KINEMATIC SOIL-MICROPILE INTERACTION

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SUMMARY

Post earthquake observations of pile foundation failures have indicated that in many cases, damage was attributed to response to vertically propagating shear waves (Kavvadas et al, 1993, Mizuno,1987). In recent years, many researchers have developed theoretical models for the seismic design of pile foundation. However, very little laboratory data is available. To fill this gap, this paper studies the kinematic interaction between a micropile and its surrounding homogeneous soil deposit based on a series of laboratory tests.

The micropile model tests were conducted in a level, dry sand deposit, prepared in a laminar tank bolted to a shaking table. Base input motion comprised 20 to 50 cycles of sinusoidal waves with frequencies varying between 0.5Hz and 10Hz. The base shaking intensities were in the range of 0.063~0.42g. The actual pile response is then compared with that of three models: the standard dynamic beam-on-Winkler-foundation model, a simplified beam-on-Winkler-foundation model and the 'Pilate' model. After a series of comparative analyses between the computations and test results, several conclusions are able to be drawn, as summarised below:

With weak base shaking (<0.25g), the micropile follows exactly the motion of the soil. In this case, neglecting the soil-pile interaction and pile inertia effect gives a reasonable approximation in estimating peak bending moment distributions of the micropile. However, under strong base shaking (≥0.25g) it becomes difficult to make good predictions of bending moments using frequency domain methods, due to effects of non-linear soil behaviour. Fitting the deflected shape of soil layer with simple polynomial functions was also attempted. As a result of the analysis, it is concluded that a second order polynomial for low frequencies between 0.5 and 1Hz and a third order polynomial for 2.0 and 4.0Hz provide the best curve-fit to the test data and give best predictions of pile response. However, at a high frequency of base shaking, both the non-linear soil behaviour and the non-homogeneity of the laminar tank complicates the deflected shape of the soil layer.

The seismic behaviour of micropiles remains unclear. The results of this research should contribute towards a better understanding of micropile behaviours and an improved seismic design practice for micropiles.

INTRODUCTION

Post earthquake observations of pile foundation failures have indicated that in many cases pile damage was attributed to the seismic response of these piles to vertically propagating shear waves (Kavvadas et al, 1993, Mizuno,1987). In recent years, many researchers have developed theoretical models for the seismic design of pile foundations. Most of this work presents relatively simple methods to be used for design purposes rather than more rigorous analyses using finite element or boundary element methods. Oweis (1981) suggested a simplified procedure for estimating curvature from seismically induced soil strains where the pile is assumed to follow the horizontal ground motion produced by vertically propagating shear waves. Dobry and O'Rourke (1983)

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presented a simplified design chart for assessing bending moments in layered soil profiles. Here the maximum induced moment is given as a function of soil shear moduli ratios and the maximum shear stress at the interface. Flores-Berrones and Whitman (1982), Gazetas (1984), Kavvadas and Gazetas (1993), Nikolaou and Gazetas (1997) used beam on dynamic Winkler foundation models to compute the seismic response of end bearing piles subjected to vertically propagating shear waves. However, very few laboratory data are available to verify the assumptions used in these analyses. For instance, it has been suggested that, since flexible piles follow the ground motion, there is no need to modify the input excitation. Consequently, analysis procedures often make use of the same design response spectra as for structures on shallow foundations. In fact, the validity of this approximation has not yet been adequately verified (Flores-Berrones and Whitman, 1982 Gazetas, 1984).

To help solve this problem, this paper studies the behaviour of micropiles under simulated seismic loading using a series of laboratory model tests. In this research, three soil-pile interaction models of different assumptions are used to compute the pile response. They are the standard dynamic beam-on-Winkler-foundation model (BWF), the simplified beam-on-Winkler-foundation model (SBWF) and the ‘Pilate’ model. Subsequently, comparisons are carried out among the computations of those models. It has been shown that with weak base shaking (<0.25g), the micropile follows exactly the motion of the soil. In this case, neglecting the soil-pile interaction and pile inertia effect is a reasonable approximation for estimating peak bending moment distributions of the micropile. However, under strong base shaking ($\geq$0.25g), the effects of the non-linear soil behaviour significantly influence the pile seismic behaviour and the pile does not follow the motion of the soil any more. The soil-pile interaction cannot be neglected. Therefore, it becomes difficult to make a good prediction of micropile bending moments with frequency domain methods based on the soil response assumption of linear or equivalent linear.

