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# A PRACTICAL EVALUATION METHOD OF SEISMIC LOAD CONSIDERING SOIL STRUCTURE INTERACTION EFFECTS

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## SUMMARY

A practical method to evaluate seismic load considering soil-structure interaction (SSI) effects is presented. The contents are : 1) to obtain seismic loads of MDOF system from equivalent SDOF by using fixed base eigen modes , 2) to calculate sway and rocking springs and dashpots for flat, embedded and pile foundations in which multi-layered soil effects are taken into account, 3) to obtain foundation input motions of embedded foundation. These practical methods are examined on validity in comparison with numerical calculations by the more rigorous method.

## INTRODUCTION

Architectural Institute of Japan (AIJ) has now prepared the draft for revision of AIJ recommendations of seismic load on buildings in 2004. The overview is presented in this WCEE [Ishiyama et al. 2004]. The draft includes the soil-structure interaction (SSI) effects on kinematic and inertial SSI problems. The basic procedure to evaluate seismic load in this draft is based on the SRSS method. In the draft the practical methods on SSI are proposed together with analytical methods such as FEM. This paper presents the practical methods. One is to obtain seismic loads of MDOF system of buildings from equivalent SDOF system connecting with SSI springs and dashpots by applying eigen modes of fixed base structures. The method can be applied for the capacity spectrum method proposed in revised Japanese national code. Next, practical formula to calculate sway and rocking springs and dashpots are proposed on directly settled, embedded and pile foundations. In those formula, the effects of multi-layered soil deposits are taken into account for determination of soil parameters on rigidity, shear wave velocity for radiation damping and hysteretic damping of soil. The formula for pile foundation are expressed on the basis of the concept of beam on elastic media and consider pile-group effect. The last proposal is on the foundation input motion for kinematic problem. The input motions of buildings subjected to Hyogoken Nanbu earthquake in 1995 had been investigated by a special project group of AIJ using observed earthquake motions. Based on this investigation, the AIJ's draft proposes a formulae on input motions of embedded

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foundations with the parameters of non-dimensional embedded depth and frequency in terms of ground predominant frequency. These practical methods are examined by numerical examples using the Thin Layer method and so on, and then the differences are discussed in this paper.

## GENERAL EXPRESSION OF SEISMIC LOADS

The seismic loads that are story shear force of the i-th level of a n-story structure are expressed by [Ishiyama et al. 2004]

$$V_{Ei} = k_{Di} k_{Fi} \sqrt{\sum_{j=1}^{j_c} V_{ij}^2} \tag{1}$$

where $k_{Di}$ is the structural factor to be specified according to ductility and overstrength, $k_{Fi}$ the shape factor determined from vertical rigidity distribution and eccentricity in plane. The $V_{ij}$ indicates the story shear force derived from the j-th eigen mode of vibration in the method of square root of sum of squares (SRSS), that is

$$V_{ij} = S_a(T_j, \zeta_j) \sum_{k=i}^{n} m_k \beta_j \phi_{kj} \tag{2}$$

where $S_a(T_j, \zeta_j)$ is the design acceleration response spectra in the j-th mode with natural period $T_j$ and damping ratio $\zeta_j$, $m_i$ the mass of i-th story. The mode shapes $\phi_{ij}$ and participation factor $\beta_j$ together with $T_j$ and $\zeta_j$ are determined from the eigenvalue problem analysis of the following dynamic equations of motion of the sway and rocking model shown in Fig.1.

$$[m]\{\ddot{x}\} + [c]\{\dot{x}\} + [k]\{x\} = -[m]\{e\}\ddot{y}$$

Using the $\phi_{ij}$ defined as Fig.2, the $\beta_j$ and $\zeta_j$ are calculated in the followings.

$$\beta_j = \frac{\{\phi_j\}^T [m]\{e\}}{M_j} \tag{3}$$

$$\zeta_j = \frac{C_j}{2\sqrt{M_j K_j}} \tag{4}$$

where $M_j, C_j$ and $K_j$ are the generalized mass, damping and stiffness of j-th mode, respectively.

$$M_j = \{\phi_j\}^T [m]\{\phi_j\} \quad , \quad C_j = \{\phi_j\}^T [c]\{\phi_j\} \quad , \quad K_j = \{\phi_j\}^T [k]\{\phi_j\}$$

