DYNAMIC PROPERTIES OF STRUCTURE-PILE SYSTEM USING MOCK-UP MODEL

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SUMMARY

The dynamic properties of mock-up models are investigated based on forced vibration tests and earthquake observation. The study investigates the response of the pile foundation and the dynamic impedance from a comparison of the experimental results and numerical results, which were calculated by the existing methods. Main results obtained were as follows.

Numerical results by thin layered element method agrees with experimental results.

2) As for the simplified method, the main characteristics of piles obtained by the experiments could be simulated by taking an appropriate model into consideration. To estimate the ground and dynamic behaviors during earthquakes and interaction problem between soil and piles, the validity of the proposal model has been verified from comparing the earthquake observation results with numerical ones. Numerical results of the proposed model considering frequency dependency and radiation damping of dynamic interaction spring agree with analyses by other analytical methods based on the Green function method and results of earthquake observation. Calculation time necessary for analysis by the proposed model is less than the other methods, and this method can consider nonlinear elastic interaction effects.

INTRODUCTION

Though pile foundations are commonly used to support structures, the mechanism of the behavior during earthquakes is not sufficiently clarified. In recent years, a number of vibration tests of field and model piles have been conducted, through which a remarkable progress has been made in

theoretical studies on dynamic behavior of pile foundations. A recent comprehensive review on the subject has been presented by [Novak, 1991]. [Novak and Sharnouby, 1984] continue to introduce the comprehensive comparison of test results and theoretical predictions performed by different approaches which are essentially based on the Green function method, and indicate a good coincidence between tests and numerical results. Currently, practical application of the interaction analysis for design purposes is also developed to compute the seismic response of a structure supported on pile groups [Hijikata *et al*., 1994]. The approaches based on the Green function method for considering the pile-soil interaction in a layered soil have been verified the effectiveness experimentally, but need many calculation time. Therefore it is not realistic to apply these methods in an earthquake resistant design.

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In this paper, a simplified procedure is proposed for estimating the dynamic interaction between soil and piles system for the purpose of reflecting on seismic design and the validity of the proposal model has been verified by comparing the earthquake observation results with numerical one.

OUTLINE OF FORCED VIBRATION TESTS

Test yard and soil profile

The forced vibration tests of mock-up pile-foundation models are carried out at Funabashi Campus of our college in Chiba Prefecture of central Japan. A layout of models and a bore-hole for investigating the soil profile in the test yard is shown in Fig.1. Number 1 – 8 in Fig. 1 show an arrangement of seismometers on ground surface and 7 points in the underground (1: ground surface, 2: -156 m, 3: -80 m, 4: -44 m, 5: -15 m, 6: -25 m, 7: -6.5 m, 8: -3.5 m). The soil profile in the yard, which is mainly obtained by the measurement from P-S seismic loggings conducted in the bore-hole A, are shown in Fig.2. The bottom of the piles is located at the depth 25 m which consists of fine sand with N-values range of 50 or more.