Obviously, the seismic behaviour of flexible piles remains unclear. The results of this research should contribute towards a better understanding of flexible pile behaviour and an improved seismic design practice for flexible piles.

MICROPILE MODEL TEST OUTLINE

In order to study flexible pile behaviour due to soil deformation in shear, a series of micropile model tests were carried out on the University of Canterbury shaking table. A laminar tank was designed to produce shear deformations within a soil stratum as near as possible to that of the free field. It has internal dimensions of 1.8m long by 2.0m high by 0.9m wide. This gives the tank an aspect ratio of 1.11, which will produce a predominantly shearing response to base excitation.

A model micropile was inserted into the level dry sand deposit, which was prepared in the laminar container bolted to the shake table. The general arrangement of model tests is shown in Fig. 1.

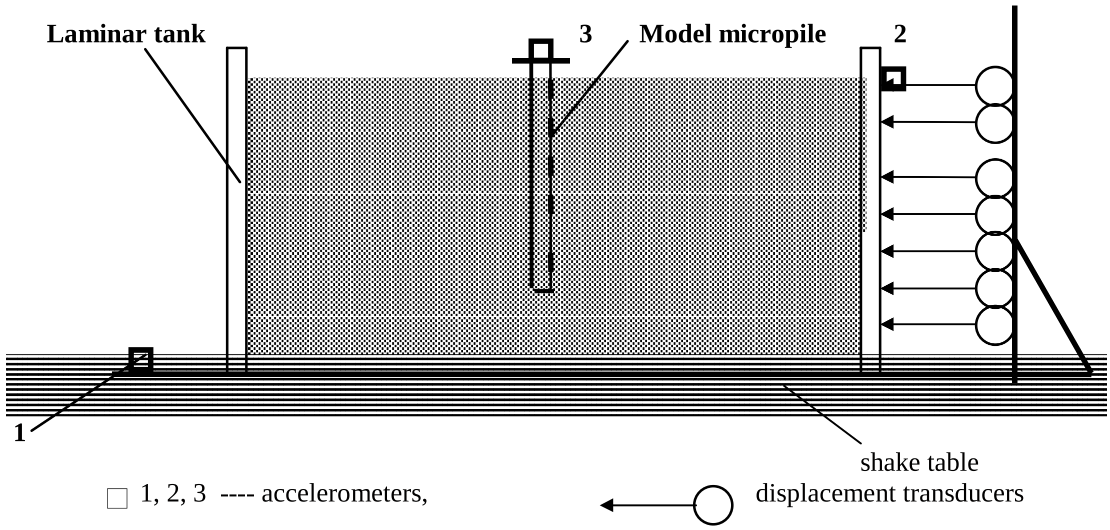

Figure 1 Cross Section of Test Setup

The soil chosen for use in the tests was an industrially prepared Grade 30/60 silica sand. It is suitable for repeated testing with little degradation. Air pluviation was used to prepare the soil deposit, which gave reproducible average sand deposit densities with a minimum of variation. When preparing a deposit, a tank full of sand was raised and poured into the container at a constant height above the sand surface in order to maintain a constant density (McManus and Chambers, 1995). An average density of the sand layer, γ_s, can be calculated based on the weight of deposited sand.

