Study on the Dynamic Characteristics of an Actual Large Size Wall Foundation by Experiments and Analyses

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The dynamic behavior of a large size wall foundation supporting a 54-story building is studied in this paper. The contents are on a response analysis with soil-foundation-superstructure interaction (SSI) applying SSI elements evaluated by a method proposed by the author and a vibratory experiment conducted after the construction of foundation. The method to evaluate SSI elements from soil consists of 4 steps : an equivalent linearlization of soil against a design earthquake, formulation of force-displacement relationship among nodal points for the wall based on the Thin Layer method (TLM), condensation of the relationship to match with a beam model of the foundation, and an evaluation of SSI elements of a Winkler type derived from LSM. From the experiment it is found that the wall foundation has a high rigidity and the wave dissipation to soil increases with frequency. The simulation analyses for the experimental results verify the validity of the method to evaluate SSI elements.

INTRODUCTION

A 54-story reinforced concrete building is now under construction at Tokyo bay area where deep soil deposits exist. The building is supported by a large size wall foundation with 46.5-meter square, 53-meter depth and 1.8-meter thickness.

For the seismic design of this building, the seismic safety had been verified by earthquake response analyses considering the effect of a soil-foundation-superstructure interaction (SSI). This paper presents on response behavior of the wall foundation during a design earthquake ground motion applying a SSI model based on the Thin Layer method proposed by the author.

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Just after the construction of foundation , a vibratory experiment had been conducted by using a vibration generator. The experimental results on amplitudes and impedances are discussed in comparison with the results of a simulation analysis.

THE OBJECTIVE BUILDING AND FOUNDATION

As shown in Fig.1, the objective is a 54-story reinforced concrete residence building of 174 meters high and the plan of 46.5 meters square with inner void space. This building is constructed by members composed of high strength material of concrete of 100MN/mm^2 and steel bar of 685MN/mm^2 and others and applies a steel damper column of low yielding stress of 225MN/mm^2 for response control to earthquake excitations. The foundation is constructed by a reinforced concrete wall of boxed type and piles shown in Fig.2 and supported by the depth of 53 meters from ground surface. Figure 3 shows the profile of surrounding soil deposit which has deep soft soil layers because the building is located at Tokyo bay area.

Figure 1. Skeleton view of the objective building

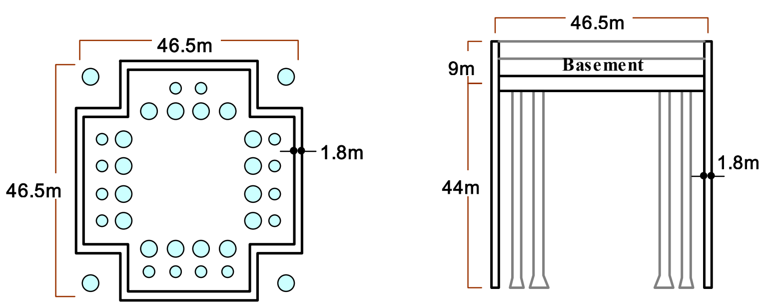


Figure 2. (a) Plan and (b) section of wall and piles foundation

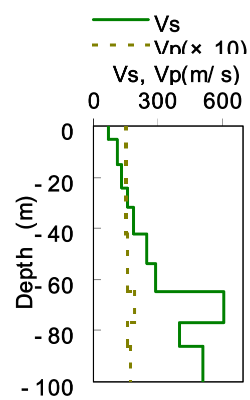


Figure 3. Soil profile

SEISMIC DESIGN

In the seismic design, various earthquake response analyses against a few input motions had been performed : 1) a push-over analysis of the super-structure to evaluate the relationship between story shear force and story drift, 2) response analyses of the super-structure based on a 3-dimensional frame model including a vertical input excitation, 3) response analyses of soil-wall foundation-superstructure interaction system , and 4) a dynamic analysis due to an input excitation with phase difference i.e. a traveling seismic wave. This paper presents the response analysis of a soil-wall foundation-superstructure interaction system (SSI) of the analyses described above.

Figure 4 is the analytical model for the SSI which is composed of the super-structure with a main structure and a steel damper column, the wall foundation and supporting soil elements. The super-structure are modeled into an equivalent beam with flexural-shear deformation converted from the push-over analysis of the frame which has a nonlinear restoring force characteristics on the basis of structural experiments.

