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SYNOPSIS: The earthquake response of a structure on group of piles is investigated. The test model, which is employed in this study, is a foundation on four piles. Three conditions are prepared for examination of the contact effects of the foundation bottom and the backfilling effects on dynamic characteristics of the foundation on the piles. Forced vibration tests are carried out for ascertaining the impedance functions as the inertial interaction, and earthquake observations for the earthquake input motion as the kinematic interaction.

On the basis of the test and the observation results, the correlation analyses are executed for examining the applicability of the analytical method based on the substructure method in which the three-dimensional wave propagation theory is applied for calculation of the Green's functions. The analytical method is concluded to have sufficient applicability for the practical design procedure of a structure on pile group.

INTRODUCTION

Earthquake response analyses of structures supported by pile groups require accurate evaluation of dynamic interaction of piles to piles through soil media. According to the concept of substructure method, this interaction is comprehended to be a composition of two interaction factors. One factor is inertial interaction, which is caused by inertial force of structure on pile groups. The other is kinematic interaction, which is closely related to earthquake input motion to structure. Both interactions are examined in this paper.

In order to improve the design procedures for structures on pile groups, the dynamic behaviors of pile groups have been considered as being important research subject. Many researches on this subject have actually been executed, and numerous technical papers have been published up to the present. [Kaynia(1982), Wolf and von Arx(1982), Waas and Hartmann(1984), Novak and El Sharnouby(1985), Sen et al.(1985), Mamoon and Banerjee(1990), Hijikata et al.(1990)] However, most of these researches dealt with the inertial interaction from analytical approaches. Meanwhile, several papers are found, in which the kinematic interaction is investigated from also the analytical approaches. As few researches can be found from experimental or observational approaches on this subject, the verification studies on both interactions are now required for examination of applicability of these approaches to practical aseismic design.

In this research project, a pile group-foundation model is constructed in a test yard. The forced vibration tests are executed for investigating the inertial interaction, and the earthquake observations for the kinematic interaction. Results of the tests and the observations are employed for the sake of verification studies regarding the analytical approach, which is based on three-dimensional wave propagation theory and is currently recognized to be the most rigorous. It becomes necessary to take into account of contact conditions between the soil and the foundation at the time of practical design. Particular attentions are paid to the contact conditions which are the contact at the foundation bottom, and the contact of the backfilling soil.

THEORETICAL BACKGROUND

Formulas are derived for the explanations of inertial and kinematic interaction regarding the building-foundation-piles-ground system. [Wolf (1985), Kobori et al.(1988)] Introducing interaction force vector $\{f_F\}$ and $\{f_P\}$ between the foundation and the piles, the equations of motion are formulated separately as:

(1) Building-foundation

$$\begin{bmatrix} S_{BB} & S_{BF} \\ S_{FB} & S_{FF} \end{bmatrix} \begin{Bmatrix} u_B \\ u_F \end{Bmatrix} = \begin{Bmatrix} 0 \\ f_F \end{Bmatrix} \tag{1}$$

(2) Piles-ground

$$\begin{bmatrix} S_{PP} & S_{PG} \\ S_{GP} & S_{GG} \end{bmatrix} \begin{Bmatrix} u_P \\ u_G \end{Bmatrix} = \begin{Bmatrix} f_P \\ 0 \end{Bmatrix} \tag{2}$$

where, $[S]$ and $\{u\}$ denote dynamic stiffness matrix and displacement vector. Subscript B is used to designate the building, F for the foundation, P for the piles and G for the ground. In Eq.(2), $[S_{PP}]$ is stiffness matrix of piles themselves, $[S_{PG}]$ and $[S_{GP}]$ are stiffness matrix of the ground which contacts the piles, and $[S_{GG}]$ represents stiffness of the ground in which the pile portions are excluded. Eq.(2) can be divided into Eq.(3) and Eq.(4) by using the interaction force $\{g_P\}$ and stiffness matrix $[S_{SP}]$ of soil columns, which mean soil occupying the pile shaft portions before the piles are driven.

$$([S_{PP}] - [S_{SP}])\{u_P\} = \{f_P\} - \{g_P\} \tag{3}$$

$$\begin{bmatrix} S_{SP} & S_{PG} \\ S_{GP} & S_{GG} \end{bmatrix} \begin{Bmatrix} u_P \\ u_G \end{Bmatrix} = \begin{Bmatrix} g_P \\ 0 \end{Bmatrix} \tag{4}$$

The coefficient matrix of the left-hand side of Eq.(4) is equal to the stiffness matrix of the entire ground(free field ground). Expressing the solutions of Eq.(4) as,

$$\begin{Bmatrix} u_P \\ u_G \end{Bmatrix} = \begin{Bmatrix} u_P^g \\ u_G^g \end{Bmatrix} + \begin{Bmatrix} u_P^f \\ u_G^f \end{Bmatrix} \tag{5}$$

the first term of the right-hand side of Eq.(5) corresponds to the solution of Eq.(6), while the second term to Eq.(7).

$$\begin{bmatrix} S_{SP} & S_{PG} \\ S_{GP} & S_{GG} \end{bmatrix} \begin{Bmatrix} u_P^f \\ u_G^f \end{Bmatrix} = \begin{Bmatrix} g_P \\ 0 \end{Bmatrix} \tag{6}$$

$$\begin{bmatrix} S_{SP} & S_{PG} \\ S_{GP} & S_{GG} \end{bmatrix} \begin{Bmatrix} u_P^g \\ u_G^g \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \end{Bmatrix} \tag{7}$$

