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Centrifuge Analysis of Coupled Soil-pile-structure Interaction

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Synopsis

A series of systematic centrifuge tests is conducted in order to study the seismic response of end bearing single piles embedded in a dry dense sand layer and supporting single or double degree of freedom structures. The model is excited with and without structures using 12 sinusoidal waves as input base accelerations with constant amplitudes of about 1.5 m/s^2 and varying frequencies ranging from 1 to 12 Hz. All the tests are conducted with the centrifugal acceleration of 40G. The obtained results provide a clear insight as to the distinct role of inertial and kinematic interactions as well as the combined effect of these two interactions on the motion of the pile head by identifying the fundamental frequencies that dominate the response not only for single degree of freedom structure but for double degree of freedom as well. The role of the frequency content of the input motion on the development of kinematic and inertial dynamic pile bending is also investigated.

Keywords: Soil-pile-structure interaction, centrifuge, frequency domain, dynamic bending moment

1. Introduction

Knowledge of the main parameters that affect the seismic soil-pile-structure interaction (SSPSI) is of utmost importance to evaluate the performance of pile foundations during earthquakes leading to a safe design. The shortage of well-documented and well-instrumented full-scale case history data and post-earthquake investigations of pile failures motivates researchers to perform physical modeling through centrifuge and shaking table model tests as an alternative technique to augment the field case histories with laboratory data obtained under controlled conditions. The advantage of this modeling lies in the ability of the centrifuge to reproduce prototype stress-strain conditions in a reduced scale model as well as the ability to provide a relatively rapid method for performing parametric studies. In this article, a series of systematic centrifuge tests is conducted in the frequency domain in order to study the seismic response of end bearing single piles embedded in a dense dry sand layer and supporting single or double degree of freedom structures. A schematic view of the end bearing single pile supporting double degree of freedom structures is shown in Fig. 1. The scope of this study is threefold: (a) to examine the kinematic soil-pile interaction, (b) to investigate the combined effect of kinematic and inertial interaction on the motion of the pile head, by identifying the fundamental frequencies that dominate the response for both single and double degree of freedom structures, and (c) to study the role of the frequency content of the input motion on the development of pile bending. The test results of centrifuge are presented in terms of prototype unless otherwise stated.

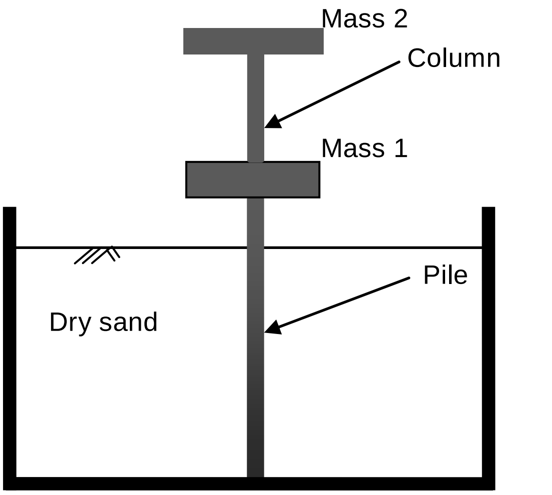


Fig. 1 A schematic view of end bearing single pile supporting double degree of freedom structure

2. Centrifuge facility at Disaster Prevention Research Institute, Kyoto University

The author carried out all the centrifuge experiments described in this paper, at the Disaster Prevention Research Institute, Kyoto University (DPRI-KU). The model is scaled down to 1/40. Thus, all model tests were carried out in the centrifugal acceleration field of 40 g. A functional scheme of the centrifuge facility is given in Fig. 2. The centrifuge has an effective radius of 2.5 m. The rated experimental capacity is 24 g-tons with a maximum attainable centrifugal acceleration of 200 g, and the maximum model mass of 200 kg. The centrifuge model is normally prepared outside the centrifuge pit, and then transferred to the swinging platform (numbered 2 in Fig. 2) and mounted on the centrifuge using a crane.

A motor (7) drives the rotating arm (1) through the shaft and bevel gear (8). The swinging platforms swing outwards and upwards as the centrifuge arm gains speed, so that the direction of the resultant acceleration field passes through the pivot and the centroid of the package. To compensate the unbalance emerging from the weight of the model placed on the platform, an equivalent platform swings on the opposite arm which can be loaded with an equivalent weight (3). A shake table driven unidirectionally by a servo hydraulic actuator is attached to a platform and it is controlled through a personal computer (PC) (6) on the centrifuge arm. It is fixed near to the axis to minimize the centrifugal force acting on the computer. All the equipment necessary for shake table control is put together on the arm. The PC is accessible during flight from a PC (12) in the control room through wireless LAN and “Remote Desktop Environment”. A second wireless LAN system connects the data loggers (5) with another PC (11) in the control room so the time response of the installed sensors can be monitored during flight. The shake table has the capacity of 15 kN, 10 g and ± 2.5 mm in maximum force, acceleration and displacement, respectively.