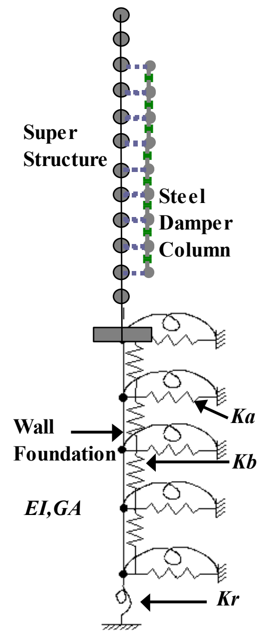


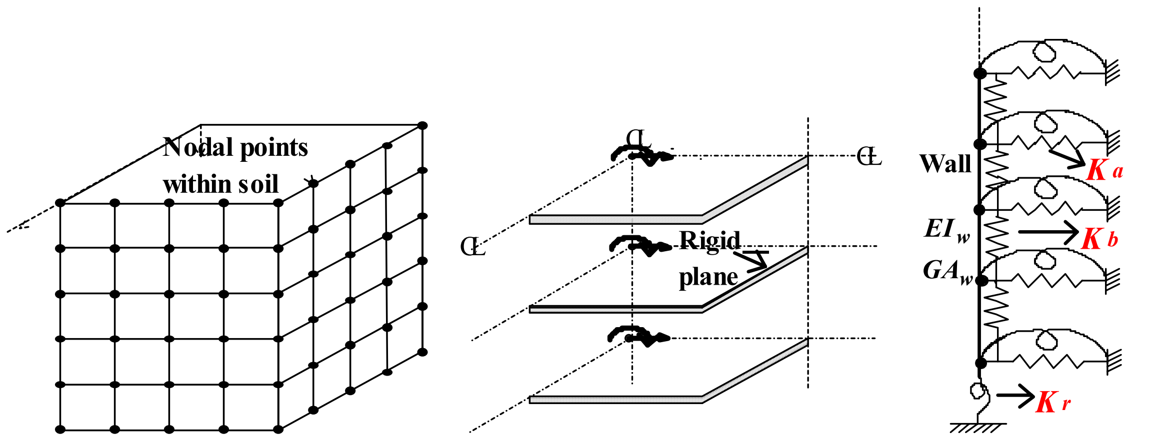
Figure 4. Model for response analyses of soil-wall foundation-superstructure system

A METHOD FOR MODELING OF SOIL AND WALL FOUNDATION INTERACTION SYSTEM

In order to perform earthquake response analyses of soil-wall foundation-superstructure interaction (SSI) system, the soil and wall foundation are modeled into a kind of the beam on continuous springs as the Winkler type. The wall foundation is modeled into a beam with flexural-shear deformation using FEM explained later. The modeling makes possible to perform nonlinear response analyses of a SSI system.

The analytical steps to convert a soil medium into springs connecting with a beam of wall foundation are as follows and the schematic view is illustrated in Fig.5.

Step.1 : To perform an earthquake response analysis of a soil deposit only considering strain dependency of soil i.e. nonlinearity and obtain the equivalent soil rigidity and hysteresis damping of soil due to equivalent linearlization.



(a) Step.2 Relationship of $P_t - U_t$ in soil by 3D-TLM

(b) Step.3 Condensation by rigid plane assumption

(c) Step.4 .Winkler type modeling

Figure 5. Procedure to evaluate spring and dashpot constants of interaction elements represented by Winkler type based on the 3D-Thin Layer method for nonlinear response analyses

Step.2 : To evaluate the relationship at a frequency excited by the function of $e^{i\omega t}$ between forces and displacements among any nodal points within the equivalent soil medium shown in Fig.5(a) where the objective wall foundation is settled, by applying 3-dimensional Thin Layer method called as TLM (Tajimi 1980).

$$\{U_t\} = [f_t^*]\{P_t\} \tag{1}$$

in which P_t and U_t are nodal forces and displacements with the freedoms of nodal points multiplied by 3D, respectively and $[f_t^*]$ is a full flexibility matrix in complex .

Step.3 : To make the condensation of the relationship of Eq.(1) in step.2 into the freedoms of translation and rotation in x-y-z of planes which are assumed to move as rigid-body shown in Fig.5(b) and locate at the same depth with the points discretizing wall foundation. Using the displacements, U_r representing the movement of plane, the displacements, U_t can be expressed as $\{U_t\} = [A]\{U_r\}$, therefore the condensed relationship becomes as follows.

$$\{P_r\} = [K_{sf}^*]\{U_r\}, \quad [K_{sf}^*] = [A]^T [f_t^*]^{-1} [A] \tag{2}$$

where $\left[K_{sf}^{*}\right]$ is a full stiffness matrix in complex. The Eq.(2) might be a rigorous expression based on the TLM.