Eq.(6) expresses the equation of motion of the free field ground which is subjected to the interaction force vector $\{g_P\}$ along the pile shaft portions. The solution of Eq.(7) indicates the earthquake response of the free field ground. In place of solving Eq.(6), the free field ground displacement vector $\{u_P^f\}$ along the pile shaft portions subjected to $\{g_P\}$ is given by the solution of Eq.(8).

$$\{u_P^f\} = [G_{SS}]\{g_P\} \tag{8}$$

in which $[G_{SS}]$ denotes Green's function matrix of free field ground at the pile shaft positions. Substituting Eq.(8) into Eq.(3), Eq.(9) can be derived.

$$[S_P^*]\{u_P\} = \{f_P\} + \{Q_P\} \tag{9}$$

where,

$$[S_P^*] = [G_{SS}]^{-1} + [S_{PP}] - [S_{SP}]$$
$$\{Q_P\} = [G_{SS}]^{-1}\{u_P^g\} \tag{10}$$

Dividing matrix and vectors in Eq.(9) into these at pile heads(subscript 1) and the other portions(subscript 2), Eq.(9) is transformed to Eq.(11).

$$\begin{bmatrix} S_{P11}^* & S_{P12}^* \\ S_{P21}^* & S_{P22}^* \end{bmatrix} \begin{Bmatrix} u_{P1} \\ u_{P2} \end{Bmatrix} = \begin{Bmatrix} f_{P1} \\ 0 \end{Bmatrix} + \begin{Bmatrix} Q_{P1} \\ Q_{P2} \end{Bmatrix} \tag{11}$$

Solving Eq.(11), the interaction force vector $\{f_{P1}\}$ between the foundation and the pile heads can be obtained as:

$$\{f_{P1}\} = ([S_{P11}^*] - [S_{P12}^*][S_{P22}^*]^{-1}[S_{P21}^*])\{u_{P1}\} - (\{Q_{P1}\} - [S_{P12}^*][S_{P22}^*]^{-1}\{Q_{P2}\}) \tag{12}$$

Assuming that the foundation is rigid and that partial deformations do not occurred, the compatibility constraint matrix $[R]$ at the interface between the foundation bottom and the pile heads are used for relating the displacement vector and the interaction force vector at the pile heads to those at the foundation bottom.

$$\{u_{P1}\} = [R]\{u_F\}$$
$$\{f_F\} + [R]^T\{f_{P1}\} = \{0\} \tag{13}$$

The interaction force vector $\{f_F\}$ can be obtained as:

$$\{f_F\} = -[K_P]\{u_F\} + \{Q_0\} \tag{14}$$

where,

$$[K_P] = [R]^T([S_{P11}^*] - [S_{P12}^*][S_{P22}^*]^{-1}[S_{P21}^*])[R]$$
$$\{Q_0\} = [R]^T(\{Q_{P1}\} - [S_{P12}^*][S_{P22}^*]^{-1}\{Q_{P2}\}) \tag{15}$$

In Eq.(14), matrix $[K_P]$ denotes the dynamic impedance matrix of pile-ground system at the pile heads and vector $\{Q_0\}$ denotes the driving force due to earthquake ground motion. Furthermore, the driving force vector $\{Q_0\}$ can be transformed to the earthquake input motion vector $\{u_0\}$ by Eq.(16).

$$\{u_0\} = [K_P]^{-1}\{Q_0\} \tag{16}$$

Substituting Eq.(14) into Eq.(1) and solving the Eq.(1), the response of the building-foundation system can be obtained. The dynamic impedance matrix $[K_P]$ corresponds with the inertial interaction and $\{Q_0\}$ with the kinematic interaction. When the dynamic impedance matrix $[K_P]$ and the driving force vector $\{Q_0\}$ or the earthquake input motion vector $\{u_0\}$ can be evaluated exactly, the earthquake response of the structure on pile groups can be obtained correctly.

TEST MODEL AND TEST YARD

Fig.1 shows the test model which consists of four piles and a solid reinforced concrete(RC) block foundation on them. The piles are cast-in-place RC pile of 0.6m diameter and 7.5m length. Dimension of the block is 4.4m square plane and 4.0m height. Its weight is approx. 185 ton. Three conditions of the test model are set up as shown in Table 1. After completing the pile driving and excavation to 3m depth from the surface, the block is constructed with keeping a 10cm gap between the block bottom and the surface of the excavated soil. The model at this stage is termed STEP-1. By grouting the gap with cement mortar, a contacting condition is made between the block bottom and the excavated soil surface.(STEP-2) STEP-2 is planned to examine the contact effects on the both interaction factors. Lastly, backfilling the excavated part with sandy soil, the test condition is prepared where the backfilling effects can be examined. (STEP-3) The forced vibration tests are executed in the order of STEP-1, 2 and 3. The earthquake observations are carried out in the reverse order of the tests. Arrangement of seismometers for the earthquake observation under STEP-3 condition are depicted in Fig.1. The sensors B1 and B2 are installed on and in the block, G1, G2 and G3 for the free field ground are placed some distance away from the model, and P1 to P5 along one of the piles.

