


Figure 9. Effect of the loading and shaking level on the bending moment profile at peak pile displacem during Test 7- p3 (2x1- s/D=3 - α=30°).

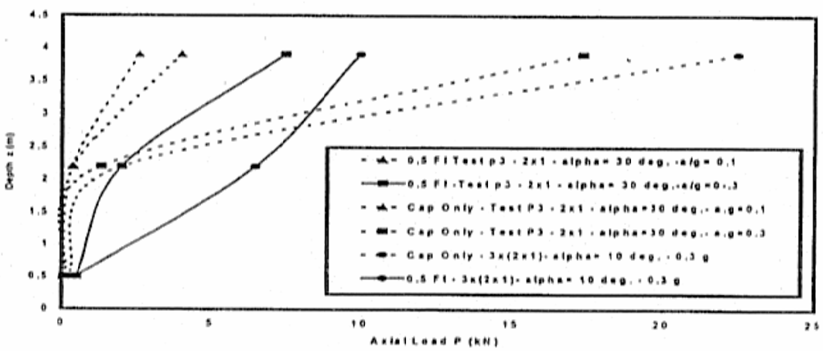
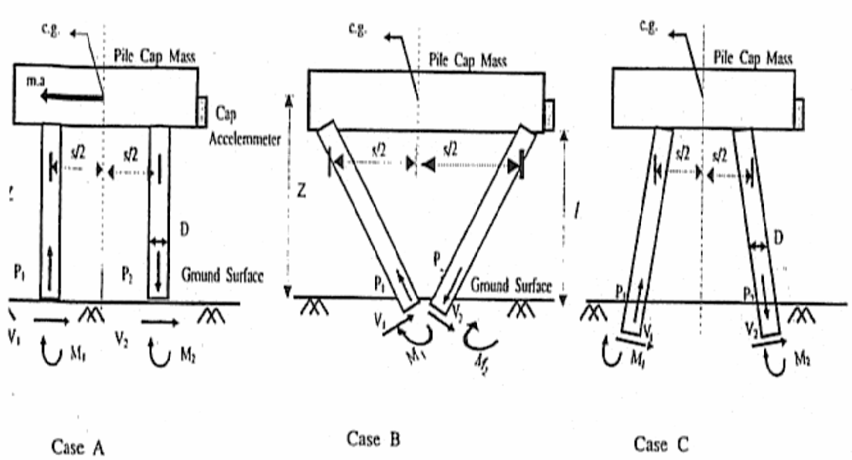


Figure 10. Effect of the loading level, shaking level on the axial load transfer in a 2x1 pile group inclined at α=30° (T P3) and 3x(2x1) network system (Test 16) with pile inclination of α=10° (L/D= 32-a/g=0.3)



ure 11. Free body diagram for the freestanding portion of 2x1 pile group subjected to an in-line shaking for the considered basic reference cases.

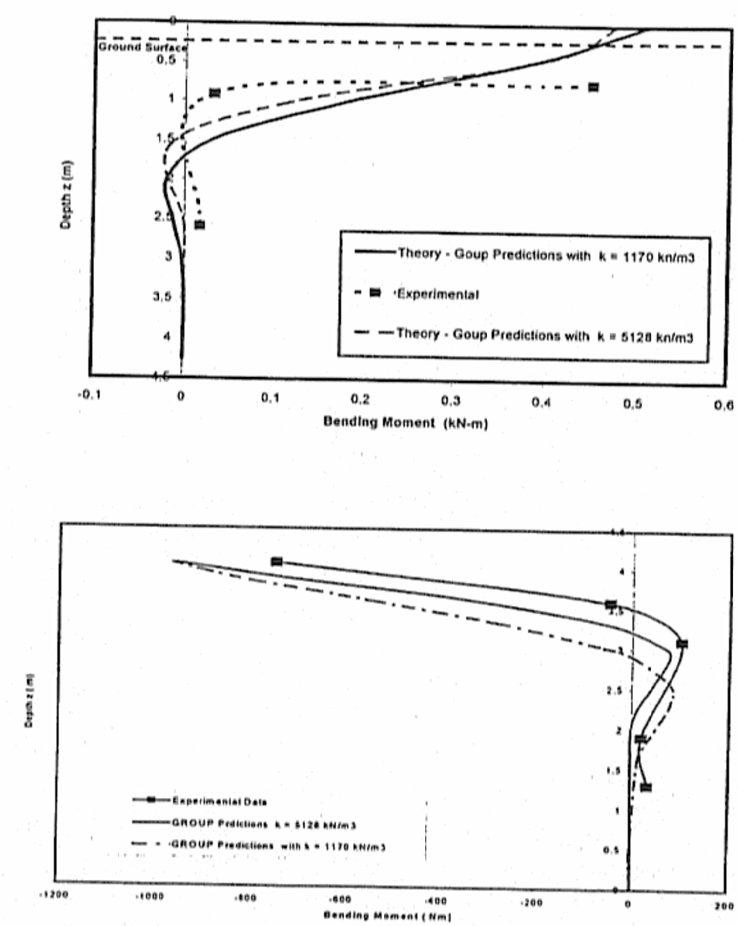
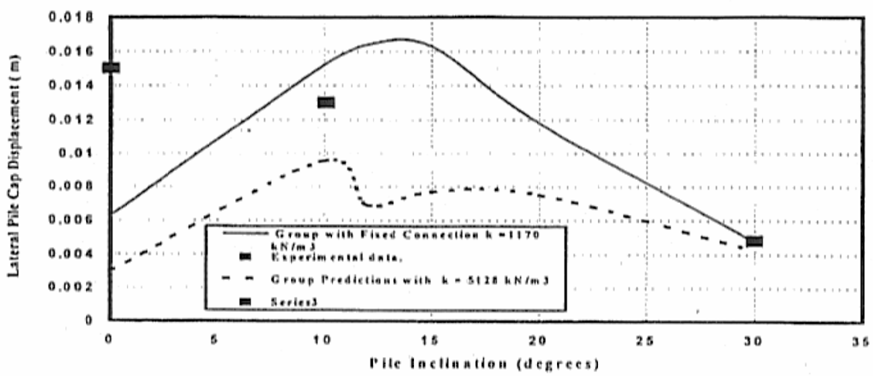
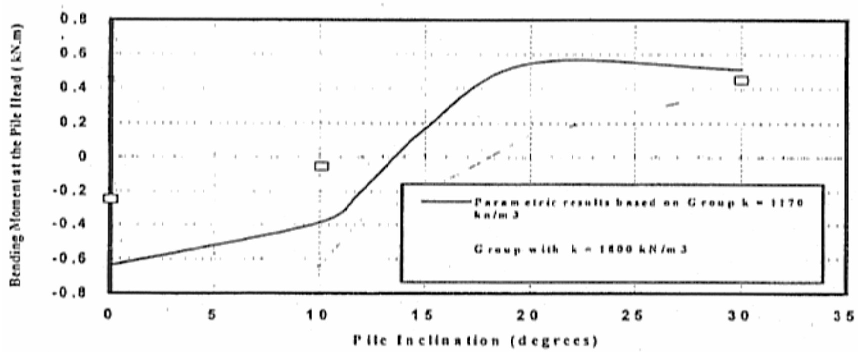