Step.4 : To evaluate spring constants (includes dashpots) of the Winkler type as shown in Fig.5 (c). We assume that the springs in horizontal response with rotation consist of K_a, K_b and K_r for axial, shear and rotational deflection of soil, respectively and those are vertically in vertical response. Using the springs, the relationship between forces and displacements is expressed as

$$\{P_r\} = \left[K_{sw}{}^{*}\right]\{U_r\} \tag{3}$$

in which the stiffness matrix is given by

$$[K_{sw}^{*}] = [K_a^{*}] + [K_b] + [K_r^{*}] \tag{4}$$

where $\left[K_a^{*}\right]$ and $\left[K_r^{*}\right]$ are diagonal matrixes in complex and $\left[K_b\right]$ is a tri-diagonal one without damping. The spring constants of the Winkler type are estimated from $\left[K_{sf}^{*}\right]$ of Eq.(2) in the following.

Let us consider the condition that a wall foundation, the dynamic stiffness of which is represented by $\left[K_w^{*}\right]$, is subjected by external forces, $\{F_w\}$ and soil responses in free field, $\{U_s\}$. Under the condition, the equation of motion of the wall foundation, $\{U_w\}$ is expressed as follows.

$$[K_w^{*}]\{U_w\} + [K_s^{*}](\{U_w\} - \{U_s\}) = \{F_w\} \tag{5}$$

in which $\left[K_s^{*}\right]$ indicates $\left[K_{sf}^{*}\right]$ in Eq.(2) or $\left[K_{sw}^{*}\right]$ of Eq.(4) . The reaction forces of soil due to $\{F_w\}^T = \{Q, 0, \cdots, 0\}$ of wall top force and $\{U_s\}$ are formulated by

$$\{R_{sQ}\} = [K_s^{*}]\{U_{wQ}\}, \quad \{R_{sG}\} = [K_s^{*}](\{U_{wG}\} - \{U_s\}) \tag{6}$$

where $\{U_{wQ}\}$ and $\{U_{wG}\}$ are the solutions due to $\{F_w\}$ and $\{U_s\}$, respectively. When $\{F_w\}$ and $\{U_s\}$ are given, the solutions of $\{U_{wQ}^{f}\}$ and $\{U_{wG}^{f}\}$ are obtained using the rigorous stiffness $\left[K_{sf}^{*}\right]$ in Eq.(2), that are converted into $\left\{R_{sQ}^{f}\right\}$ and $\left\{R_{sG}^{f}\right\}$ by Eq.(6). Here we consider on $\left\{R_{sQ}^{w}\right\}$ and $\left\{R_{sG}^{w}\right\}$ expressed by using the unkown stiffness of $\left[K_{sw}^{*}\right]$ in Eq.(4) and substituting $\left\{U_{wQ}^{f}\right\}$ and $\left\{U_{wG}^{f}\right\}$ into Eq.(6). The square errors between $\left\{R_{sL}^{w}\right\}$ and $\left\{R_{sL}^{f}\right\}$, L=Q and G, can be expressed as follows.

$$\varepsilon^{2} = \sum_{L=Q,G} w_{L}\left(\{R_{sL}^{w}\} - \{R_{sL}^{f}\}\right)^{T}\left(\{R_{sL}^{w*}\} - \{R_{sL}^{f*}\}\right) \tag{7}$$

in which w_L is weighting factors and the superscript of (*) indicates the conjugate complex values. The Eq.(7) leads to a least square method (LSM) for the unknown spring and dashpot constants of K_a^{*} and K_b as:

$$\varepsilon^{2} \rightarrow \min, \quad \frac{\partial \varepsilon^{2}}{\partial x_{i}} = 0 \quad \text{in any } x_i \text{ of } K_{ai}^{*} \text{ and } K_{bi} \tag{8}$$

Consequently, the unknowns of K_a^{*} and K_b in Fig.5(c) can be estimated. Here we obtain K_r^{*} for rotation from reaction moment in Eq.(2) due to independently given rotation before the procedure described above.

The method to evaluate spring constants from TLM solutions described above can be applied for pile foundations as well (Tohdo 2002).

AN EARTHQUAKE RESPONSE ANALYSIS

An earthquake ground motion is synthesized for the verification of seismic performance of the objective structure shown in Fig.1. The design earthquake is assumed to be a Kanto earthquake shown in Fig.6 which has the seismic moment of $7.6 \cdot 10^{27}$ dyne$\cdot$cm and the fault plane of 130km$\cdot$$70$km. The procedure to estimate earthquake ground motions is a semi-empirical wave synthesis method using observed accelerograms by a small event (Tohdo et al. 1992). Figure 7 is the acceleration wave forms of horizontal and vertical earthquake ground motions at an engineering bed-rock at the site.