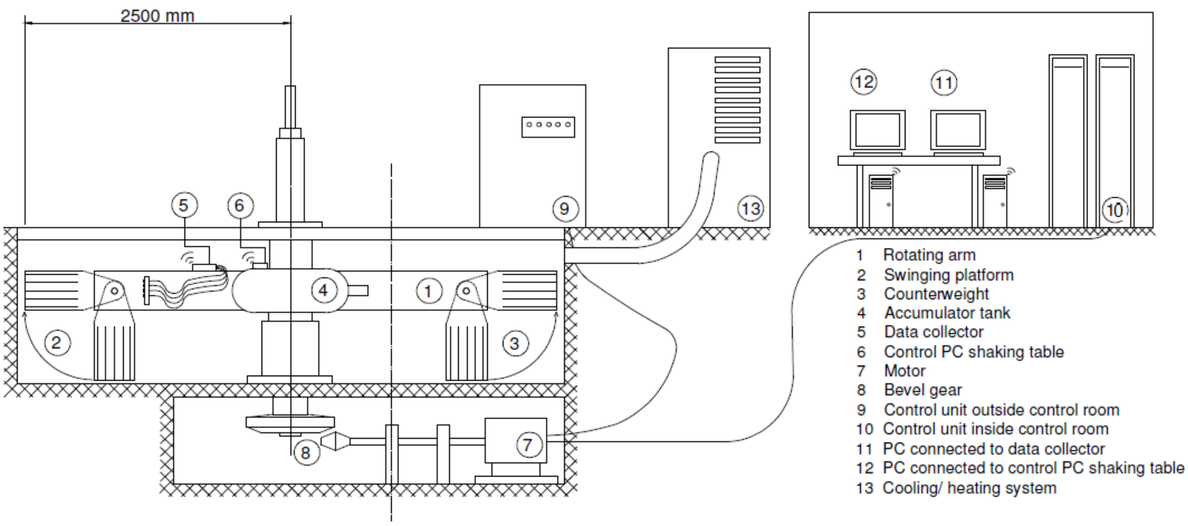


Fig. 2 Schematic view for centrifuge facility at the Disaster Prevention Research Institute, Kyoto University (DPRI-Ku)

3. Material properties

All tests were carried out using a rigid soil container with inner dimensions of 0.45 m (L) × 0.15 m (W) × 0.29 m (H).

The model ground in this study was made of Silica sand No. 7 having the physical and mechanical properties shown in Table 1 and the particle size distribution curve shown in Fig. 3. The soil is classified as "poorly graded sand (SP)". This sand is an industrial material so that consistency of the material properties throughout all models can be assured. A dry sand deposit was prepared by air pluvation. After fixing the pile in a bottom plate in the soil container base, silica sand was rained in 1 g field using a hopper fixed at the specified height until the sand deposit formed 10 m thick deposit (250 mm in model scale). The sand deposit was then consolidated in 40 g centrifugal acceleration field for 5 min. By measuring the heights of the ground surface after the consolidation, relative density was obtained as 90%. The soil was instrumented with accelerometers at different depths.

Table 1 Physical properties of Silica sand No. 7

e_{max}	e_{min}	D_{50}(mm)	Uc	Gs
1.19	0.710	0.13	1.875	2.66

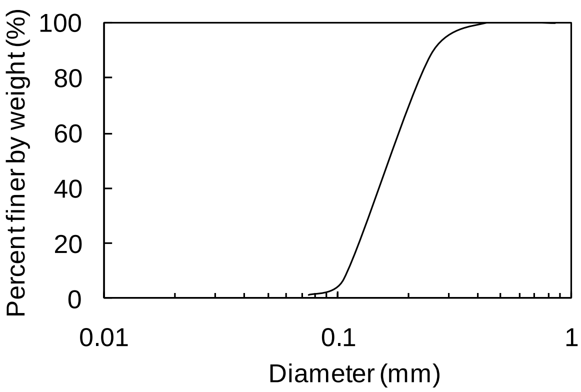


Fig. 3 Particle size distribution curve for Silica sand No.7

The pile and column model used in this study were made of steel tubes with the properties shown in Tables 2 and 3, respectively. Steel masses (mass 1 and mass 2) with the properties shown in Tables 4 and 5 fixed to the pile head and at the top of column, respectively were used.