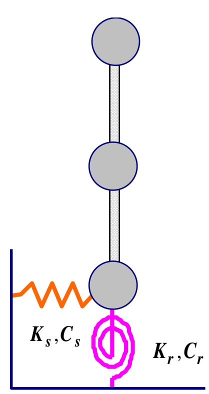



Fig.1 Sway-rocking model of MDOF

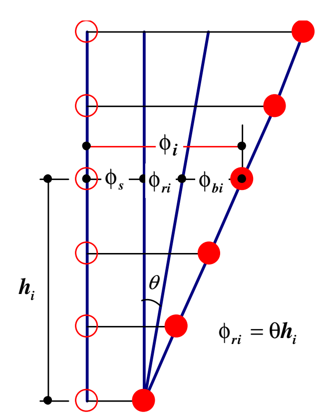



Fig.2 Definition of mode shapes

## EXPRESSION BY EQUIVALENT SDOF MODEL

**Proposed method**

The replacement of a MDOF super-structure system into the equivalent SDOF model shown in Fig.3 is expressed using the equivalent mass, stiffness and height obtained by

$$\overline{M}_{fj} = \sum_{k=1}^{n} m_k \beta_{fj} \phi_{fkj} \quad , \quad \overline{K}_{fj} = \omega_{fj}^2 \overline{M}_{fj} \quad , \quad \overline{h}_{fj} = \sum_{k=1}^{n} m_k \beta_{fj} \phi_{fkj} h_k / \overline{M}_{fj} \tag{5}$$

where $\phi_{fij}$ is the j-th mode shape of vibration in the condition of fixed base, $\beta_{fj}$ is obtained by the same way in Eq.(3) using $\phi_{fij}$ and $\omega_{fj}$ is a natural circular frequency of the super-structure.

The story shear force with the SSI effect corresponding to Eq.(1) is expressed by :

$$V_{Ei} = k_{Di} k_{Fi} \sqrt{V_{i1}^2 + \Delta V_i^2} \tag{6}$$

The $V_{i1}$ is the 1st mode contribution and $\Delta V_i$ the higher mode. They are obtained by Eqs.(7) and (8), respectively.

$$V_{i1} = S_a(T_1, \zeta_1) \sum_{k=i}^{n} m_k \beta_1 \phi_{k1} \tag{7}$$

$$\Delta V_i = S_a(T_2, \zeta_2) \left( \sum_{k=i}^{n} m_k - \sum_{k=i}^{n} m_k \beta_1 \phi_{k1} \right) \tag{8}$$

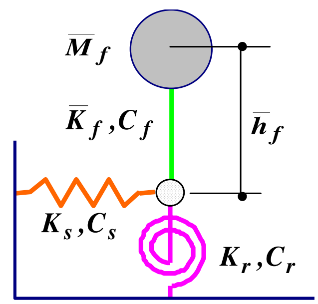



Fig.3 Equivalent SDOF model of super-structure

in which $S_a(T_j, \zeta_j)$ is an acceleration response spectrum in the j-th mode. The $T_j$ and $\zeta_j$ are obtained considering the SSI effect in the followings. Since super-sturcture, sway and rocking springs shown in Fig.3 are connected with each others in a series, $T_j$ becomes

$$T_j = \sqrt{T_{fj}^2 + T_{sj}^2 + T_{rj}^2} \tag{9}$$

where $\quad T_{sj} = 2\pi \sqrt{\dfrac{\overline{M}_{fj}}{K_s}} \quad , \quad T_{rj} = 2\pi \sqrt{\dfrac{\overline{M}_{fj} \overline{h}_{fj}^2}{K_r}}$

The damping ratio $\zeta_j$ is approximately obtained by

$$\zeta_j = \zeta_{fj} \left( \frac{T_{fj}}{T_j} \right)^3 + \zeta_{sj} \left( \frac{T_{sj}}{T_j} \right)^3 + \zeta_{rj} \left( \frac{T_{rj}}{T_j} \right)^3 \tag{10}$$

where $\quad \zeta_{fj} = \dfrac{C_f}{2\sqrt{\overline{M}_{fj} \overline{K}_{fj}}} \quad , \quad \zeta_{sj} = \dfrac{C_s}{2\sqrt{\overline{M}_{fj} K_s}} \quad , \quad \zeta_{rj} = \dfrac{C_r}{2\sqrt{\overline{M}_{fj} \overline{h}_{fj}^2 Kr}}$