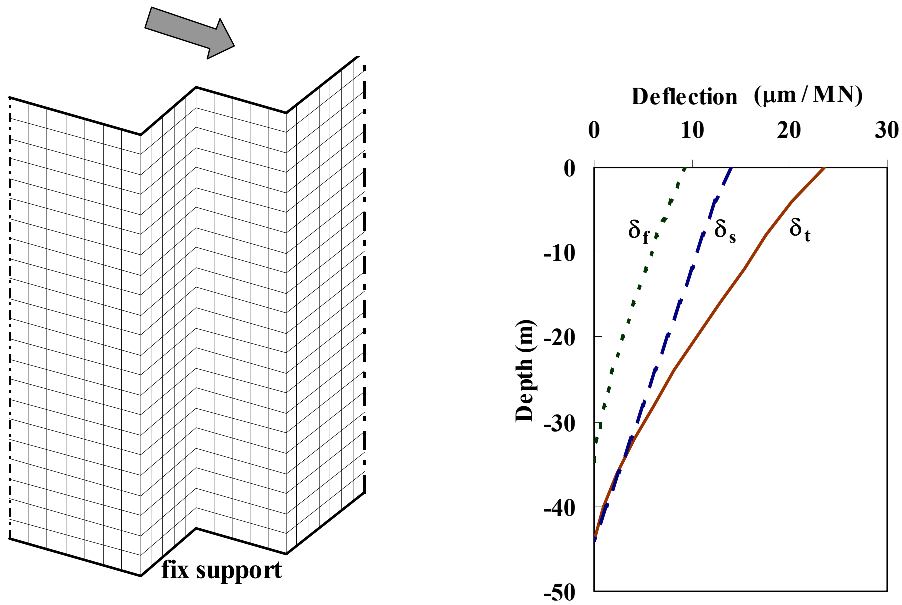


Figure 8. (a) Mesh and (b) deflection of FEM analysis of the pure wall foundation to model into a beam with flexure-shear deformation

In order to model the wall foundation shown in Fig.2 into a beam, an analysis by FEM is carried out, the condition of which is taking the pure wall foundation of boxed-type without soil and a support fixed at bottom as shown in Fig.8(a). The total displacement distribution, δ_t due to a top force is shown by a solid line in Fig.8(b) and the shear-like displacement, δ_s by a dashed line which is obtained under the condition of web-wall only and restriction vertically at top. The shear and flexural rigidities of the modelled beam are determined from δ_s and $\delta_f = \delta_t - \delta_s$, respectively. It is noted here that pile foundations shown in Fig.2 are

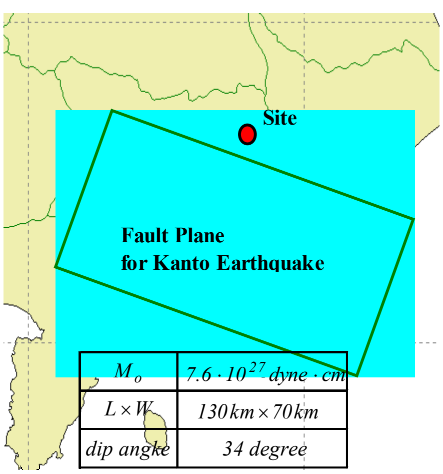


Figure 6. Desgin earthquake

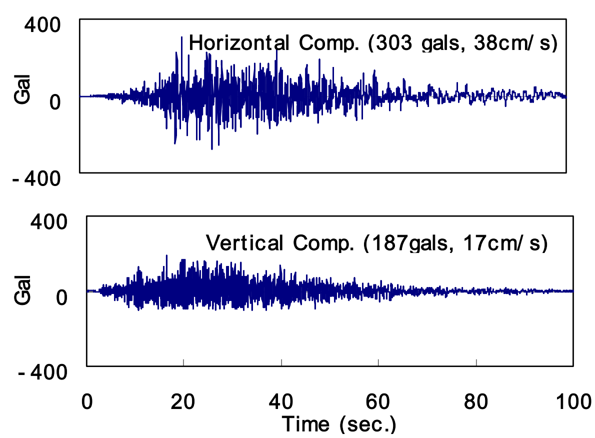
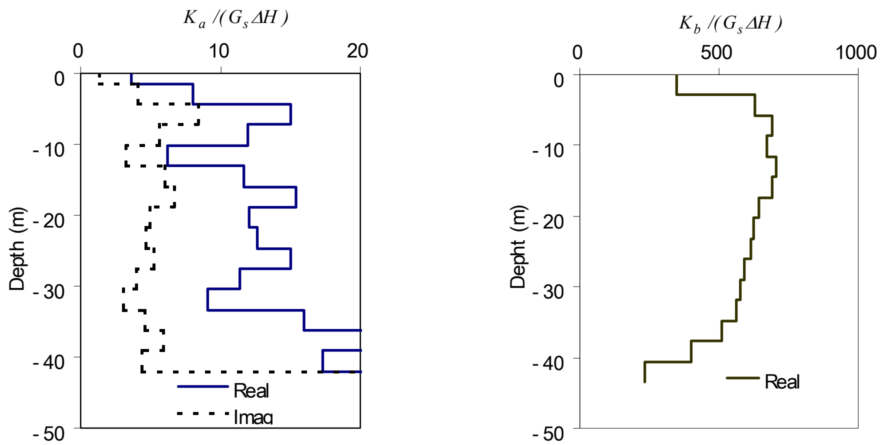