Table 2 Properties of pile modeling

	Steel tube		
	Model	Prototype	Units
Length	0.29	11.6	m
Outer diameter	10	400	mm
Wall thickness	0.75	30	mm
Young's modulus	206	206	GPa
Moment of inertia	2.35×10^2	6.00×10^8	mm^4
Bending stiffness	48.41	1.24×10^8	$MN\text{-}mm^2$

Table 3 Properties of column modeling

	Steel tube		
	Model	Prototype	Units
Length	0.075	3.0	m
Outer diameter	10	400	mm
Wall thickness	0.75	30	mm
Young's modulus	206	206	GPa
Moment of inertia	2.35×10^2	6.00×10^8	mm^4
Bending stiffness	48.41	1.24×10^8	$MN\text{-}mm^2$

Table 4 Properties of mass 1 modeling

	Model	Prototype	Units
Mass	0.3792	24231	kg
Moment of inertia	9.0×10^4	2.33×10^{11}	mm^4
Bending stiffness	1.85×10^4	4.75×10^{10}	$MN\text{-}mm^2$

Table 5 Properties of mass 2 modeling

	Model	Prototype	Units
Mass	0.297	19008	kg
Moment of inertia	1.41×10^4	3.61×10^{10}	mm^4
Bending stiffness	2.90×10^3	7.42×10^9	$MN\text{-}mm^2$

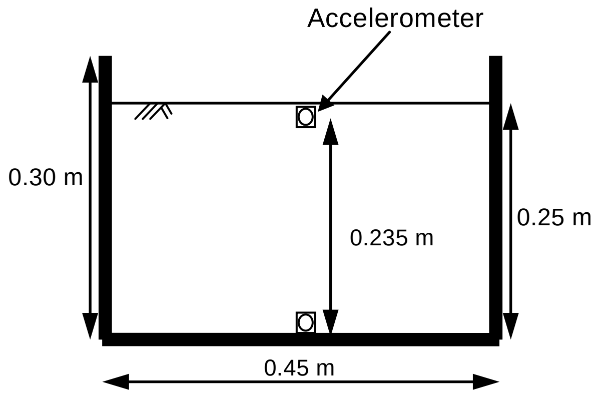
The pile was placed in the model before the soil was pluviated, attempting to simulate a pile installed with minimal disturbance to the surrounding soil, as may be the case when a pile inserted into a pre-augered hole.

4. Test program

A total of 4 tests were performed and these tests are listed in Table 6. Typical cross sections of free-field and single pile model tests are shown in Figs. 4, 5, 6, and 7.

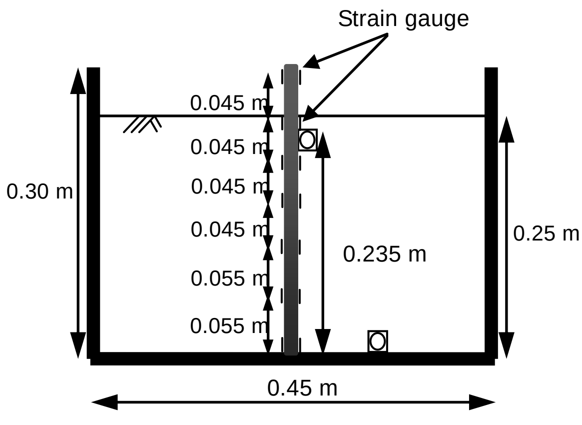
Table 6 Tested cases

Test No	Test setup	Relative density (%)	
		Before the test	After the test
1	Free field	77	81
2	Single pile	78	80
3	Single pile +single degree of freedom	80	82
4	Single pile+ double degree of freedom	80	82



Test 1

Fig. 4 Illustration of free-field test setup (test 1)

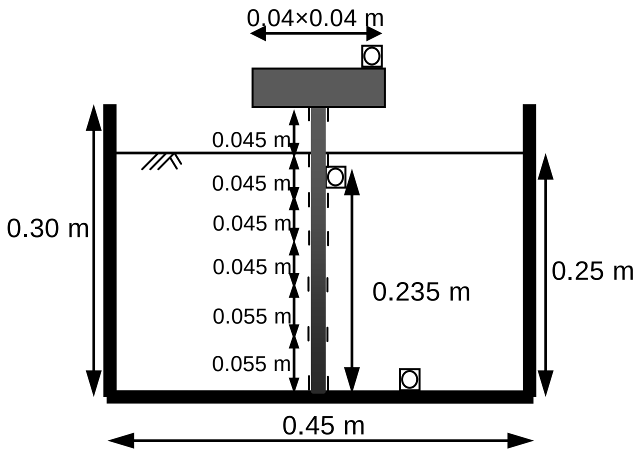


Test 2

Fig. 5 Illustration of single pile test setup (test 2)