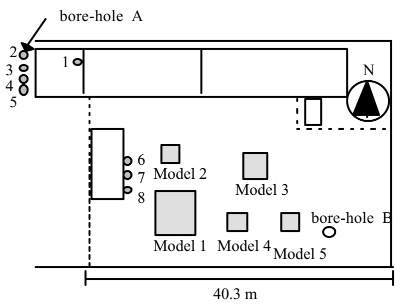


Fig. 1 Test yard and layout of test model

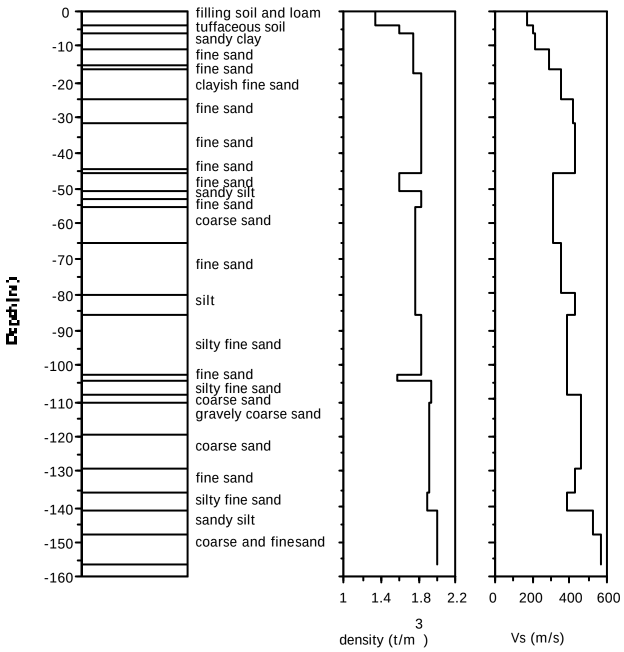


Fig. 2 Soil profile in test yard

Test models and layout of instruments

The piles are made up from steel with the diameter of 0.406 m, wall thickness of 9.5 mm and the length of 26.6 m for all models. The Model 1 is constructed in order to study the behavior of single pile-soil interaction problem. It has four piles which was arranged by a square and the interval 4.0 m, i.e. about 10 times as much as the diameter of the piles. The Model 2 and 3 are constructed in order to study the dynamic stiffness of pile-groups and have four and nine piles respectively, with 1.0 m interval between piles. The Model 4 is constructed in order to study the effect of embedded foundation and its pile arrangement is same as the Model 2 but its floor slab is buried by 0.6 m. The Model 5 is constructed in order to study the effect of piles by comparison with other models and it has no pile. The dimensions of the floor slab for the Model 1 are 5.0 m in both length and width and 1.0 m in height, those for the Model 2, 4 and 5 are 2.0 m in both length and width and 1.2 m in height, and those for the Model 3 are 3.0 m in both length and width and 1.4 m in height. To measure the swaying and rocking response of the floor slab, accelerometers are arranged on the floor slab. The instrumented piles are equipped with strain gauges and accelerometers to detect the response distribution of the pile along depth. Strain gauges and accelerometers are arranged in the ground around the Model 3 to observe the ground behavior.

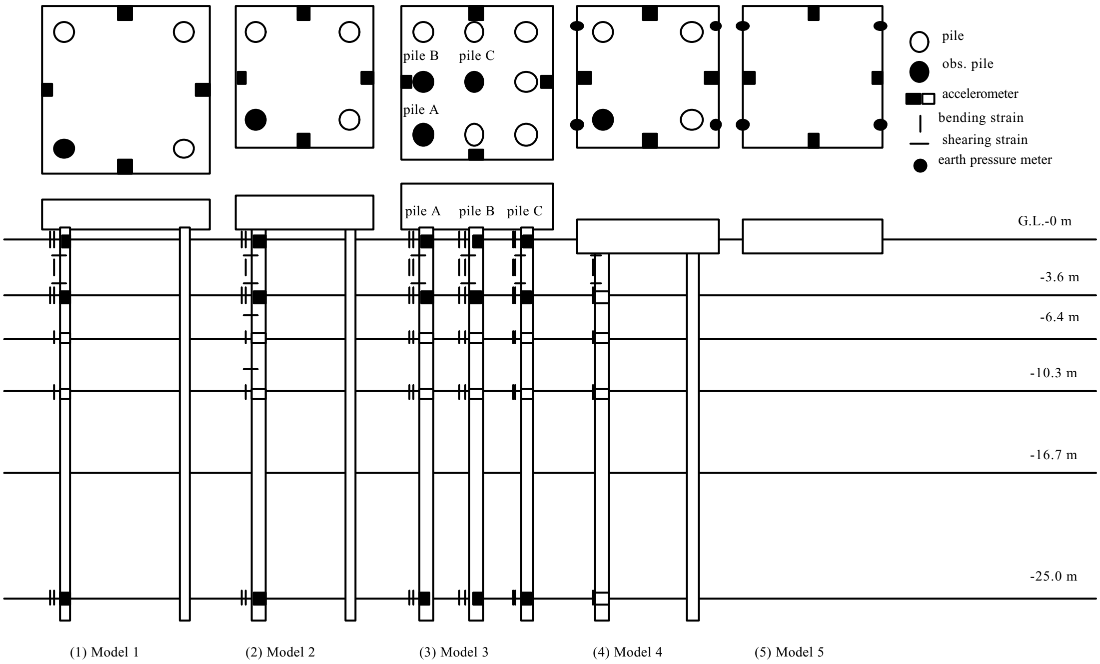


Fig. 3 Test models and location of instruments.