Figure 7. Accelerations at engineering bed-rock for design earthquake synthesized by a fault model

ignored in the SSI analysis because we find the fact derived from the FEM analysis of a wall-pile-soil model that piles share less than 10% of stresses against a top force.



(a) K_a for axial spring (b) K_b for shear spring

Figure 9. Spring constants for interaction elements connecting with wall foundation against design earthquake ground motions shown in Fig.7

The step.1 analysis of soil shown in Fig.3 against the base-rock input of Fig.7 is carried out applying a modified R-O model for restoring force characteristics of soil. The relative displacement and acceleration are shown in Fig.10.

The spring constants of SSI elements due to equivalent linearlized soil are evaluated on the basis of the method explained in step.2 through step.4. In the step.4, the conditions are assumed : 1) $\{U_s\}$ shown in Fig.10(a) and some top Q of $\{F_w\}$, and 2) weighting factors of w_Q and w_G as to be $w_Q \cdot \left\{R^f_{sQ}\right\}_{max} = w_G \cdot \left\{R^f_{sG}\right\}_{max}$ by Eq.(6). The real part of of K^*_a and K_b and the imaginary part of K^*_a are analyzed at the frequency of almost 0, i.e. statically, and the 1st frequency of SSI system, respectively. The results are shown in Fig.9, which are normalized by shear rigidity of soil and element thickness in discretization. The real parts are spring constants and the imaginary parts are converted into dashpots.

At first, the pure wall foundation is subjected to earthquake ground motions due to the input in Fig.7 which in free field are the time histories of response acceleration at base-rock, and velocity and displacement of surface soil ground from top to bottom. Here the vertical response analysis of soil is carried out by assuming P-wave traveling in surface soil (Tohdo et al. 1998). The maximum response displacements and accelerations of wall vary little at depth as shown in Fig.10. Figure 11(a) is maximum shear by horizontal input and axial stress by vertical input which are normalized by weight of wall itself summed from top to the depth. The shear stresses are strongly affected by the difference of response displacement between wall and soil as shown in Fig.10(a). Figure 11(b) shows response spectra by the response acceleration at wall top to be a foundation input motion, and the free field motion at ground surface. The effect of high rigidity of wall foundation appears that the foundation input motions at the period less than fundamental period of soil ground become smaller than free field motions.

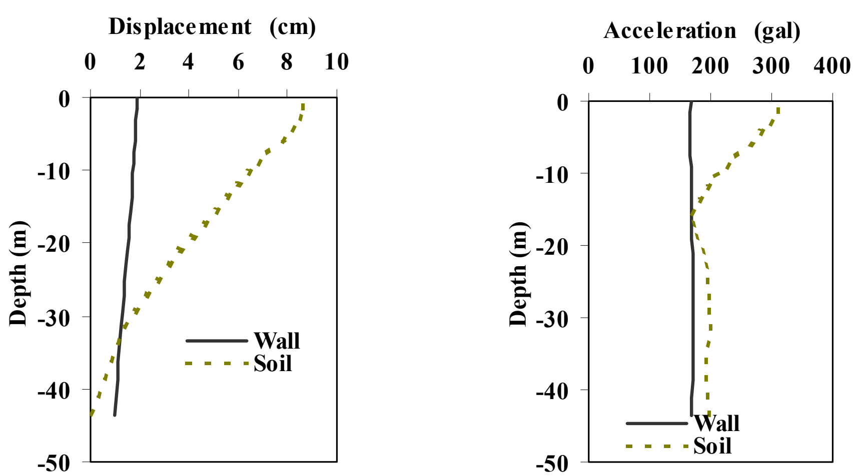
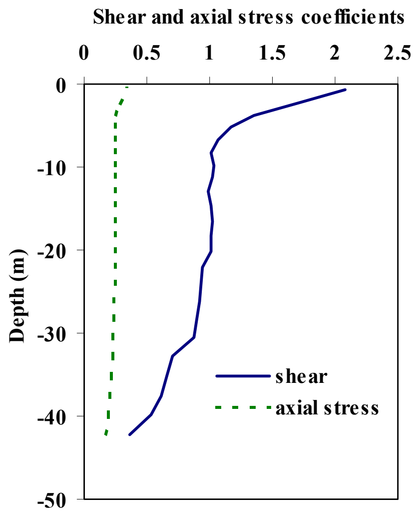