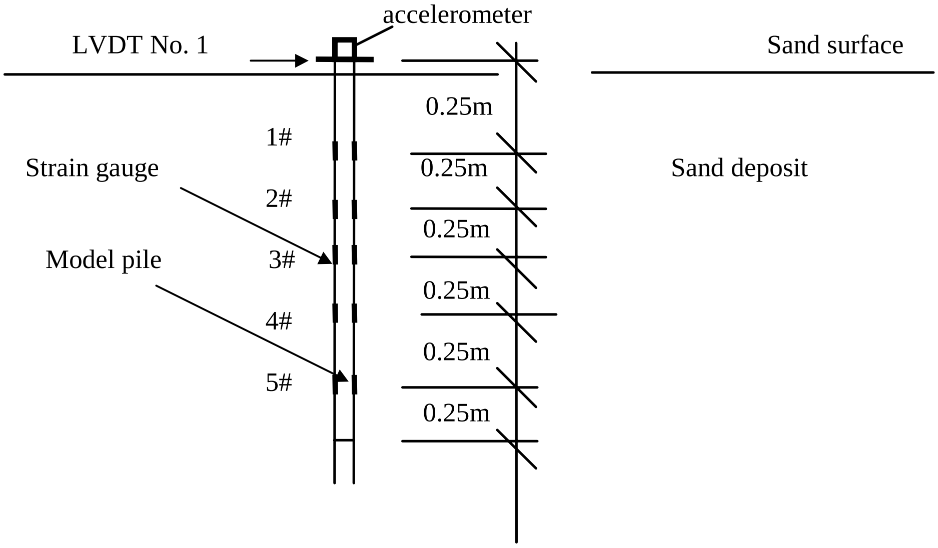

Figure 2 Location of the Instruments of the Model Pile

A linear potentiometer was fixed to the table to measure the table displacement and an accelerometer was fixed to the table to measure the base acceleration. An array of seven linear potentiometers was placed against the container to measure the sand deflections relative to the shaking table. The absolute displacements of the sand could be obtained by adding laminate and table displacements. The sand surface acceleration was measured by an accelerometer attached to the container at 100mm below the top of the initial sand surface.

The model pile was constructed of hollow aluminium tube having an outside diameter of 15.88 mm and a wall thickness of 1.44mm. The total length of the model pile is 1500mm. The strain gauges were placed on the both sides of the model pile every 0.25m depth from the pile top. Fig. 2 shows the location of the instruments of the piles. After the instrumented model pile was set in the position in the sand container, the sand was filled by means of a long flexible nozzle which was held approximately 20cm above the sand surface.

The flexural rigidity ($E_p I_p$) of the pile was measured using transverse dead loading of the pile while it was simply supported at both ends. From measurements of pile deflection versus applied transverse load, the pile stiffness $E_p I_p$ was computed to be 114.54Nm using static beam bending formulae. In the same way, the strain gauges on the pile were calibrated (Finn, W. D. L. and Gohl, W. B., 1987).

The spaced strain gauges on the model pile were used to measure bending and compression, from which the bending moment of the pile can be computed. A lightweight plate with a weight about 55mg, which was prepared for the installation of the accelerometer, was put on the top of the pile. Because of the small dimensions of the plate, the inertia moment for rotation of the plate about the centre of gravity of the mass has been neglected.

Sinusoidal vibrations are applied in the horizontal direction. The acceleration of the vibration was varied slightly keeping the table displacement within a reasonable range at different frequencies. After a series of cyclic tests on the sand deposit at 1.0Hz in loose sand, a dense sand deposit was achieved and the micropile model tests were conducted. The sand density was assumed to be constant at each depth, and an average density of 1570 kg/m^3 was computed by using the total weight over the total volume of the sand deposit. The average void ratio of 0.68 was achieved.

A series of micropile tests were carried out, in which the frequency of the excitation was 0.5, 1.0, 2.0, 4.0, 8.0 and 10.0Hz respectively. There were ten tests during the shaking, each of which was carried out at three different frequencies. At each frequency there were five repeated tests at the same intensities of shaking. All the test results were collected and recorded using a Burr-Brown Analogue to Digital dynamic data logger which was set to scan at 30 times per second per channel for shaking below 4.0Hz and at 100 times per sec. at higher frequencies. Each record consists of a series of 20-50 cycles of sinusoidal motion.