Figure 13. Comparison of the experimental results with the GROUP predictions of pile bending profiles of



ıre 14. Parametric effect of the pile inclination on the lateral pile cap displacement of a 2x1 pile group system (Cap only – a/g = 0.3)



ıre 15. Parametric effect of the pile inclination on the bending moment at the pile head of 2x1 pile group

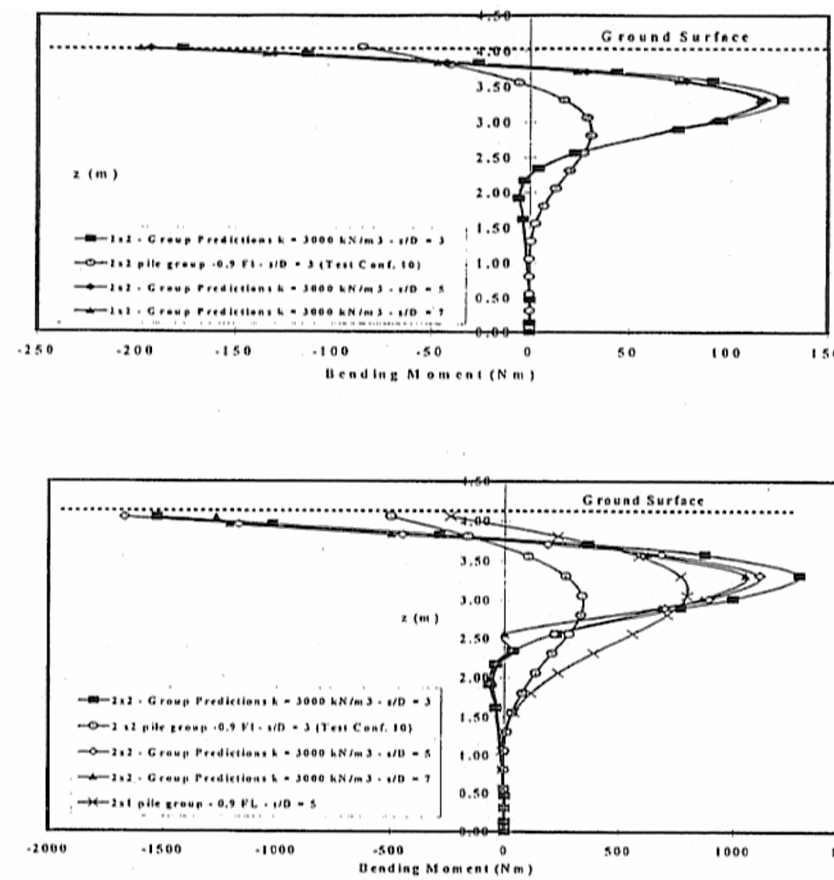


Figure 17. GROUP prediction of the parametric effect of s/D on the pile bending profiles for 2x2 vertical pile group and comparison with experimental data, a) cap only; b) 0.9FL (a/g = 0.3)