The relationship of Eq.(10) is derived from the following expression for a damping ratio.

$$\zeta_j = \frac{1}{4\pi} \frac{\Delta W_j}{W_j}$$

The $W_j$ and $\Delta W_j$ are strain and absorbing energy during a cycle obtained from

$$W_j = \frac{1}{2} \left\{ \overline{K}_{fj} \phi_{bj}^2 + K_s \phi_{sj}^2 + K_r \theta_j^2 \right\} \quad , \quad \Delta W_j = \pi \omega_j \left\{ C_f \phi_{bj}^2 + C_s \phi_{sj}^2 + C_r \theta_j^2 \right\}$$

where $\quad \phi_b : \phi_s : \phi_r = \left( \dfrac{T_{fj}}{Tj} \right)^2 : \left( \dfrac{T_{sj}}{T_j} \right)^2 : \left( \dfrac{T_{rj}}{T_j} \right)^2$

The 1st mode shapes $\phi_{i1}$ in Eqs.(7) and (8) with SSI effect based on the fixed base mode are approximated as follows.

$$\phi_{i1} = \phi_{fi1} + \phi_{s1} + \phi_{r1}\frac{h_i}{\bar{h}_{f1}}$$

(11)

where $\phi_{s1} = \dfrac{1}{\beta_{f1}}\left(\dfrac{T_{s1}}{T_{f1}}\right)^2$ , $\phi_{r1} = \dfrac{1}{\beta_{f1}}\left(\dfrac{T_{r1}}{T_{f1}}\right)^2$

**Examination for validity**

The accuracy of the proposed method is examined here. The conditions to calculate examples are shown in Table 1. The relationship between $T_{f1}$ with fixed base and the story number n is average for reinforced concrete frame structures in Japan. Spring and dashpot constants for sway and rocking are calculated by Eqs.(13) and (14). Figure 4 shows the results on natural periods in which the approximate are obtained by Eq.(9) and the exact by eigenvalue analysis of undamped MDOF in Fig.1. Figure 5 shows the results on modal damping ratios in which the approximate are obtained by Eq.(10) and the exact by Eq.(4) using undamped mode shapes. Figure 6 shows the results on effective masses at the 1st floor of the 1st mode in which the approximate are calculated applying the approximate mode by Eq.(11) and the exact by using eigenvalue analysis of MDOF. From these comparisons it is recognized that the proposed method using

Table 1 Conditions for calculation

*Super-stucture* (n-story shear model)

- Weight distribution : 11,500kN constatnt
- Stiffness distribution : linear $k_1 : k_n = 2:1$
- Story height : 3.5m
- 1st natural period : $T_{f1} = 0.07n(sec.)$
- Damping ratio: $\zeta_{fj} = 0.03\omega_{fj} / \omega_{f1}$

*Foundation and soil*

- Foundation settled on surface : 30m squares
- Soil : Uniform with Vs of 100m/sec.