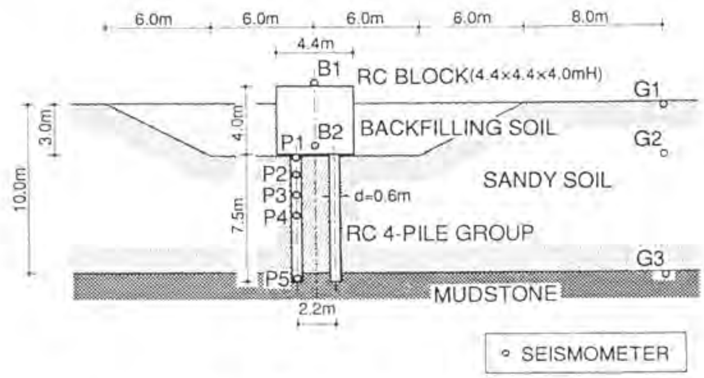


Fig.1 Test Model and Seismometers Arrangement

Table 1 Conditions of Test Model

STEP	TEST MODEL	
STEP-1	10cm GAP	Before BACKFILLING and GROUTING
STEP-2	MORTAR GROUTING	Before BACKFILLING and after GROUTING
STEP-3	BACKFILLING	After BACKFILLING and GROUTING

Table 2 Soil Profile

Depth(m)	Soil Profile	Mass Density (t/m^3)	S-Wave Velocity (m/s)	Poisson's Ratio
	Backfilling Soil	1.85	95	0.199
			145	0.341
			180	0.219
5	Fine Sand	1.80	250	0.179
			320	0.292
10			320	0.429
15	Mudstone	1.75	510	0.459

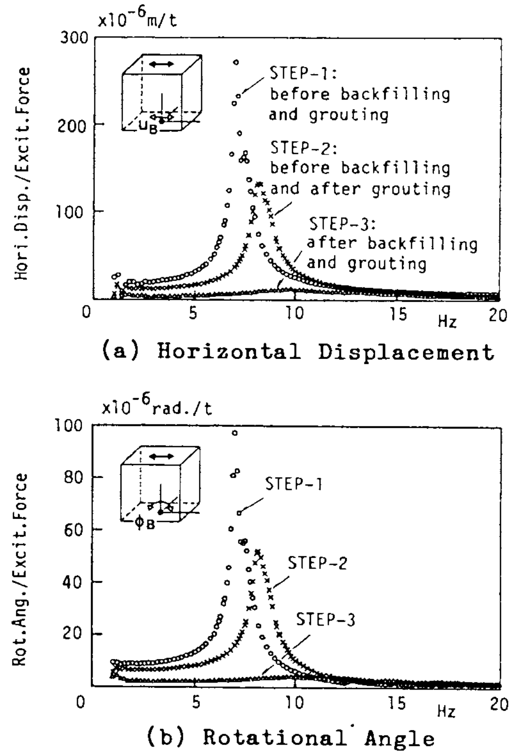


Fig.2 Resonance Curves

Earthquake records are analyzed in the same horizontal direction as the forced vibration tests.
Soil investigations are conducted for PS logging before excavation, ground surface seismic investigation just after excavation, and PS logging for backfilled soil. The soil condition in the test yard is confirmed to be of distinct two layers formation of sandy soil and underlying mudstone as shown in Table-2. The mudstone layer is adopted as the bearing layer in which the pile tips are embedded to 0.5m depth.

INERTIAL INTERACTION

1.Forced Vibration Tests

The inertial interaction means the interaction which is caused by the inertia force of the structure on the pile groups during an earthquake and is understood to be the dynamic impedance problem of pile groups. In order to examine the inertial interaction, an exciter is installed on the top of the block and the forced vibration tests are carried out by applying the sinusoidal excitation force in the horizontal direction. The excitation forces are kept small so that both the test model and the surrounding soil remain in the linear region. The features of the inertial interaction are examined by the resonance curves and the impedance functions which are obtained from the test results.

2.Tests and Correlation Analyses

Fig.2 shows the resonance curves with superposing of the results of STEP-1 to 3. These results are as for the horizontal displacements and the rotational angles at the center of the block bottom. They are plotted by coordinate conversion of the observed displacements at the measuring points on the block to the bottom center after confirming that the block is vibrating as a rigid body. The resonance frequency of STEP-1, 2 and 3 is 7.0, 8.2 and 9.8Hz, respectively. The resonance frequency indicates an increase of 1.2Hz after the grouting(STEP-1 to 2) and 1.6Hz after backfilling(STEP-2 to 3), and the resonance amplitude of both the displacement and the angle decrease to 50% and 10% respectively. It can be seen from these results that the backfilling effects are more prominent than the gap effects.
The correlation analyses for STEP-1 results are conducted by using Green's function method based on three-dimensional wave propagation theory in multi-layered soil media. Discretizing the pile shafts into segments, the Green's functions are calculated which relate the soil displacements at the segment positions to the forces on the other segment positions. Cylindrical loads are applied along the segments and disk loads on the tip of piles as shown in Fig.3. Of course, the existence of piles are ignored in this computations, and the flexibility matrix of which components are Green's functions is obtained. Computing the inverse matrix of the flexibility

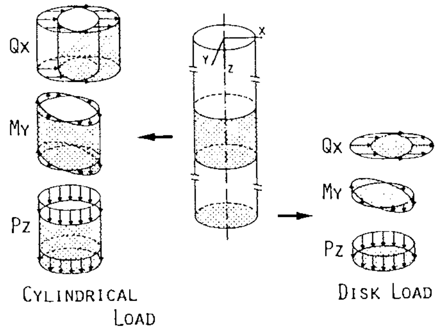


Fig.3 Loading Conditions for Green's Functions

Table 3 Calculation Method for Dynamic Impedance of Each STEP

STEP	TEST MODEL	CALCULATION METHOD FOR DYNAMIC IMPEDANCE
STEP-1	10cm GAP	K_p — With a gap by method based on 3-D wave propagation theory
STEP-2	MORTAR GROUTING	$K_p^{*} = K_p + K_S - K_{SP}$ — Without a gap = With a gap (obtained in STEP-1) + Surface foundation − Soil columns
STEP-3	BACKFILLING	K_p^{*}, K_ϕ, K_U — Without a gap (obtained in STEP-2); Backfilling soil spring by Novak's theory

matrix, $[G_{SS}]^{-1}$ in Eq.(10) is given. As for the compatibility constraint matrix $[R]$, it is assumed that the piles are fixed to the foundation and that the foundation is vibrating as a rigid body and its movements are expressed by five degrees $\{u_x,\phi_x,u_y,\phi_y,u_z\}$ of freedom. The dynamic impedance matrix $[K_P]$ of the pile groups is obtained from Eq.(15). The components of this matrix correspond with the impedances concerning the degree of freedom of the foundation. Calculation methods for dynamic impedance functions of STEP-2,3 are shown in Table 3.