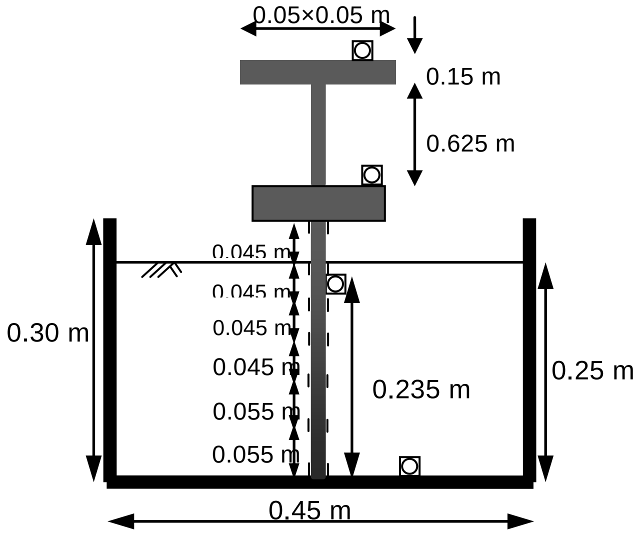
5. Instrumentation

In a series of model tests in the present study, electronic instruments were used to measure bending moments of piles, accelerations of ground, piles heads, and superstructure masses. These instruments can be listed as:.



Test 3

Fig. 6 Illustration of single pile supporting single degree of freedom structure test setup (test 3)



Test 4

Fig. 7 Illustration of single pile supporting double degree of freedom structure test setup (test 4)

1) Accelerometers (SSK, A6H-50), to record dynamic motions of ground, pile, and masses.
2) Seven strain gauges were placed at different locations along the piles to measure pile

bending moments.

The general locations of instruments are shown in Figs. 4, 5, 6 and 7.

6. Test procedures

After confirming that all equipments and sensors are well functioning without any abnormality, centrifugal acceleration was increased gradually up to 40 g. The sand deposit was then consolidated in 40 g centrifugal acceleration field for 5 min. By measuring the heights of the ground surface after the consolidation by a ruler, the actual relative density before shaking was obtained. The centrifugal acceleration is again increased up to 40 g to apply the dynamic motion to the model. Each model was subjected to 12 sinusoidal waves as input base accelerations. These waves have constant amplitude of about 1.5 m/s^2 and varying frequencies ranging from 1 to 12 Hz. After shaking the ground surface settlement was measured again to determine the actual relative density after shaking. The actual relative densities of each tested case before and after shaking are given in Table 6.

7. Centrifuge test results

The comparative results presented in this section are given in terms of different spectral ratio moduli, corresponding to the ratio between displacements at different points in frequency domain. In determining a time history of displacement, measured acceleration was numerically integrated twice. Displacement values shown in this study were all calculated in the same way. Twelve key control points were defined in this study:

- A point located at the base of soil container, input motion, (**o**, in all tests).
- A point on the free-field (i.e. soil ground surface response without soil-structure interaction) (**ff**, in test 1).
- A point located at the single pile head (at the same elevation with the free-field point) (**p**, in tests 2, 3, and 4).
- A point located at the top of M1 (m1, in tests 3 and 4).
- A point located at the top of M2 (m2, in test 4).

8. Free-field response

The first step in any soil-structure seismic analysis is the evaluation of the free-field response of the site, that is, the spatial and temporal variation of motion before excavating or rigging the soil and superimposing the structure. Test 1 was used as a benchmark test and performed to monitor the free-field dynamic response of the dry sand deposit to be as a reference in the subsequent discussions of both kinematic and inertial effect on pile behaviour. It is now a well-established fact of geotechnical earthquake engineering that soil deposits can amplify the seismic rock motion through the soil column toward the surface. In this study, time histories of the free-field displacements were obtained at different frequencies of excitations (1-12 Hz). From these time histories, the amplitude of steady-state displacement (U_{ff}) is noted and normalized with respect to the amplitude of the base displacement (U_o). Thus amplification of free-field response is derived at different frequencies and plotted as shown in Fig. 8. As shown in Fig. 9, the fundamental period of the ground (T_g is the period corresponds to the local maxima of the free-field response) equals to 0.14 sec and the corresponding fundamental frequency of the ground ($f_g \approx 7.0$ Hz).