COMPUTER MODELLING

An analysis of soil-pile interaction in a seismic environment usually follows the two procedures: (i) the soil-pile interaction is modelled, and then (ii) free-field motion is applied at the soil-pile interface. The model used in the present study is based on the standard dynamic beam-on-Winkler-foundation model. It consists of the elastic pile model and a Winkler foundation soil model. Interaction between the pile and the free-field soil is usually taken into account by using linear or non-linear lateral compliances (springs and equivalent viscous dashpots) placed along the length of the pile. These compliances simulate the forces acting on the pile due to relative movement between the pile and the soil. This seismic analysis of pile foundations is made in the frequency domain, so the basic equations for the soil-pile interaction in the frequency domain could be written as:

$$E_p I_p \frac{\partial^4 U_p}{\partial z^4} + \left(K_x + iC_x\omega - \overline{m}\omega^2\right)U_p = \left(K_x + iC_x\omega\right)U_{ff} + \overline{m}\omega^2 U_g - P_0$$

The homogeneous solution is $U_p{}^h = e^{\lambda z}(A\cos\lambda z + B\sin\lambda z) + e^{-\lambda z}(C\cos\lambda z + D\sin\lambda z)$,

where, $\lambda = \sqrt[4]{\dfrac{K_x + i\omega C_x - \overline{m}\omega^2}{4E_p I_p}}$. The constants of integration, A, B, C and D may be determined from the compatibility conditions or boundary conditions.

The particular solution to the above equation depends on the function of the free-field move displacement. For a cubic relationship, the particular solution can be written as:

$U_p{}^p(z) = \Gamma\left(C_0 + C_1 z + C_2 z^2 + C_3 z^3\right) + \dfrac{\overline{m}\omega^2 U_g - P_0}{K_x + i\omega C_x - \overline{m}\omega^2}$, where the amplification factor is $\Gamma = \dfrac{K_x + i\omega C_x}{K_x + i\omega C_x - \overline{m}\omega^2}$.

For an exponential relation, Schnabel P. B., Lysmer, J. and Seed, H. B., 1972),

$u_{ff}(z,t) = U_{ff}e^{i\omega t} = (c_1 e^{iKz} + c_2 e^{-iKz})e^{i\omega t}$, the particular solution is:

$U_p{}^p(z) = \Gamma\left(c_1 e^{iKz} + c_2 e^{-iKz}\right) + \dfrac{\overline{m}\omega^2 U_g - P0}{K_x + i\omega C_x - \overline{m}\omega^2}$, where $\Gamma = \dfrac{K_x + i\omega C_x}{E_p I_p K^4 + K_x + i\omega C_x - \overline{m}\omega^2}$.

So the total solution can be obtained by simply adding these two parts.

When the soil deposit surrounding the piles is made of several layers with different properties, it can be assumed that the pile and the soil deposit are divided into a series of sub-layers of arbitrary length. It is assumed that the subgrade modulus remains constant within each sub-layer and compatibility conditions apply at the interface of two sub-layers. In the research presented here the other two conditions are also considered. When the soil-pile interaction at the interface is weak, the effect of soil damping and the pile inertia is assumed to have negligible effect on the seismic response of the pile. Basically, the standard beam-on-dynamic-Winkler-foundation model becomes a static soil-pile interaction model; we call this the 'Pilate' model after the well-known pile analysis programme of Frank and Romagny (1990).

In the extreme case for slender piles embedded in stiff soil sites, neglecting soil-pile interaction assumes that the stiffness of the piles has a negligible effect on the seismic response of the pile, and the pile response is computed from the free field soil displacements. Accordingly, the pile deformation is obtained by: $U_p = U_{ff}$. The pile then exactly follows the free field motion.

The computations of the above three models are included in the program SPI-97 written by the principal author; comparisons will be provided in the next section.