Fig.4 and Table 4 show the material constants of soil and piles-foundation for the correlation analyses. Three cases of soil constants are set up for STEP-1 analyses. Constants of CASE(1) coincide with those of Table-2 which are obtained by the soil investigations. CASE(2) and (3) have the reduced S-wave velocities of 10% and 20% for sandy soil, respectively. In view of the accuracy of the soil investigation, this degree of modification of soil constants is acceptable. Fig.5 illustrates the correlation analysis results of resonance curves and impedance functions for the horizontal K_H and rotational K_R component. The impedance functions for tests are calculated by using the inertia force and moments of the block, the excitation force, and the horizontal displacement and rotational angle at the block bottom center. The impedances of STEP-3 are composed of those at the bottom and on the side walls of the block.

	S-Wave Velocity Vs(m/s)		
	CASE(1)	CASE(2)	CASE(3)
①	180	x0.9	x0.8
②	250		
③	320		
④	320		
⑤	510	510	510

Damping factor:
2%(①~④),1%(⑤)

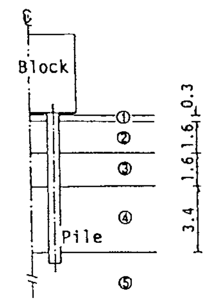
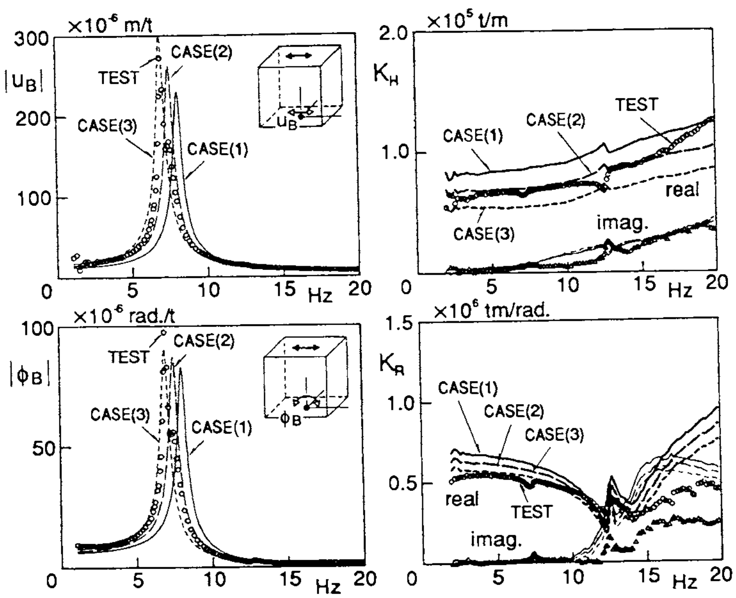


Fig.4 S-Wave Velocities for STEP-1 Analysis

Table 4 Material Constants of Pile and Block

	Mass Density (t/m^3)	Young's Modulus (t/m^2)	Poisson's Ratio	Damping Factor (%)
Pile	2.4	2.4×10^6	0.167	0
Block	2.4	Rigid Assumption		



(a) Resonance Curves **(b) Impedances**

Fig.5 Correlation Analyses of STEP-1

The analysis results using CASE(1) constants are of somewhat higher resonance frequency and larger real part of impedances than the tests. CASE(2) and (3) show good agreement with the tests except for K_R in high frequency range. Numerical conditions CASE(4) and (5) for STEP-2 are set up as shown in Fig.6. As for impedance K_S of the surface foundation for STEP-2, the entire bottom area contacts with soil in CASE(4), while 64%(=0.8x0.8x100%) of the area contacts in CASE(5). Furthermore, the impedances of pile group in STEP-2 are provided with the same values as these of CASE(2) in STEP-1. Fig.7 shows correlation analysis results of impedance functions for STEP-2. The results of CASE(4) are somewhat overestimated, but the results of CASE(5) conform well with the tests.

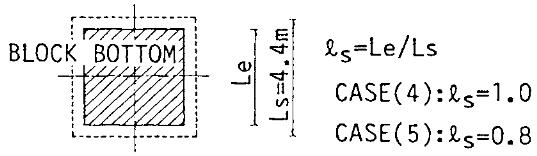
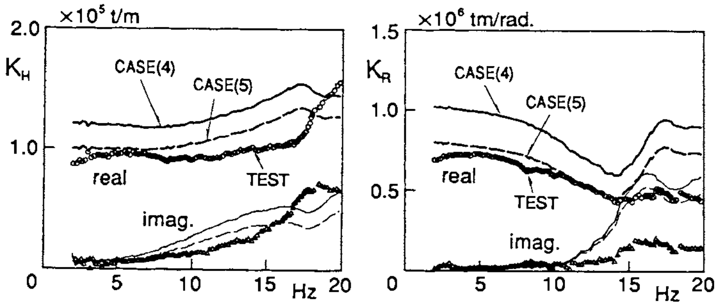


Fig.6 Contact Area of Block Bottom



(a) Horizontal (b) Rotational

Fig.7 Correlation Analyses of Impedances in STEP-2

Applying the impedances of CASE(5) in STEP-2 to the impedances at the block bottom in STEP-3, CASE(6),(7) and (8) for backfilling soil conditions are considered as shown in Fig.8. Fig.9 shows correlation analysis results of impedance functions for STEP-3. In comparison with analysis results for STEP-1 and 2, good agreement can not be obtained in STEP-3 where the foundation is embedded. Among the three CASEs, CASE(7) accounts the best for the test results.
The correlation analyses show slight discrepancies between the tests and the analyses, but the analysis methods aforementioned are judged to have sufficient applicability for evaluation of inertial interaction from the practical design viewpoint.