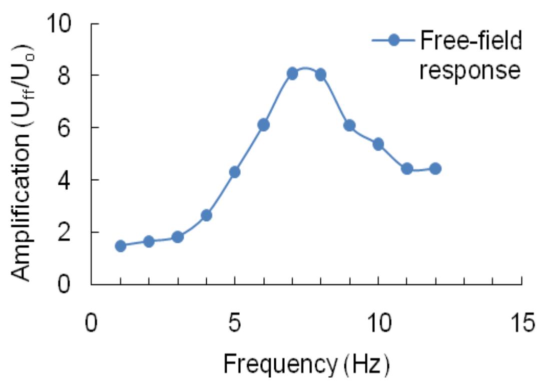


Fig. 8 Free-field amplification versus frequency

9. kinematic interaction

In the absence of the superstructure, the motion of the pile head may be different from the free-field motion. This difference is due to the kinematic interaction mechanism. Kinematic effects are described by frequency dependent transfer functions. The transfer function (kinematic interaction factor, I_u) is defined by the ratio of the pile-head displacement (U_p) to the free-field displacement (U_{ff}) in the absence of a structure. In order to study kinematic soil-pile interaction, Test 2 was performed. Time histories of pile-head displacements were obtained at different frequencies of excitations (1-12 Hz). From these time histories, the amplitude of steady-state pile-head displacement is noted and normalized with respect to the amplitude of the free-field displacement. Normalized pile-head displacements (kinematic interaction factor) versus base excitation frequencies are shown in Fig. 9. It is worth to note from Fig. 9 that

- As expected and at a low-frequency region ($f \leq 4$ Hz), the pile-head displacement may be same as or very close to the dynamic response at the free-field surface.
- At a relatively high-frequency region ($f \geq 4$ Hz), the pile produces strong filtering effects of the base excitation. This difference between pile and free-field response in high frequency region is the result of the difference in pile and soil stiffness ($E_{pile}/E_{soil} \approx 1300$).

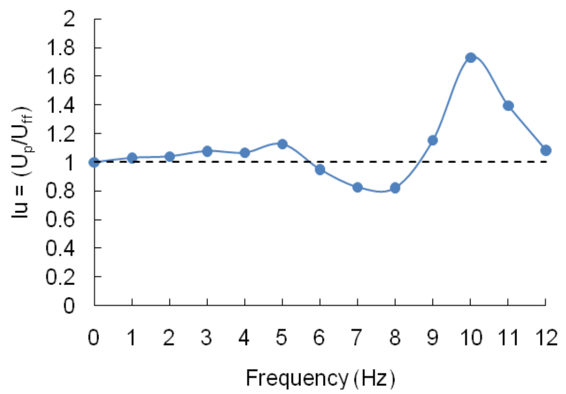


Fig. 9 Kinematic interaction factor versus base excitation frequencies

10. Coupled soil-pile-structure: combined kinematic and inertial action

Tests 3 and 4 were performed to investigate the coupled soil-pile-structure interaction by fixing a single and double degree of freedom structures on the single pile head, respectively. Figures 10 and 11 show the calculated amplification of the single (U_{m1}) and double (U_{m1} and U_{m2}) degree of freedom structures displacement with respect to the base displacement (U_o), respectively. Figures 10 and 11 were obtained using the results of 2D finite element analyses using the 2D FE program FLIP (Iai et al. 1992). In these analyses, structures were considered as fixed base. From Fig. 10, it is easy to realize that the fundamental period of the single degree of freedom structure (T_{s0}) equals to 0.2, and from Fig. 11, it can be realize that the first and the second fundamental periods of the double degree of freedom structures (T_{s1} and T_{s2}) are 0.5 and 0.09 sec, respectively.

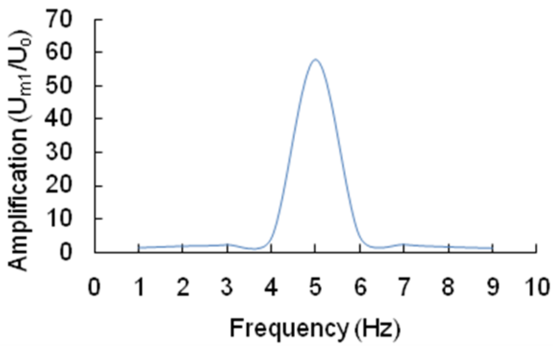


Fig. 10 Fixed base structure amplification versus frequency (test 3): Single degree of freedom

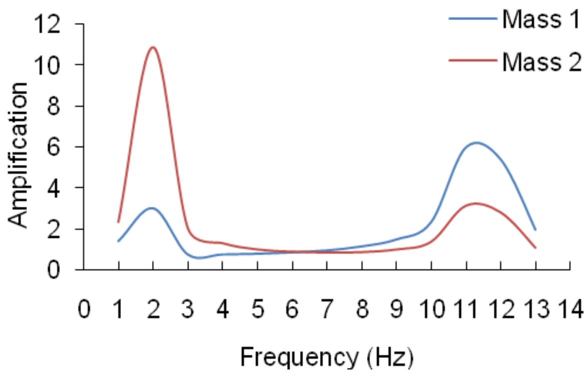


Fig. 11 Fixed base structure amplification versus frequency (test 4): Double degree of freedom