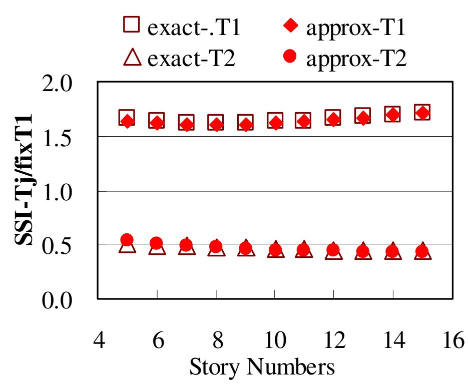



Fig.4 Approximate natural periods and exact ones

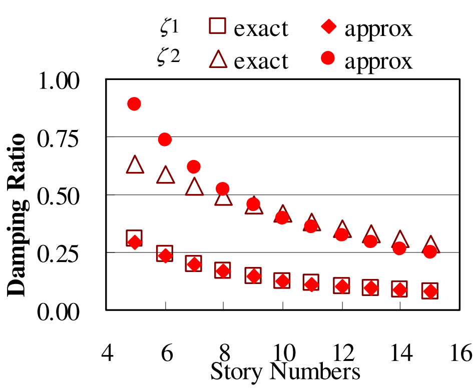



Fig.5 Approximate modal dampings and exact ones

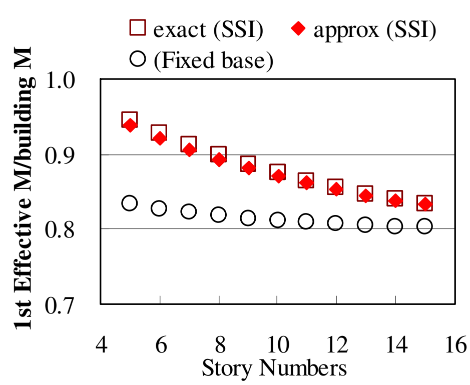



Fig.6 Approximate effective masses and exact ones of the 1st mode

the equivalent SDOF model can evaluate almost same story shear forces with the general expression.

## SPRING AND DASHPOT CONSTANTS FOR SSI

### Evaluation from Impedance functions

Of flat, embedded and pile foundations the spring and dashpot constants for sway and rocking motions shown in Fig. 1 or 3 can be obtained from the relationship between force and displacement at the base of super-structure , that is, when a force is $P = P_o e^{i\omega t}$ and the displacement $U = U_o e^{i\omega t}$ at the base such as the pile top, the impedance function $K_{imp}(\omega)$ is expressed :

$$K_{imp}(\omega) = P / U = K(\omega) + iK'(\omega)$$

where $K(\omega)$ and $K'(\omega)$ are the real and imaginary parts of a complex impedance function, respectively. In this practical method, the spring and dashpot constants, $K_d$ and $C_d$, are defined in Eq.(12) derived from a impedance function as shown in Fig.7.

$$K_d = K(\omega = 0) \quad , \quad C_d = \frac{K'(\omega = \omega_1)}{\omega_1} \tag{12}$$

where $\omega_1$ is the 1st natural circular frequency of the interaction model. The definitions in eq.(12) indicate that $K_d$ is calculated statically and $C_d$ is determined as the damped force by a dashpot at $\omega_1$ becomes

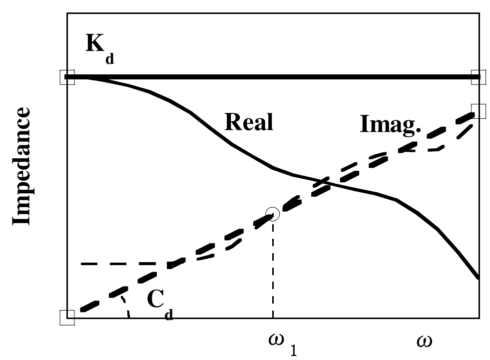



Fig.7 Definition of spring and dashpot constants from an impedance function

the same one using an impedance function.

The impedance function or the constant are evaluated applying the method based on the wave propagation theory such as Thin Layer method, finite element method with viscous boundary or discrete spring type method [AIJ 1996]. The draft recommends an practical method to evaluate impedance functions for applying Eq.(12) as follows. The subscripts of (s) and (r) in the following indicate sway and rocking motion, respectively. The soil property is represented by shear modulus $G$ , mass density $\rho$ , shear wave velocity $V_s$ and intrisic hysteresis damping ratio $\zeta_g$ , which for earthquake motion levels are converged into each value against strain dependency by equivalent linearlization.

**Flat foundation on ground surface**

*Proposed method*

The spring constants for flat foundation are obtained by

$$K_{bs} = \frac{8Gr_s}{2-\nu} \quad , \quad r_s = \sqrt{\frac{A_f}{\pi}} \qquad (13a) \qquad\qquad K_{br} = \frac{8Gr_r^3}{3(1-\nu)} \quad , \quad r_r = \sqrt[4]{\frac{4}{\pi} I_f} \qquad (13b)$$

where $A_f$ and $I_f$ are the area and the second moment of cross-section of a flat foundation, respectively. The $\nu$ is Poisson's ratio of soil. The imaginary part, $K'(\omega)$ can be approximated as [Iiba et al, 2000]

$$\begin{aligned} K'_{bs}(\omega_1) &= 2\varsigma_g K_{bs} && for \quad \omega_1 \le \omega_{g1} \\ &= 2\varsigma_g K_{bs} + (\omega_1 - \omega_{g1})\rho V_s A_f && for \quad \omega_1 > \omega_{g1} \end{aligned} \qquad (14a)$$

$$\begin{aligned} K'_{br}(\omega_1) &= 2\varsigma_g K_{br} && for \quad \omega_1 \le 2\omega_{g1} \\ &= 2\varsigma_g K_{br} + (\omega_1 - 2\omega_{g1})\rho V_L I_f && for \quad \omega_1 > 2\omega_{g1} \end{aligned} \qquad (14b)$$

where $\omega_{g1}$ is the predominant circular frequency of a layered soil as shown in Fig.8 and $V_L$ is the apparent velocity of $3.4V_s/\pi/(1-\nu)$ defined by Lysmer.