Figure 10. (a) Maximum displacements and (b) accelerations of wall foundation and soil due to response analyses against horizontal earthquake ground motions shown in Fig.7



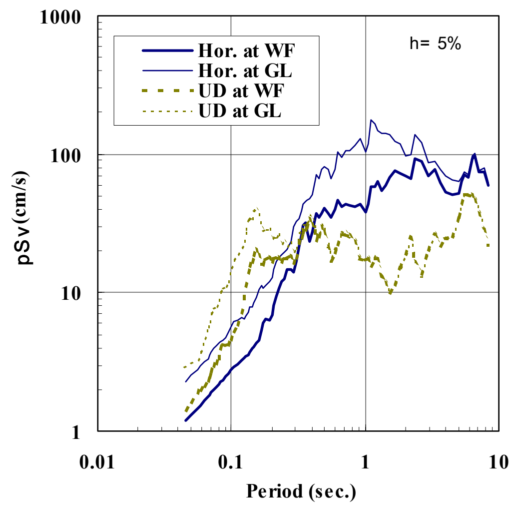


Figure 11. (a) Maximum shear and axial stresses of wall foundation normalized by wall weight, and (b) response spectra obtained by accelerations at the top of wall and free field surface, due to horizontal and vertical earthquake ground motions shown in Fig.7

The earthquake response analysis of the soil-wall foundation-superstructure interaction system is performed. The results of maximum story drift are shown in Fig.12 in comparison with the results by the input of free field surface acceleration. It is recognized in this analysis that the response of the structure becomes fairly small due to the SSI effect, that is, the structure has such seismic performance against this design earthquake.

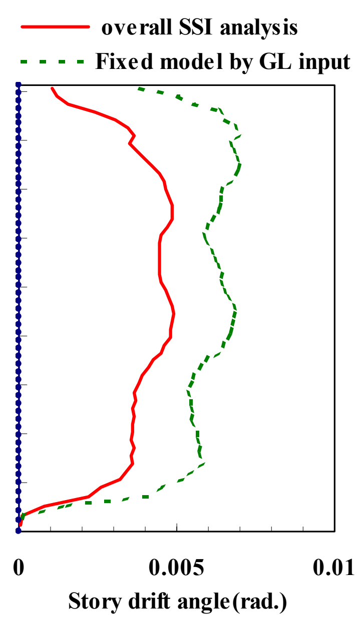


Figure 12. Maximum story drift angles of super-structure due to horizontal earthquake ground motion shown in Fig.7

VIBRATORY EXPERIMENT

An investigation by microtremor observation in free field at the site had been carried out to clarify the elastic wave velocity profile of the soil ground. The spectral ratios of horizontal components to vertical one of microtremors (H/V spectrum) are shown in Fig.13., which have the peak amplitude at frequency of about 0.9Hz and the low amplitude around 2Hz. Using the soil profile obtained from P-S loggings shown in Fig.3, an analysis derived from the Rayleigh surface wave theory is done and its H/V spectrum due to the fundamental mode is shown by a dashed line in Fig.13. Comparing the spectrum by the theory with one by observation, both of the spectrum have similar variation in terms of frequency, that is, the soil profile shown in Fig.3 is accurate.

A vibratory experiment had been conducted just after construction of the foundation by using a vibration generator with unbalanced masses shown in Photo 1 which have the maximum force of 0.03MN and is settled at .the 1st floor. The accelerogragh sensors of horizontal and vertical components are arranged on the wall. Since the order of observed vibration is 1 micron meter, the sensitive vibration of wall foundation is extracted by analyzing the correlation between the signal of excitation and vibratory measurement.