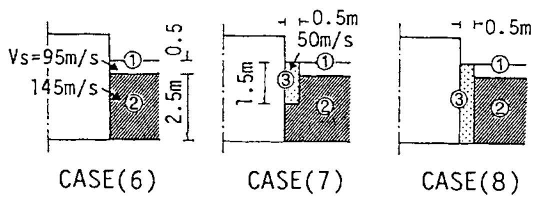
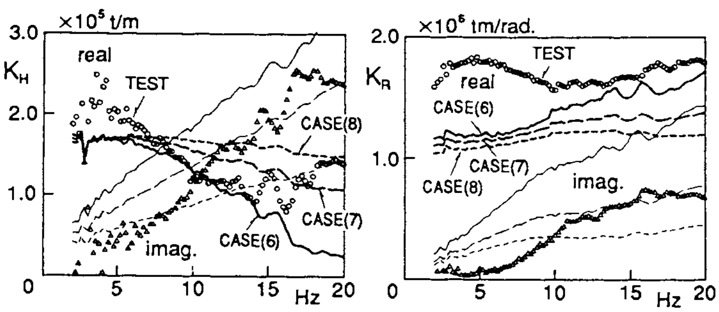


Fig.8 Analysis Conditions of Backfilling Soil for STEP-3



(a) Horizontal (b) Rotational

Fig.9 Correlation Analyses of Impedances in STEP-3

KINEMATIC INTERACTION

1.Earthquake Observations

Kinematic interaction means the interaction which is caused by earthquake disturbances and is recognized to be the earthquake driving force $\{Q_b\}$ expressed by Eq.(15) or the earthquake input motion $\{u_o\}$ by Eq.(16). Since the earthquake observations were started in 1987 after completing the inertial interaction studies, many earthquakes were recorded up to the present. Selecting the earthquake records with comparatively higher acceleration level from among them, the kinematic interaction is examined. The epicenter and magnitude of the selected earthquakes are shown in Fig.10. Nos.1 to 3 earthquake are under STEP-3 model condition, Nos.4 to 6 under STEP-2 and Nos.7 to 9 under STEP-1. The maximum acceleration at the top of the block(B1) is observed during No.6 earthquake, and its value is 80gal.

2.Earthquake Response of Free Field Ground

Fig.11 shows spectral ratios of observed earthquakes at the surface(G1) of the free field ground to those at the top(G3) of the mudstone, and at 3m depth(G2). Correlation analysis results are also drawn in this figure. This figure plots spectral ratios of three earthquakes in which one earthquake is chosen for each of STEPs. Resonance frequencies appear first at about 7.5Hz and secondly at around 18Hz. The first resonance is caused by the first natural frequency of sandy soil of 10m thickness. Examining the spectral ratios G1/G2 shown in Fig.11(b), it can be seen that the second resonance frequency corresponds with the natural frequency of soil near the surface.
For the correlation analyses of the spectral ratios of the ground, three numerical CASEs are set as shown in Table 5. The analysis method is

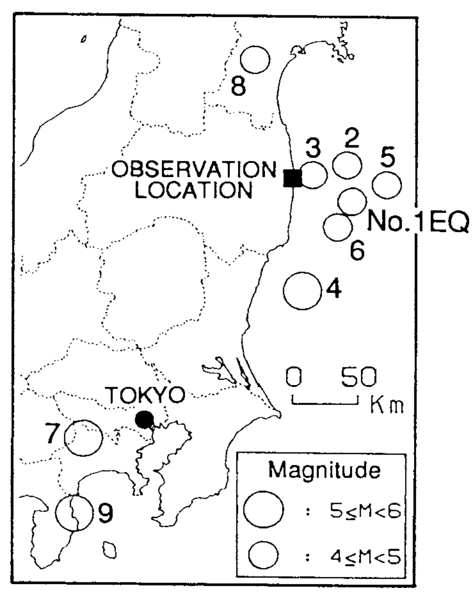
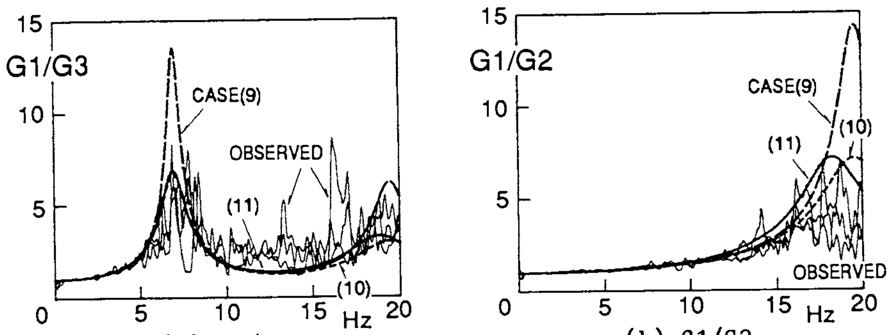