The ratios of the pile-head to free-field horizontal displacements (U_p/U_{ff}) are examined in

Fig. 12. These ratio incorporates the combined effect of inertial and kinematic interaction-as opposed to the kinematic interaction factor ($I_u = U_{sp}/U_{ff}$) which expresses solely kinematic effects. The frequency variation of the ratio (U_p/U_{ff}) is dominated by two discrete frequencies: a lower frequency (the effective natural frequency (f_{SSI})) where the pile-head motion is amplified and a higher one (the pseudo-natural frequency (f_{pSSI}) of the system) where the response is suddenly de-amplified with respect to the free-field motion. The centrifuge results shown in Fig. 12 depict this distinctive behavior of pile supporting structures. It is worth noting that this variation pattern was found to exist in transient response analyses using real earthquake motions of a pile-supported structure (Ohata et al. 1980) as well as analytical studies of soil-pile-structure systems based on the principle of superposition (Mylonakis et al. 1997).

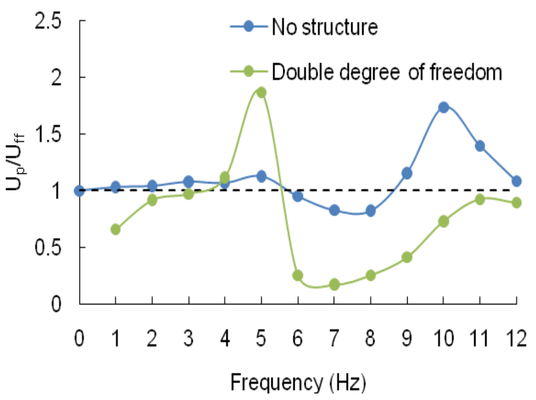


Fig. 12 Pile-head to free-field displacement ratio versus frequency

The frequency at which the ratio of structural mass to free-field horizontal displacement is maximized was computed for each examined case, to determine the effective natural frequency of the soil-structure system (SSI). The effective frequency of the system was then compared to the natural frequency of the structure under fixed-base conditions, thus quantifying soil-structure interaction in terms of fundamental dynamics considerations.

Figure 13 shows amplification ratios obtained from Test 3 that corresponds to single pile supporting a single degree of freedom structure. In this test the fundamental period of the structure, $T_{s0} = 0.2$, is greater than the fundamental period of the ground, $T_g = 0.14$. Figure 13 shows that the effective natural frequency f_{ssi}, the frequency where both structural mass and pile-head motions are maximized relative to the free-field soil surface motion, coincides with the pseudo-natural frequency (f_{pSSI}) of the system, the frequency where the pile-head motion is minimized relative to the free field soil surface motion. In this case, f_{ssi} and f_{PSSI} are found to be equal to 2 Hz as shown in Fig 13. As a result of SSI, a significant reduction of the effective natural frequency f_{SSI} with respect to the natural frequency of the structure under fixed-base conditions (f_s) is observed.

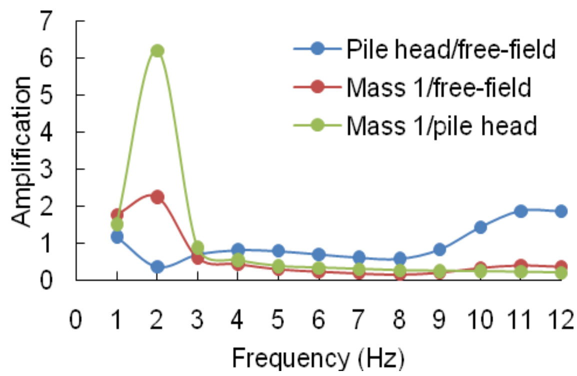


Fig. 13 Amplification ratios: single degree of freedom structure

Figure 14 shows amplification ratios obtained from Test 4 that corresponds to a single pile supporting a double degree of freedom structure. In this test, the relation between the first and second fundamental periods of the structure (T_{s1} and T_{s2}) and that of the ground (T_g) can be given as the following inequality:

$$T_{s2} < T_g < T_{s1} \tag{1}$$

or in term of frequency as:

$$f_{s1} < f_g < f_{s2} \tag{2}$$

Evidently, the response of the system is amplified at the effective natural frequency f_{ssi} of 5.0 Hz while the significant de-amplification of the pile-head motion with respect to the free-field motion takes place at the pseudo-natural frequency

f_{pSSI} of 6.0 Hz. This means that there is a clear evident of the existence of effective and pseudo-natural frequencies not only in the single degree of freedom structures but for double degree of freedom structures also.