TEST RESULTS AND ANALYSIS

A series of tests under different shaking intensities at different frequencies were conducted to examine the effect of both the level of soil shear strain and of frequency. Fig 3 shows the location of the measured maximum pile bending moment associated with different frequencies and excitation levels. It can be seen that under low level

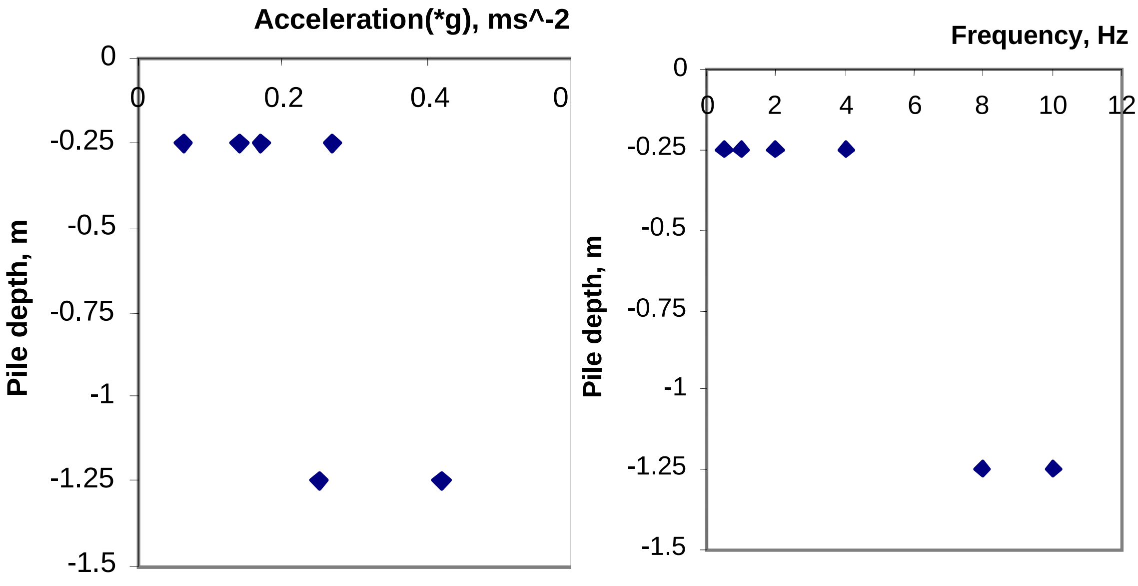

Figure 3 Location of Measured Maximum Micropile Bending Moment

shaking maximum bending moments occur near the sand surface, which indicates that the inertial effect plays an important part in pile bending during shaking. However, during the strong base shaking, the maximum bending moments appear near the pile bottom, which reveals that the pile bending is completely dominated by the deformation of surrounding soil. Therefore, the inertial effect from pile head can be ignored.

It also seems that the micropile seismic behaviour is not only influenced by the shaking intensities but also affected by the excitation frequency. A study of the effect of excitation frequencies is deferred for a future study and is not included in this paper.

Under weak base shaking between 0.063g and 0.17g at frequencies between 0.5 and 2.0Hz, the soil behaves linearly. Soil displacements are in phase at all depths and the frequency methods are valid. All three models provide good prediction of the micropile bending moments. Typical waveforms of micropile bending moment distributions are given in Fig. 4.