Fig.10 Epicenter and Magnitude of Observed Earthquakes



(a) G1/G3 (b) G1/G2

Fig.11 Spectral Ratios of Observed Earthquake in Free Field Soil

Table 5 Soil Constants for Free Field Soil Analyses

<table>
<tr><th rowspan="2">Depth(m)</th><th colspan="2">CASE(9)</th><th colspan="2">CASE(10)</th><th colspan="2">CASE(11)</th><th rowspan="2">Mass Density (t/m^3)</th><th rowspan="2">Poisson's Ratio</th></tr>
<tr><th>Vs</th><th>h</th><th>Vs</th><th>h</th><th>Vs</th><th>h</th></tr>
<tr><td rowspan="5"></td><td>160</td><td rowspan="4">5.0</td><td rowspan="5">x1.0</td><td rowspan="4">10.0</td><td>140</td><td rowspan="4">10.0</td><td>1.50</td><td>0.258</td></tr>
<tr><td>260</td><td rowspan="4">x1.0</td><td rowspan="3">1.80</td><td>0.304</td></tr>
<tr><td>280</td><td>0.258</td></tr>
<tr><td>280</td><td>0.488</td></tr>
<tr><td>580</td><td>5.0</td><td>5.0</td><td>5.0</td><td>1.75</td><td>0.442</td></tr>
</table>

Vs: S-Wave Velocity (m/s)
h : Hysteretic Damping (%)

the one-dimensional shear wave propagation theory such as a computer code SHAKE. Constants of CASE(9) is obtained by PS-logging in a boring hole at G1 location, but damping factor h as an hysteretic type is assumed at 5% throughout the soil. In CASE(10) and CASE(11), 10% damping is given for sandy soil. Soft surface soil layer is considered in CASE(11). All CASEs account well for the first resonance frequency shown in Fig.12(a). Spectral ratio at that frequency is overestimated by CASE(9), while CASE(10) and (11) show good agreement with the observations. As for spectral ratios near the second resonance frequency shown in Fig.11(b), CASE(9) gives higher spectral ratio, and both CASE(10) and (11) exhibit good agreement with the observations. CASE(11) gives a more suitable resonance frequency than CASE(10). Hereafter, soil constants of CASE(11) are employed for correlation analyses of earthquake records on the test model.

3.Earthquake Response of Test Model

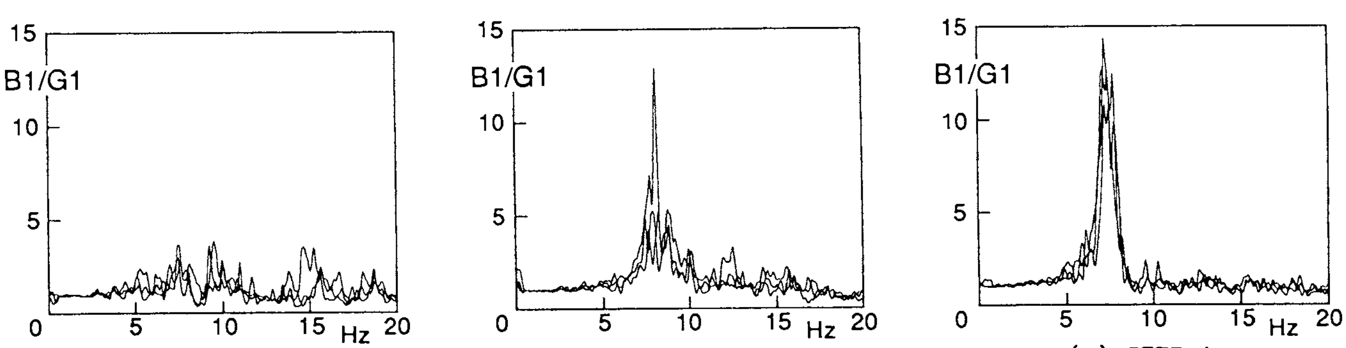
Fig.12 shows spectral ratios of observed earthquakes at the top of the block(B1) to those at the surface of the free field ground(G1). Therefore, these results indicate the amplification characteristics of the pile group-foundation system under each STEP condition. The resonance frequencies of each STEP were confirmed to be 9.8Hz for STEP-3, 8.2Hz for STEP-2 and 7.0Hz for STEP-1 by the forced vibration tests, and the first resonance frequency of the free field ground was also confirmed to be around 7.5Hz through earthquake observations. Reflecting these resonance frequencies, the peaks appear in spectral ratio curves of each STEP. In other words, the peaks are recognized at about 7.5 and 10.0Hz for STEP-3, and about 7.5 and 8.0Hz for STEP-2. Only one peak appears at about 7.0Hz for STEP-1, which is formed by combining the resonance frequency of the pile groups-foundation system and that of the ground. The spectral ratio at the resonance frequencies become larger in the order of STEP-3, 2 and 1.

4.Earthquake Input Motion

Based on the earthquake observations, the earthquake input motions expressed by Eq.(16) are derived and examined. Assuming that the block can be mathematically modeled as a rigid body supported by the impedance functions K_H and K_R aforementioned, the vibrations of the block can be expressed by Eq.(17).