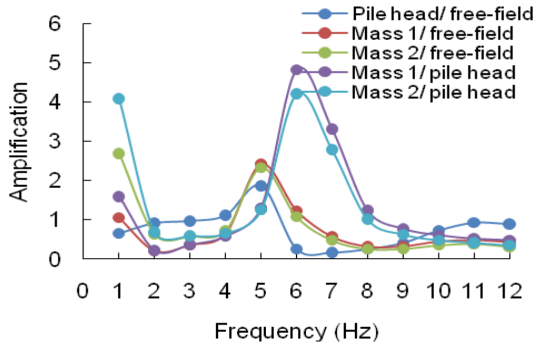


Fig. 14 Amplification ratios: double degree of freedom structure

11. Dynamic pile bending

The response of the coupled soil-pile-structure system is furthered examined in terms of pile bending as a function of frequency content of base excitation. The pile bending moments were normalized to the amplitude of bedrock acceleration following Kavvadas and Gazetas (1993) and Rovithis et al. (2009):

$$M_{nor} = \frac{M}{\rho_p D_p^4 \ddot{U}_g} \qquad (3)$$

where $\ddot{U}_g = w^2 U_g$ is the amplitude of the harmonic input motion introduced at the base of the soil profile.

To investigate the role of the frequency content of the input motion as well as the relative contributions of kinematic and inertial interactions on dynamic pile bending, pile bending moments profiles calculated at frequencies content of input motions closed to the natural frequency of the soil stratum and effective natural frequencies of coupled soil-pile-structure systems (for both single and double degree of freedom structures) are plotted and compared to the corresponding kinematic moments for free-head pile as shown in Fig. 15. Figures 15 (a), (b), (c), and (d) present dynamic pile bending obtained at 1.5 m/s^2 input acceleration and 2, 5, 6, and 7.0 Hz, respectively.

When the frequency content of the base excitation is closed to 2 Hz as shown in Fig. 15(a), the maximum pile bending moment occurs near the ground surface (pile head) and it corresponds to the case of pile supporting single degree of freedom structure. In this case, soil-structure interaction has a major effect on the dynamic characteristics of the structure, resulting in an effective natural frequency f_{SSI} of 2 Hz significantly lower than the fixed base frequency of the single degree of freedom structure (5 Hz). This case reveals the dominant role of inertial interaction in the development of bending moments on or near the pile head when the superstructure is close to resonance.

When the frequency content of the base excitation is closed to 5 Hz as shown in Fig. 15(b), the maximum pile bending moment occurs near the ground surface (pile head) and it corresponds to the case of pile supporting double degree of freedom structure. In this case, soil-structure interaction has also a major effect on the dynamic characteristics of the structure, resulting in an effective natural frequency f_{SSI} of 5 Hz significantly lower than the second fixed base frequency of the double degree of freedom structure (11.4 Hz). This case reveals also the dominant role of inertial interaction in the development of bending moments on or near the pile head when the superstructure is close to resonance.

For the double degree of freedom structure, Fig. 15(c) shows a substantial reduction in pile bending under the combined action of kinematic and inertial interaction when the frequency content of the input motion is closed to f_{pSSI} = 6.0 Hz. This reduction in pile bending may be important in pile design. It is worth to note that, although this significant reduction, the maximum pile bending still corresponds to the single pile supporting double degree of freedom structure but it occurs at a depth of 2 m below ground surface.

When the frequency content of the base excitation is closed to 7.0 Hz as shown in Fig. 15(d), the pile bending moment corresponding to the free-head pile case significantly increases indicating a clear kinematic effect.