The $G$ in Eq.(13) in a layered soil is represented by the modification in Eq.(15) [Tajimi 1968].

$$\frac{1}{G} = \sum_{l=1}^{n} \frac{1}{G_l} \left\{ F_{bs}\left(\frac{z_{l-1}}{r_s}\right) - F_{bs}\left(\frac{z_l}{r_s}\right) \right\} \qquad (15)$$

where $F_{bs}(\chi) = \frac{1}{3}\left[ \frac{3+4\chi^2}{\sqrt{1+\chi^2}} - 4\chi \right]$ and

$$z_0 = 0 \quad , \quad F_{bs}\left(\frac{z_n}{r}\right) = 0 \, .$$

The $\zeta_g$ in Eq.(14) is modified by Eq.(16).

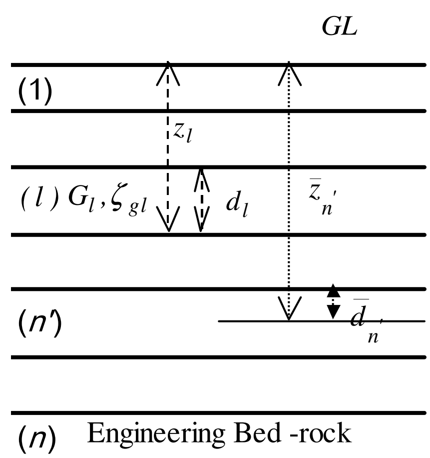



Fig.8 Definition of layered Soil

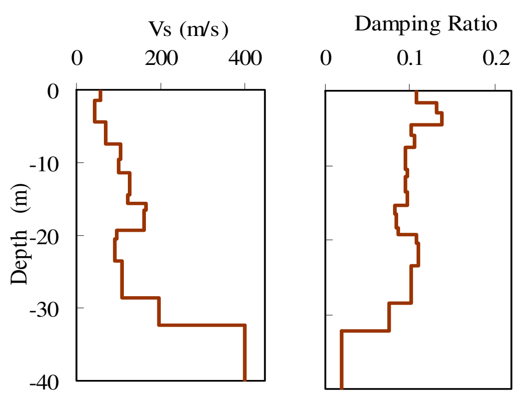



Fig.9 Soil profile for an example

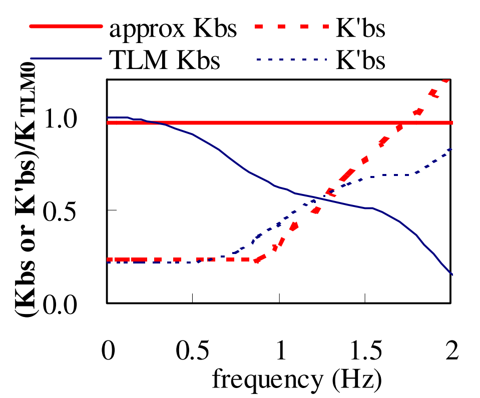



Fig.10 Example of impedance functions of a flat foundation

$$\varsigma_g = \sum_{l=1}^{n} \frac{G}{G_l} \varsigma_{gl} \left\{ F_{bs}\left(\frac{z_{l-1}}{r_s}\right) - F_{bs}\left(\frac{z_l}{r_s}\right) \right\} \tag{16}$$

*Examination for validity*

An example of flat foundation with 30m squares on soil ground shown in Fig.9 ( $f_g = 0.9Hz$ ) is compared for the sway impedance function with the result by 3-D Thin Layer method(TLM) [Tajimi 1980]. As shown in Fig.10 in which the denominator $K_{TLMo}$ is the real part value by the TLM at 0.01Hz, the comparison shows that $K_{bs}$ by Eq.(13a) is good agreement with the TLM result at near $\omega = 0$ and $K'_{bs}(\omega)$ by Eq.(14a) has a similar tendency in terms of frequency with the TLM result.