2.3 Test condition

The test models are excited in the horizontal E-W direction with a harmonic wave which is generated by a rotating mass type shaker placed on the floor slab. The shaker used here has the specification listed in Table 1. The excitation patterns are shown in Table 2.

Table 1 Shaker profile

excentric moment	0 ~ 17.5 MNm continuous
max. force	100.0 MN
frequency range	2.0 ~ 25.0 Hz
total weight	30.90 MN
vibration direction	horizontal vertical
control wave	sin wave
output voltage	10 V

Table 2 Contents of experiment

Model	Excitied force (MN)	Frequency range (Hz)
1	10.0	2.0 ~ 25.0
	20.0	2.0 ~ 25.0
	30.0	2.0 ~ 25.0
2	2.5	2.0 ~ 25.0
	5.0	2.0 ~ 25.0
	3.75	2.0 ~ 25.0
3	5.0	2.0 ~ 25.0
	10.0	2.0 ~ 25.0
	7.5	2.0 ~ 25.0

SIMULATION OF THE FORCED VIBRATION TESTS

We simulated the experimental result using the thin layered element method in the first place. The simulation models are shown in Fig. 4. To take account of the nonlinear interaction effects that include both the material nonlinearity of the soil around the pile and the separation between the pile and its surrounding soil, four different cases are considered as follows :

Case 1 : The condition that does not take account of the nonlinear interaction effects.

Case 2 : The condition that take account of the separation between the pile and its surrounding soil at a depth of 50 cm in ground.

Case 3 : The condition that take account of the material nonlinearity of the soil around the pile and the separation between the pile and its surrounding soil at a depth of 50 cm in ground.

Case 4 : The condition that take account of the separation between the pile and its surrounding soil . at a depth of 100 cm in ground.at a depth of 100 cm in ground.

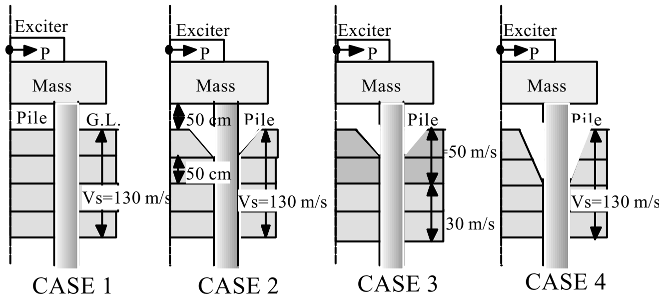
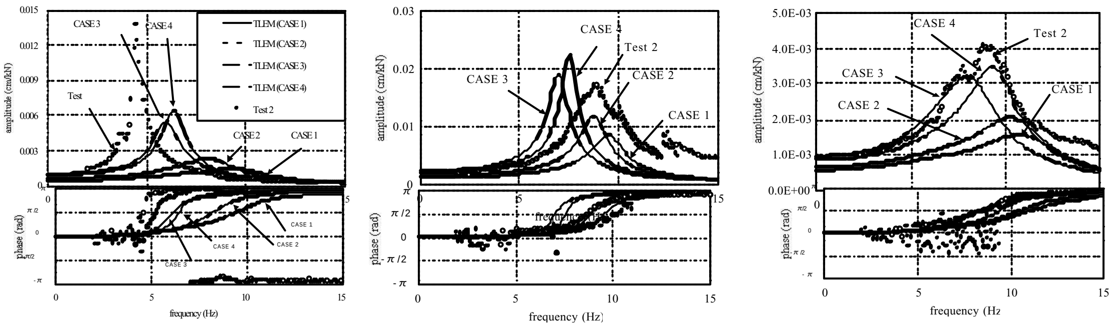