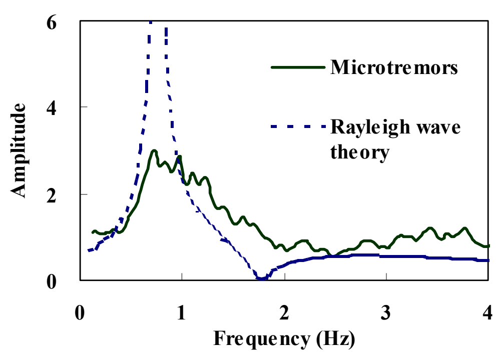


Figure 13. H/V spectrum due to microtremors and Rayleigh wave theory based on the soil profile shown in Fig.3

Photo 1. Vibration generator

The circle marks in Fig.14 show the amplitudes and phase differences taken out for horizontal translation and Figure 15 is the rotational ones obtained from vertical measurement on the web of wall. These amplitudes are converted for excitation force as to be 1MN. It is found in these results that the horizontal and rotational amplitudes are so small and do not have evident resonance, it seems the characteristics of wall foundation with high rigidity, and the phase differences increase gradually as frequency becomes large, that is, the wave dissipation from wall to surrounding soil increases.

A simulation analysis against experimental results is conducted on the basis of the method described in the above section. Applying the condensed stiffness in Eq.(2) based on

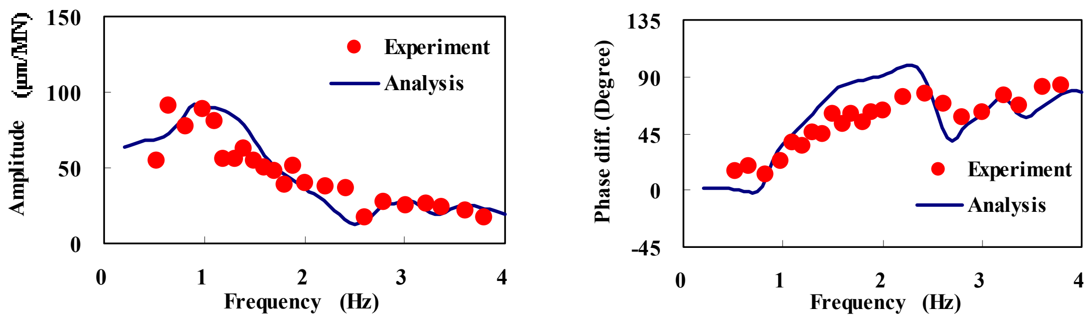


Figure 14. (a) Amplitude and (b) phase difference of horizontal displacement subjected to a lateral force at the top of wall

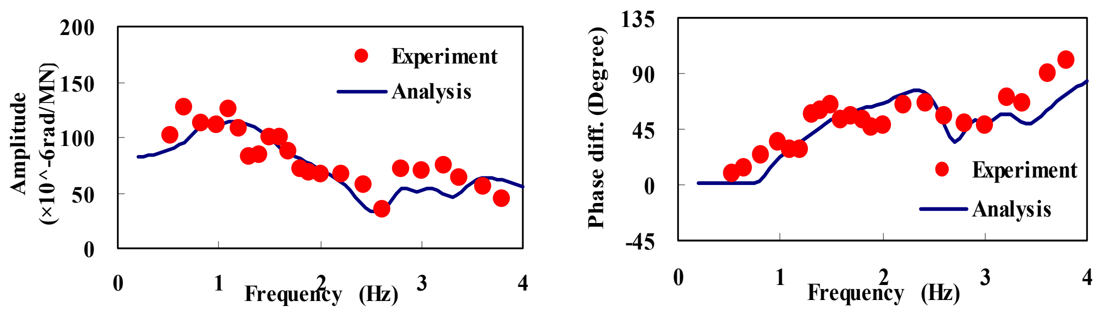


Figure 15. (a) Amplitude and (b) phase difference of rotational displacement subjected to a lateral force at the top of wall

the TLM, the movement of wall foundation can be written by

$$\left([K_w^*]+[K_{sf}^*]\right)\{U_w\} = \{F_w\} \tag{9}$$

From Eq.(9), the impedance function between forces and displacements at the top of the wall foundation is obtained as follows.

$$\begin{Bmatrix} P \\ M \end{Bmatrix} = \begin{bmatrix} K_{HH} & K_{HR} \\ K_{RH} & K_{RR} \end{bmatrix} \begin{Bmatrix} U \\ \Theta \end{Bmatrix} \tag{10}$$

where $K_{HR} = K_{RH}$. Therefore the displacements due to a force, P of the generator are expressed by

$$\begin{Bmatrix} U \\ \Theta \end{Bmatrix} = \begin{bmatrix} \mathrm{K_{HH}} & \mathrm{K_{HR}} \\ \mathrm{K_{RH}} & \mathrm{K_{RR}} \end{bmatrix}^{-1} \begin{Bmatrix} \mathrm{P} \\ 0 \end{Bmatrix} \tag{11}$$

The solid lines in Figs.14 and 15 are the simulated results by Eq.(11) which agree well with the observed results.