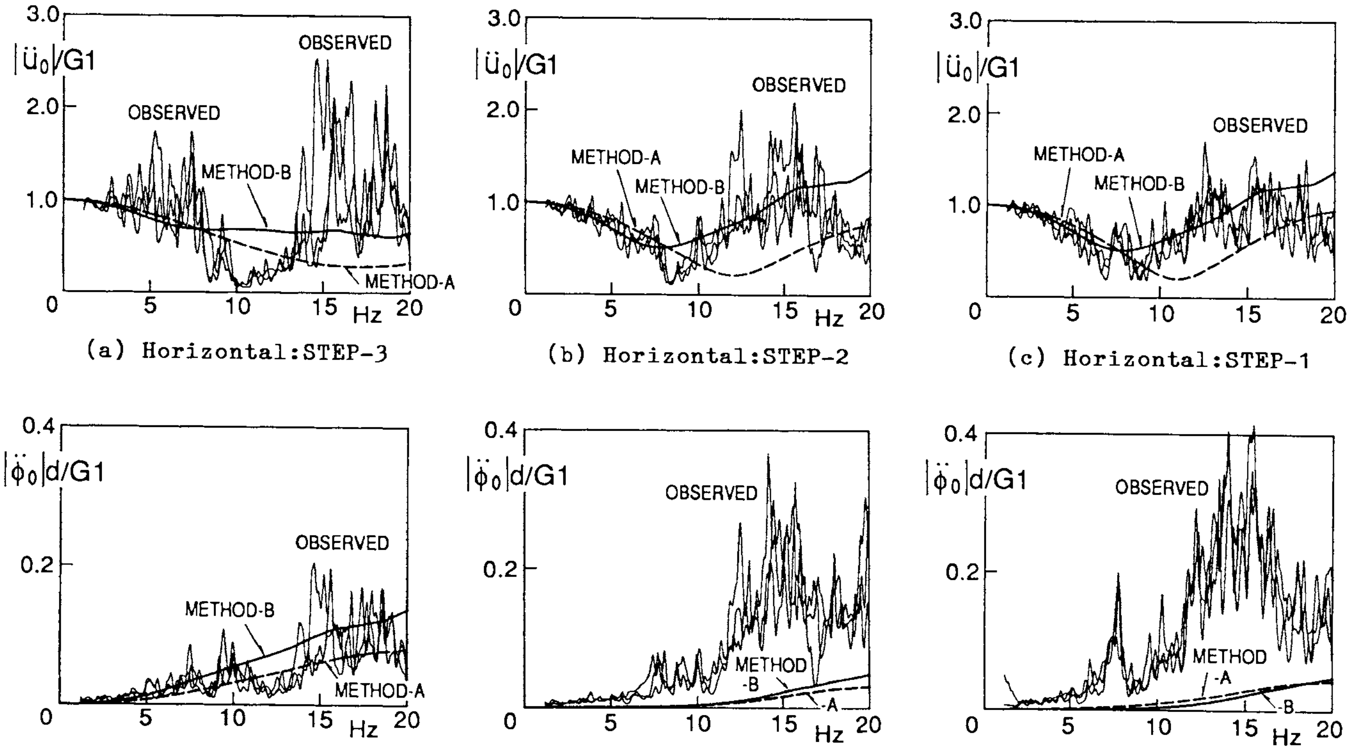
$$\begin{aligned} & m\left(\ddot{u}_B + H/2\cdot\ddot{\phi}_B\right) + K_H\cdot u_B = K_H\cdot u_0 \\ & I_y\cdot\ddot{\phi}_B + H/2\cdot m\cdot\ddot{u}_B + K_R\cdot\phi_B = K_R\cdot\phi_0 \end{aligned} \qquad (17)$$

in which, H, m and I_y are height, total mass and inertia moment at the bottom center of the block, u_B and ϕ_B are horizontal displacement and rotational angle at the block bottom center. u_0 and ϕ_0 indicate earthquake input motions of horizontal displacement and rotational angle. In STEP-3 where the block is embedded, earthquake input motions are obtained as composed input motion of these through the block bottom and through the side walls. Solving Eq.(17) in frequency domain which is obtained by Fourier transformation of observed earthquake records, earthquake input motions are expressed as;



(a) STEP-3 (b) STEP-2 (c) STEP-1

Fig.12 Spectral Ratios of Observed Earthquake at Top of Block to That on the Surface of Free Field Soil



(a) Horizontal:STEP-3 (b) Horizontal:STEP-2 (c) Horizontal:STEP-1

(d) Rotational:STEP-3 (e) Rotational:STEP-2 (f) Rotational:STEP-1

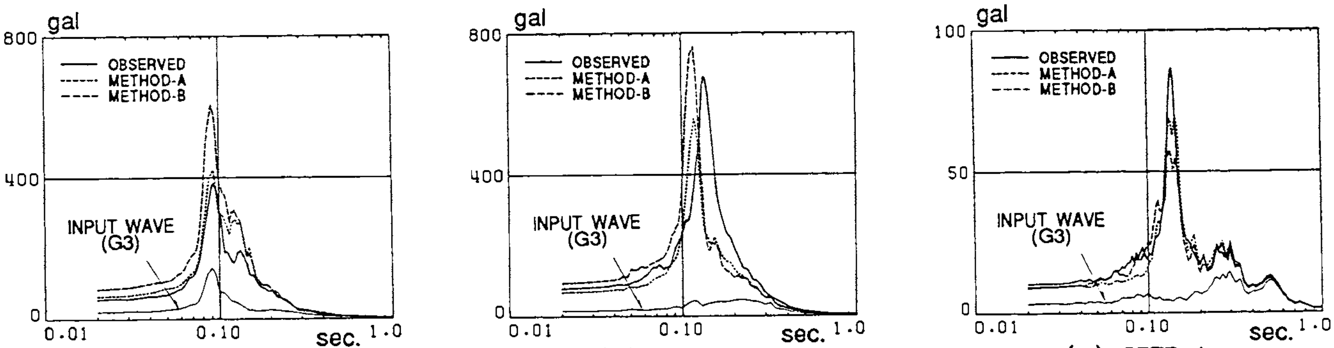
Fig.13 Earthquake Input Motions

$$u_0 = (1-\omega^2 m/K_H)u_B - \omega^2 H/2\cdot m/K_H\cdot\phi_B$$
$$\phi_0 = (1-\omega^2 I_y/K_R)\phi_B - \omega^2 H/2\cdot m/K_R\cdot u_B \qquad (18)$$