Fig. 4 Numerical case of footing

Table 3 Soil constants for numerical model

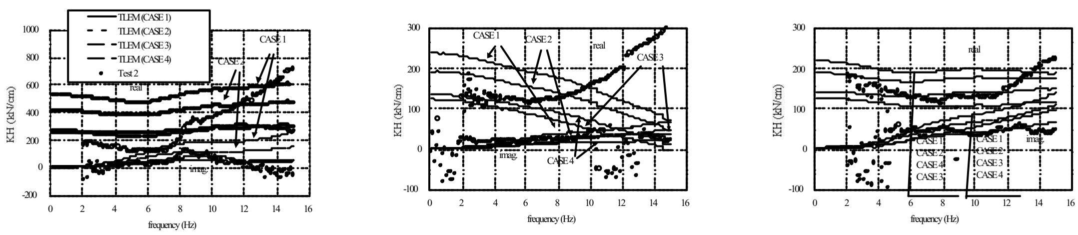
Depth (m)	Vs (m/s)	Density (kN/m³)	Poisson's ratio	damping ratio
3.15	130	13.7	0.420	0.01
6.95	150	14.7	0.488	0.01
19.1	255	18.1	0.486	0.01
26.0	350	18.1	0.476	0.01
142.0	400	18.1	0.469	0.01
semi-infinite soil	550	19.6	0.451	0.01



(1) Model 1 (2) Model 2 (3) Model 3

Fig. 5 Comparison of theoretical and experimental resonance and phase curve .

The simulation results of several cases are shown in Fig. 5. The simulations used case 3 or case 4, reproduces the experiment results well except model 1. Figure 6 shows the dynamic impedance curves calculated backward from resonance and phase curves. The numerical results that take account of the material nonlinearity of the soil around the pile and the separation between the pile and its surrounding soil almost reproduces an experimental result to frequency range 8 Hz.



(1) Model 1 (2) Model 2 (3) Model 3

Fig. 6 Comparison of theoretical and experimental dynamic impedance curves.

Next, we analyzed the experimental results using simplified method based on [Nogami, 1985] method. The numerical model is shown in Fig. 7. It was assumed that pile groups are embedded in homogeneous soil deposits. Figure 8 shows the dynamic impedance curves calculated by the simplified method. The numerical result that take account of the material nonlinearity of the soil around the pile and the separation between the pile and its surrounding soil almost reproduces an experimental result to frequency range 8 Hz, in the same manner as the results by thin layered element method.

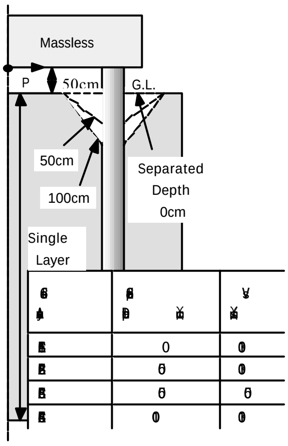
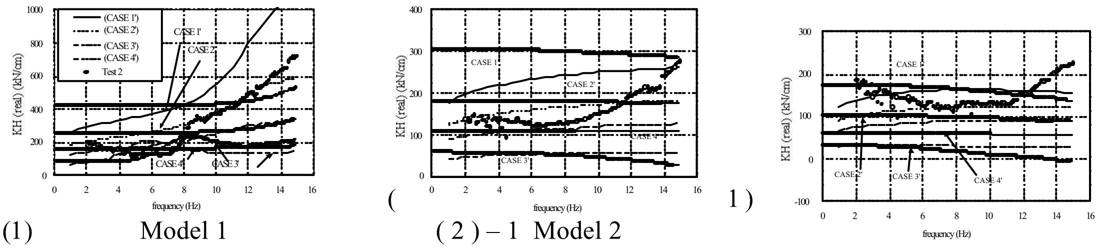
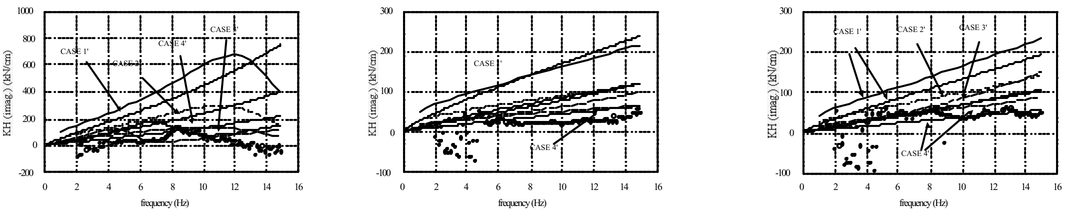