**Embedded foundations**

*Proposed method*

The impedance functions for embedded foundations with section area of $A_f$ and embedded depth of $d$ are obtained by the sum of impedances of the side wall, $K_w$ and the bottom plane, $K_b$ by Eqs.(13) and (14). The spring constants for side wall are given by Eq.(17) of the product of $K_b$ and some coefficients which were proposed using the investigation by rigorous Green's function method [Tohdo et al. 1986].

$$K_{ws} = K_{bs} 2\eta \tag{17a}$$

$$K_{wr} = K_{br}(2.6\eta + 5.6\eta^3) \tag{17b}$$

where $\eta = \dfrac{d}{\sqrt{A_F}}$.

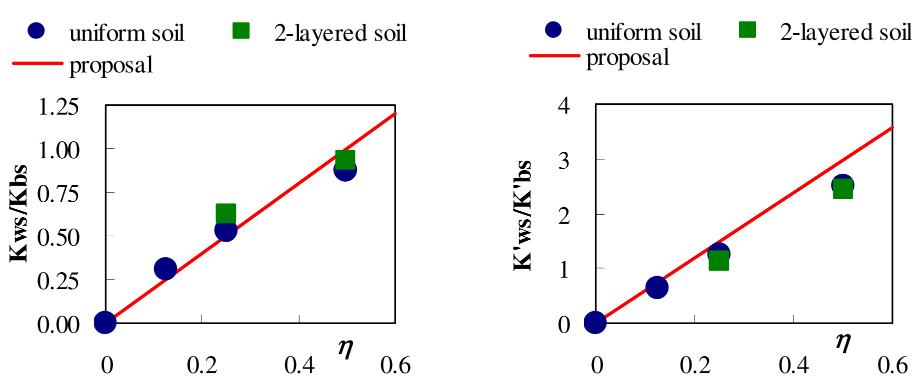
The imaginary part of side wall for sway is obtained by Eq.(18) [Fukuwa et al. 1987].

$$K'_{ws}(\omega_1) = 2\xi_g K_{bs} + \omega_1 (\rho V s A_1 + \rho V_L A_2) \tag{18}$$

where $A_1$ and $A_2$ are the area of parallel and perpendicular side wall against the direction of motion, respectively. It is noted here that the average soil property of surface ground i.e. soil supporting side-wall is applied to calculate $K_b$ in Eqs.(17) and (18).

*Examination for validity*

The accuracy of the proposed formula of Eqs.(17a) and (18) for sway is investigated in comparison with the results by TLM. The condition of examples are that a foundation plane is square and soils with no




(a) Real part (b) Imaginary part

Fig.11 The relationship between normalized depth $\eta$ and impedance functions of side wall for embedded foundations intrisic damping are uniform or 2-layered medium, bed rock beneath foundation base of which has the twice shear wave velocity of surface one. The $K_{ws}$ and $K_{ws}^{'}$ by TLM shown in Fig.11 are calculated by subtracting flat foundation impedances on surface of bed-rock from the total and by averaging the impedances between 0 and 2 of $\omega\sqrt{A_f}/V_s$. The proposed formula in Fig.11 agree well with the results by TLM.

**Pile foundations**

*Basic formulation*

Let us consider a beam for a pile of a pile-group supported by visco-elastic medium as shown in Fig.12. The equation of the beam in terms of motion, $U$ can be written as :

$$E_P I_P \frac{\partial^4 U}{\partial z^4} + c_{gN}\frac{\partial U}{\partial t} + k_{gN}^{*} U = -\rho_P A_P \frac{\partial^2 U}{\partial t^2} \tag{19}$$

in which $E_P I_P$ is the flexural rigidity of pile, $\rho_P A_P$ mass per unit length and $k_{gN}$ and $c_{gN}$ are stiffness and radiation damping coefficients per unit length derived from supporting soil respectively, for which we assume to be a function of pile-group factor, $\gamma_P$ defined later as follows.