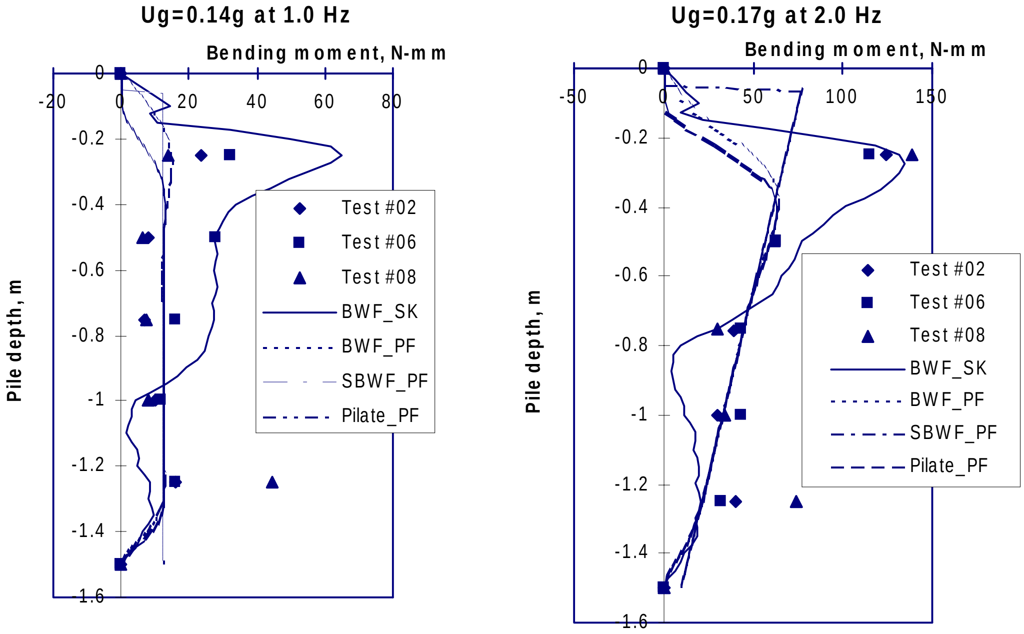

Figure 4 Typical Data of Computed and Measured Micropile Bending Moments

Fig. 4 shows that the results computed by all the three models with the same polynomial-function free-field motion are close, and all fit the test data quite well. The computation results are totally dependent on the shape function of the soil free-field motion. Therefore it is concluded that neglecting the soil-pile interaction and the pile inertia effect can give a reasonable approximation for estimating peak bending moment distributions of the micropile. Both the micropile inertia and stiffness have a negligible effect on micropile response. It can also be concluded from this study that under low level of base shaking a simple polynomial function provides the best curve-fit for the horizontal soil layer deflection. The BWF model with exponential relation (BWF_SK) can also provide close predictions to the test data. Good agreement between the computed values and test data indicates that the assumptions of the one-dimensional propagation model and the soil equivalent linear model hold under this level of shaking intensity.

With base shaking amplitude between 0.25g and 0.42g at frequencies between 8.0 and 10.0Hz, non-linear soil behaviour is likely to have a significant impact on the dynamic response of the soil. Given the dependency of strain on dynamic soil properties, increasing the soil base motion results in a decrease in soil stiffness and an increase in damping. These factors have the effect of changing the natural period of the soil deposit, and thus the response to the base motion. Moreover, the laminar tank might not provide uniform resistance under strong shaking at high frequencies, and so it likely that the soil deposit became inhomogeneous. Because of all these factors, it is difficult to make good predictions of the micropile bending moments with strong shaking. At 8.0 and 10Hz, these three models with both polynomial functions and exponential functions under-estimate micropile bending moments as seen in Fig. 5. Another reason for the poor match of predictions with the test data at high frequencies is that soil displacements may no longer be in phase. Part of the soil deposit may move in one direction while another moves in the opposite direction. Therefore, the frequency domain method may not be suitable for strong shaking or high excitation frequencies and a time history analysis is required.

It is important to match deflected shape of the soil layer for a given intensity of base shaking in order to achieve meaningful estimates of pile response. In theory, pile bending moments can be estimated from the soil displacements using a finite difference method by neglecting soil-pile interaction. However, the differentiation process may lead to serious errors unless extremely accurate free-field displacements are obtained. Therefore, some smoothing of the data would be required to obtain better results before performing such an analysis of pile response. In addition, continuous polynomial functions are valid only if the soil deposit is homogeneous. If it becomes inhomogeneous, piecewise continuous functions may give a better fit to the soil deposit shape.