Fig. 7 Numerical case of footing.



(1) Model 1 (2) – 1 Model 2 (3) – 1 Model 3

(a) Real part



(1) – 2 Model 1 (2) – 2 Model 2 (3) – 2 Model 3

(b) Imaginary part

Fig. 8 Comparison of theoretical and experimental dynamic impedance curves.

SIMULATION OF THE EARTHQUAKE OBSERVATION

It was shown that the main characteristics of piles obtained by the experiments could be simulated by the simplified method based on Nogami method in the same manner as the results by thin layered element method in a foregoing paragraph. Therefore we used the simplified method based on Nogami method to estimate the ground and dynamic behaviors during earthquakes and interaction problem between soil and piles firstly, and applied this result to [Penzien, 1964] model. When the horizontally harmonic oscillating force loaded on massless rigid body disk of unit length, the relation between forces P^H_i, P^H_j and displacements u_i, u_j are expressed as

$$\begin{Bmatrix} P_i^H \\ P_j^H \end{Bmatrix} = \begin{bmatrix} f_{ii} & f_{ij} \\ f_{ji} & f_{jj} \end{bmatrix}^{-1} \begin{Bmatrix} u_i \\ u_j \end{Bmatrix} = \begin{bmatrix} f(r_0) & f(R) \\ f(R) & f(r_0) \end{bmatrix}^{-1} \begin{Bmatrix} u_i \\ u_j \end{Bmatrix} \quad . \qquad (1)$$

In case of N piles, the global flexibility matrix assembled from the element matrices of eq. (1) and the interaction spring matrix is given by an inverse matrix. Function $f(r)$ for an angle from the direction of loading are expressed as

$$\left.\begin{aligned} f(r) &= \frac{1}{G}\frac{S_1}{S_0} & \theta &= 0^\circ \\ f(r) &= \frac{1}{G}\frac{S_2}{S_0} & \theta &= 90^\circ \end{aligned}\right\} \quad , \qquad (2)$$

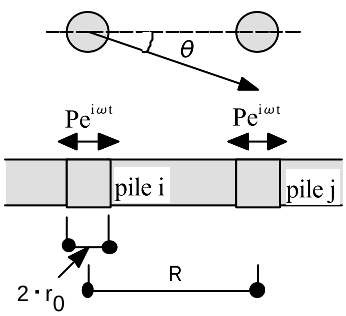


Fig. 9 Simplified plane-strain model

$$S_0 = \pi a_0{}^2 \frac{r}{r_0} \{4 K_1(q)K_1(s) + sK_1(q)K_0(s) + qK_0(q)K_1(s)\} \quad , \tag{3}$$

$$S_1 = -\left\{ K_1(q\frac{r}{r_0}) + q\frac{r}{r_0} K_0(q\frac{r}{r_0}) \right\}\{2K_1(s) + sK_0(s)\} + K_1(s\frac{r}{r_0})\{2K_1(q) + qK_0(q)\} \quad , \tag{4}$$

$$S_2 = -K_1(q\frac{r}{r_0})\{2K_1(s) + sK_0(s)\} + \left\{ K_1(s\frac{r}{r_0}) + s\frac{r}{r_0} K_0(s\frac{r}{r_0}) \right\}\{2K_1(q) + sK_0(q)\} \quad , \tag{5}$$

$$\left. \begin{aligned} a_0 &= r_0\omega / V_s \\ q &= a_0 i / (\eta\sqrt{1+2hi}) \\ s &= a_0 i / \sqrt{1+2hi} \\ \eta &= \sqrt{\{2(1-\nu)/(1-2\nu)\}} \end{aligned} \right\} \tag{6}$$

G : *Shear stiffness.*
K_0, K_1 : Modified Bessel functions of the 0 th and 2 nd kind respectively.
h : Damping constant.
v : Poisson's ratio.