where, ω is the angular frequency.
Fig.13 shows the earthquake input motions which are derived from earthquake records by applying Eq.(18). Horizontal input motions u_0 are plotted as the ratio to earthquake motion(G1) at the surface of free field ground, and rotational input motion as the ratio of motion due to rotation($\phi_0\cdot d$: d= diameter of pile) to G1. The horizontal input motions become smaller at the resonance frequency of the block of each STEP, while the tendencies are not recognized obviously in the rotational input motion. The rotational input motion is not significant in lower frequencies, but this becomes more significant in higher frequencies. The horizontal input motion becomes the largest in STEP-3 where the block is embedded, but in this STEP the rotational input motion is the smallest. The difference of the input motion between STEP-2 and STEP-3 is not distinct, but it can be recognized that the horizontal input motion of STEP-2 is slightly larger and the rotational input motion of STEP-2 is slightly smaller than those of STEP-1.
Eq.(7), Eq.(10) and Eq.(16) are employed for the calculation of the earthquake input motion.
Eq.(7) expresses equation of the motion of the ground subjected to earthquake disturbances. Assuming that the earthquake disturbances are restricted to the shear wave propagating upward, the earthquake response of the free field ground can be calculated by one-dimensional shear wave propagation theory such as the computer code SHAKE, or with more accuracy by Axi-symmetric Finite Elements Method(FEM) in which the form of the excavation in STEP-1,2 and the configuration of the backfilling soil in STEP-3 are taken into account. The input motion by SHAKE is termed METHOD-A and that by FEM is termed METHOD-B. The analyses results are shown in Fig.13. Method-B account well for the horizontal input motion in STEP-2 and STEP-1, while METHOD-A somewhat underestimates the horizontal input motion in these STEPs. The rotational input motion in all STEPs and the horizontal input motion in STEP-3 are underestimated by the two METHODs.

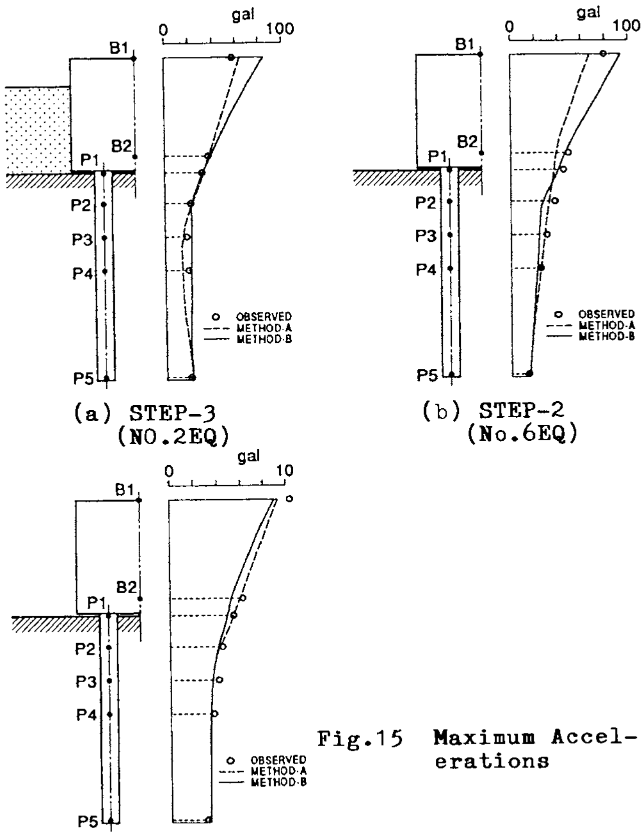
5. Correlation Analyses of Earthquake Response

Earthquake responses of the block are calculated by applying the impedance functions and the input motions as aforementioned. Fig.14 shows the comparison of response spectra(h=3%) at the top of the block. The analysis results by METHOD-A and -B show good agreement with the observed results. In the analyses of the earthquake input motion, some discrepancies are recognized between the analyses and the observations. However, these discrepancies lead to the minor difference in the response spectra. The comparisons of maximum acceleration are depicted in Fig.15. The analysis results show close agreement with the observed values.
In the practical design analyses of the structure on pile groups, it is more important to apply the analytical method which is simple procedure and leads to the appropriate response spectra and maximum acceleration. From this point of view, METHOD-A is judged to be the suitable method for the evaluation of the earthquake input motion.



(a) STEP-3 (No.2EQ) (b) STEP-2 (No.6EQ) (c) STEP-1 (No.9EQ)

Fig.14 Correlation Analyses of Response Spectra at Top of Block



(a) STEP-3 (NO.2EQ) (b) STEP-2 (No.6EQ) (c) STEP-1 (No.9EQ)

Fig.15 Maximum Accelerations

CONCLUSIONS

The forced vibration tests, the earthquake observations and the correlation analyses for their results are carried out to investigate the earthquake response of structure on piles. The test model, which is used for this paper, is a block foundation on four piles. Three conditions are prepared for examining the contact effect of the foundation bottom and the backfilling effect regarding the dynamic behavior of the foundation on piles. The effects of these factors are ascertained on the dynamic impedance functions as the inertial interaction and the earthquake input motions as the kinematic interaction.

The correlation analyses are carried out for the examination of applicability of the analytical method as the practical design procedure of the structure on piles. The examined analytical method is based on the substructure method where the three dimensional wave propagation theory in multilayered soil media is applied for the calculation of Green's functions. A slight modification of soil constants is required so as to obtain good agreement of the analytical results with the tests and the observed results. However, when the accuracy of soil investigations is considered, the modification are of minor degree which can be acceptable. The analytical method is concluded to have the sufficient applicability for the practical design of the structure on pile groups.

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