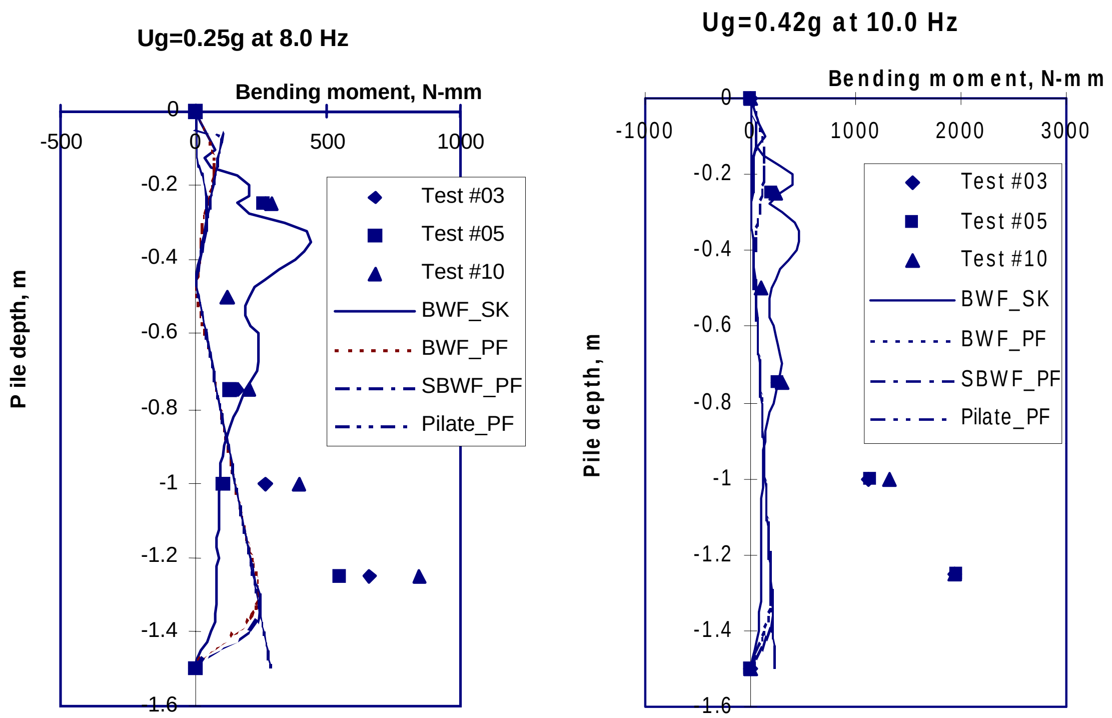

Figure 5 Typical Data of Computed and Measured Micropile Bending Moments

Comparing the results of those methods from Fig. 4 under low level base shaking, it was also found that the computed pile bending moments were not overly sensitive to the absolute values of soil moduli, but to its distribution. It can be also concluded that small variations in system damping do not have a large effect on computed pile response with weak shaking. The radiation damping coefficients are assumed to be constant in the dynamic pile analysis. The material damping ratios are estimated from the average amplitude of soil shear strain around the pile. It is considered that these two damping models are accurate enough for estimating the micropile response under weak base shaking.

SUMMARY AND CONCLUSION

Whether flexible piles always follow the ground motion is an unsolved problem in structural analysis and design. Current procedures often make use of the same design response spectra for piles as for structures on shallow foundations. However, the validity of this approximation has not yet been adequately verified.

The research presented here studies the kinematic performance of a single micropile based on the laboratory model tests. It is shown that under weak base shaking between 0.063g and 0.17g at frequencies between 0.5 and 2.0Hz, neglecting the soil-pile interaction and the pile inertia effect can give a reasonable approximation for estimating peak bending moment distributions of the micropile because the pile exactly follows the soil motion. Also, using the same design spectra as for structures on shallow foundations is a good approximation.

However, with a high level of base shaking, between 0.25g and 0.42g at frequencies between 8.0 and 10.0Hz, non-linear soil behaviour is likely to have a significant impact on the dynamic response of the soil. The interaction between pile and its surrounding soil then cannot be ignored. Also, use of the same response spectrum as for structures on shallow foundations will lead to underestimation of the pile bending moments. Further research should consider, among other problems, the effects of high level base shaking, higher excitation frequencies, and layered soil deposits.

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