Next, we discuss on the impedance functions derived from the observed data. It is noted here that the unknown 3-components of the impedance can not be obtained directly from observed data, because the excitation by generator is horizontal only, i.e. the equations are composed of 2-unknowns only with shortage. So we assume the relationship of Eq.(12) which is obtained using the analytical impedance of Eq.(10).

$$\mathrm{r_K} = \frac{\mathrm{K_{HR}}}{\sqrt{\mathrm{K_{HH}K_{RR}}}} \tag{12}$$

Using this relation, the diagonal terms of impedance can be obtained by

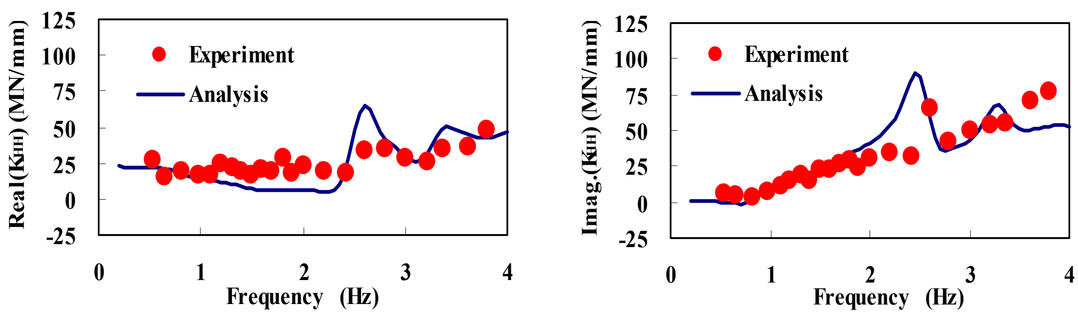


Figure 16. (a) Real and (b) imaginary parts of horizontal translation impedance, K_{HH}

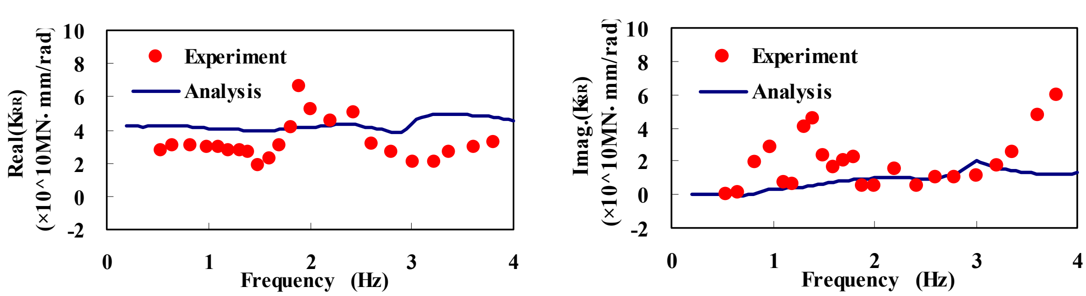


Figure 17. (a) Real and (b) imaginary parts of rotational impedance, K_{RR}

$$K_{HH} = \frac{P}{U} \frac{1}{1 - r_K^2}, \quad K_{RR} = \frac{PU}{\Theta^2} \frac{r_K^2}{1 - r_K^2} \tag{13}$$

in which P is a excited force, and U and Θ are observed displacements. The resulted impedances are shown by circles in Figs.16 and 17 in comparison with the analytical impedance. Although K_{RR} has rather differences because of the indirect part due to horizontal excitation, both of K_{HH} have well agreement.

From these comparisons between observed data by experiments and the analytical results, it is recognized that the method applied here on the basis of TLM is appropriate for SSI analyses in seismic design.

CONCLUSIONS

The study on the dynamic behavior of a large size wall foundation supporting a 54-story building was presented, which considers the effect of a soil-foundation-superstructure interaction (SSI). The contents are summarized as follows.

From an earthquake response analysis of the SSI system in the seismic design, it is recognized that the structure is strongly influenced by SSI effects such that the response displacements of the wall foundation are remarkable smaller than those of soil ground in free field, and a foundation input motion into the super-structure is suppressed, consequently the response of the super-structure becomes small.

A vibratory experiment for the wall foundation has made clear the dynamic characteristics such that horizontal and rotational amplitudes at the top of wall are so small and do not have evident resonance and the phase differences increase gradually with frequency, that is, the wave dissipation from wall to surrounding soil increases. The results are simulated well by the method which was applied for the earthquake response analysis. This might show that the method applied for SSI analyses in seismic design is appropriate.

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