As the same way, when the vertical harmonic excitation force loaded, the relation between vertical forces $P^V{}_i$, $P^V{}_j$ and vertical displacements w_i, w_j are expressed as

$$\begin{Bmatrix} P_i^V \\ P_j^V \end{Bmatrix} = \begin{bmatrix} k_{ii} & k_{ij} \\ k_{ji} & k_{jj} \end{bmatrix}^{-1} \begin{Bmatrix} w_i \\ w_j \end{Bmatrix} = \begin{bmatrix} \beta & 0 \\ 0 & \beta \end{bmatrix} \begin{bmatrix} \alpha_{ii} & \alpha_{ij} \\ \alpha_{ji} & \alpha_{jj} \end{bmatrix}^{-1} \begin{Bmatrix} w_i \\ w_j \end{Bmatrix} \quad , \tag{7}$$

$$\left. \begin{aligned} \alpha_{ij} &= K_0(a_0{}^* r / r_0) \\ \beta &= 2\pi G^* a_0{}^* K_1(a_0{}^*) \end{aligned} \right\} \quad , \tag{8}$$

$$\left. \begin{aligned} G^* &= G(1+2hi) \\ a_0{}^* &= a_0 i / \sqrt{1+2hi} \\ a_0 &= r_0\omega / V_s \end{aligned} \right\} \quad . \tag{9}$$

The numerical analysis model is illustrated in Fig. 10. The piles are modeled by lumped mass model severally. The simulation was done in frequency domain, and the excitation force into a piles through the interaction spring was calculated using the multi-reflection theory and equivalent linearization theory. The earthquake response that was observed in December, 1997 with a hypocenter at Northwest Chiba prefecture was simulated. The maximum acceleration at ground surface was 62.0 gal and time series are shown in Fig. 11.