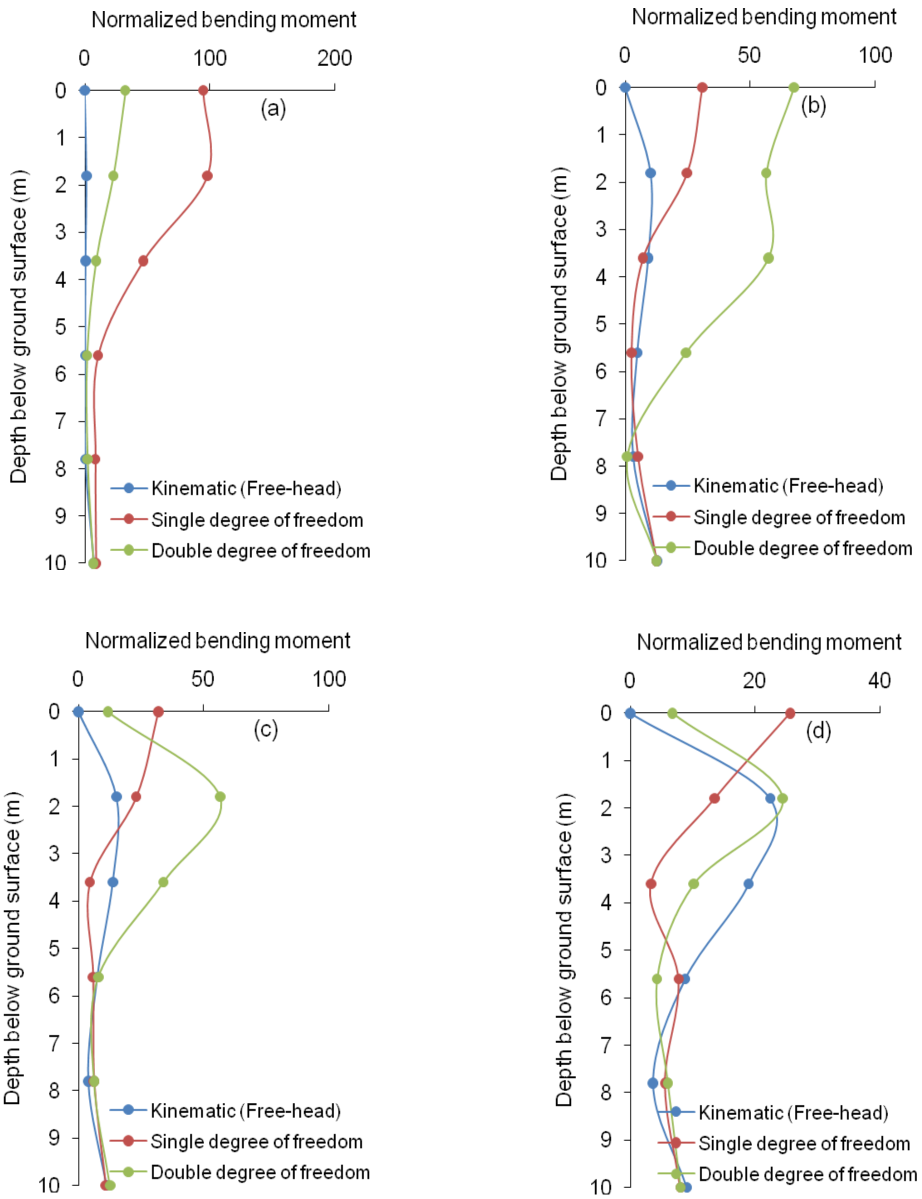


Fig. 15 Distributions of amplitudes of normalized steady state bending moments at: (a) 2.0 Hz, (b) 5.0 Hz, (c) 6.0 Hz, and (d) 7.0 Hz

12. Conclusions

A series of systematic centrifuge tests is conducted in order to study the seismic response of end bearing single piles embedded in dry sand layer and supporting single and double degree of freedom structures. All tests are conducted with the centrifugal acceleration of 40 g. Air pluviation method was used to place the dry sand in the soil container. A total of 4 tests including free-field and single pile cases were performed. Each model was subjected to 12 sinusoidal waves as input base accelerations. These waves have constant amplitude of about 1.5 m/s^2 and varying frequencies ranging from 1 to 12 Hz. Of the findings of this study, the following conclusions can be drawn:

- The pile-head motion is found to be dominated by two discrete frequencies: a lower frequency (the effective natural frequency (f_{SSI})) where the pile-head motion is amplified and a higher one (the pseudo-natural frequency (f_{pSSI}) of the system) where the response is suddenly de-amplified with respect to the free-field motion. These results confirm the numerical results found in the literature and generalize the finding to include higher degree of freedom structures.
- As the frequency content of base excitation

approaches the effective natural frequency of the coupled soil-pile-structure system, the bending moment of the pile dramatically increases. On the contrary, when the input motion is close to the fundamental frequency of the ground, strong kinematic effects are mobilized and generate significant pile bending.

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杭—地盤—構造物の動的相互作用問題に関する遠心模型実験

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要　旨

乾燥砂地盤中の単杭の地震時挙動について調べるため，遠心模型実験を行う。模型短杭は先端支持杭であり，１または２自由度の上部工を杭頭に取り付 ける。動的挙動を調べるため，４０ｇの遠心場で，一定の振幅約1.5m/s²，１Hzから１２Hzの振動数で正弦波加振する。実験結果に基づい て，慣性の相互作用と幾何的相互作用，および自由度の違いが全体系の応答に与える影響について考察したところ，入力加速度の振動数が相互作用に与 える影響について興味深い結果が得られたので報告する。

キーワード：杭，遠心模型実験，曲げモーメント