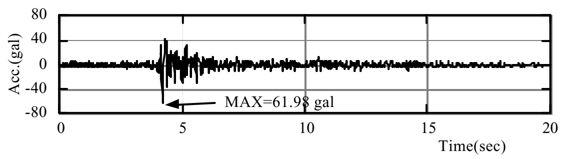


Fig. 11 Observed acceleration at ground surface

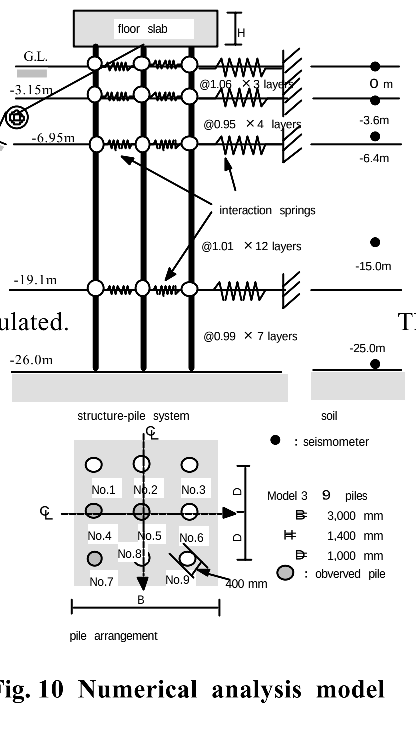
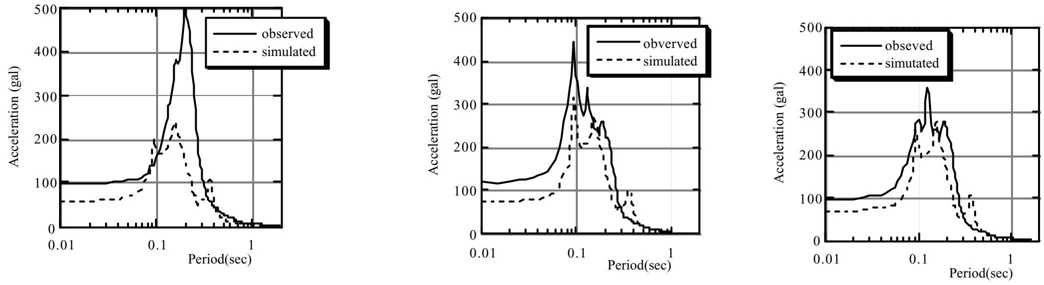
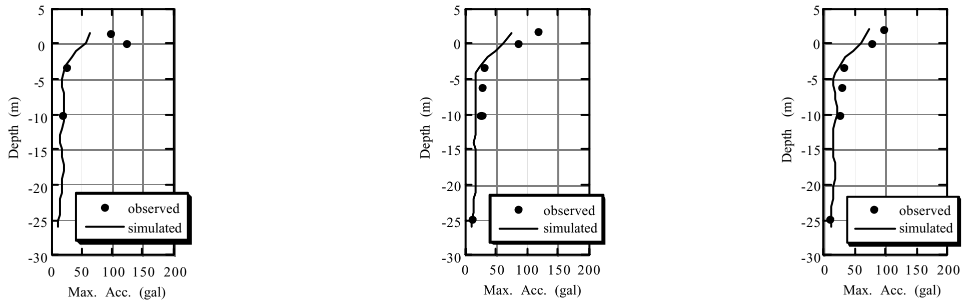


Fig. 10 Numerical analysis model



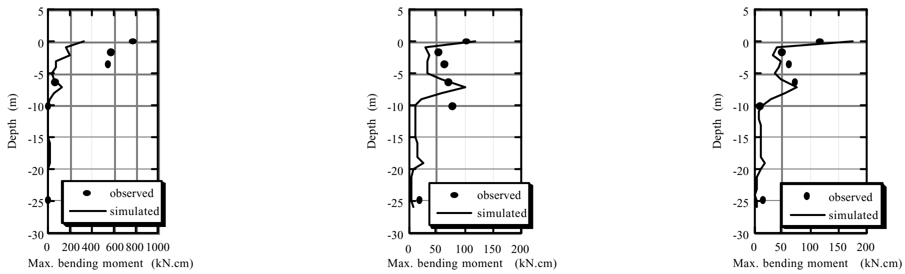
. (1) Model 1 (2) Model 2 (3) Model 3

Fig. 12 The acceleration response spectrum at surface of floor slab.



(1) Model 1 (2) Model 2 (3) Model 3

Fig. 13 The distribution of maximum acceleration of corner pile.



(1) Model 1 (2) Model 2 (3) Model 3

Fig. 14 The distribution of maximum bending moment of corner pile.

The simulation results are shown in Fig 12 ~ Fig. 14. Numerical results agree with earthquake observation results in Model 2 and 3. But in Model 1, numerical results are smaller than earthquake observation one. A observation is done only one corner pile in Model 1, so it is thought that there is influence of eccentric behavior.

CONCLUSION

The study investigates the response of the pile foundation and the dynamic impedance from a comparison of the results of forced vibration tests and numerical results, which were calculated by the existing methods first. An analysis shows that as for the simplified method, the main characteristics of piles obtained by the experiments could be simulated by taking an appropriate model into consideration. Then we used the simplified method based on Nogami method to estimate the ground and dynamic behaviors during earthquakes and interaction problem between soil and piles, and applied this result to Penzien model. To estimate the ground and dynamic behaviors during earthquakes and interaction problem between soil and piles, the validity of the proposal model has been verified from comparing the earthquake observation results with numerical ones. Numerical results of the proposed model considering frequency dependency and radiation damping of dynamic interaction spring agree with analyses by other analytical methods based on the Green function method and results of earthquake observation. Calculation time necessary for analysis by the proposed model is less than the other methods. Then it is thought that this method is useful for an earthquake resistant design.

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