$$k_{gN} = \gamma_P k_g \quad , \quad c_{gN} = \gamma_P^{1/4} c_g \tag{20}$$

$k_{gN}^{*}$ is the complex stiffness including hysteresis damping effect,

$$k_{gN}^{*} = (1 + i2\zeta_g)k_{gN}$$

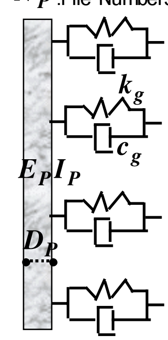



Fig.12 Pile foundation

The complementary solution of Eq.(19) becomes

$$U = exp(\beta^{*} z)\{C_1 \sin(\beta^{*} z) + C_2 \cos(\beta^{*} z)\} + exp(-\beta^{*} z)\{C_3 \sin(\beta^{*} z) + C_4 \cos(\beta^{*} z)\} \tag{21}$$

where $\beta^{*} = \left(\frac{-\rho_P A_P \omega^2 + i\omega c_{gN} + k_{gN}^{*}}{4E_P I_P}\right)^{1/4}$ and $C_1 - C_4$ are some coefficients determined from boundary conditions. Therefore, the dynamic complex spring at the pile top $K^{*}$ is obtained from the relationship between pile top force $P_o$ and displacement $U_o$ under the assumption of long pile based on the Chan's solution as follows.

$$K^{*} = \frac{P_o}{U_o} = \frac{4E_P I_P \beta^{*3}}{2-\alpha}$$

where $\alpha$ is the pile top condition. Here assuming $\rho_P A_P$ is small, the $K^{*}$ is approximated using the static spring constant, $K_o$ with the pile characteristic value, $\beta$ in Eq.(23) :

$$\begin{aligned} K^{*} = K_o\left(\frac{\beta^{*}}{\beta}\right)^3 &\cong K_o(1 + i1.5\zeta_g)\left\{1 + i\omega 0.75\frac{c_{gN}}{k_{gN}}\right\} \\ &\cong K_o + i\left\{1.5\zeta_g + \omega 0.75\frac{c_{gN}}{k_{gN}}\right\}K_o \end{aligned} \tag{22}$$

*Proposed method*
Following the basic formulation, the spring constant, $K_{ps}$ for sway motion at the top of a pile-group with the number of $N_P$ is expressed as

$$K_{ps} = N_P \frac{4E_P I_P \beta^3}{2-\alpha} \quad , \quad \beta = \sqrt[4]{\frac{\gamma_P k_g}{4E_P I_P}} \tag{23}$$

where $\alpha$ is 1 for fixed support and 0 for pin support at the pile top. The pile-group factor, $\gamma_P$, we assume, is given by Eq.(24) in a mean sense considering parameters due to actual foundations on the basis of investigations associated with $N_P$, pile diameter and spacing [Hasegawa et al. 1990, Hijikata et al. 1997].

$$\gamma_P = N_P^{-0.5} \tag{24}$$

The exponent of 1/4 in Eq.(20b) has a similar meanings. The imaginary part of impedance function is given by Eq.(25) based on Eq.(22) and taking into account the cut-off effect due to the predominant circular frequency of a layered soil, $\omega_{g1}$.

$$\begin{aligned} K'_{ps}(\omega_1) &= 1.5\varsigma_g K_{ps} && for \quad \omega_1 \le \omega_{g1} \\ &= 1.5\varsigma_g K_{ps} + (\omega_1 - \omega_{g1}) 0.75\gamma_P^{-0.75} \frac{c_g}{k_g} K_{ps} && for \quad \omega_1 > \omega_{g1} \end{aligned} \tag{25}$$

The stiffness and radiation damping coefficients due to soil for single pile are obtained by Eq.(26a) [Francis 1964] and Eq.(26b), respectively.

$$k_g = \frac{1.3E_s}{1-\nu^2} \sqrt[12]{\frac{E_s D_P^4}{E_P I_P}} \quad (26a) \qquad\qquad c_g = 0.5\pi\rho D_p (V_s + V_L) \quad (26b)$$

where $E_s$ is the Young's modulus of soil and $D_P$ the diameter of a pile.
The application of Eq.(23) for the spring constant of a pile-group in a layered soil shown in Fig.8 can be performed as follows [Tohdo 2003]. The characteristic value, $\beta$ in Eq.(23) is determined by

$$\frac{1}{\beta^3} = \sum_{l=1}^{n'} \frac{1}{\beta_l^3} \left\{ F_p(z_{l-1}) - F_p(z_l) \right\} \tag{27}$$

in which $\beta_l = \sqrt[4]{\gamma_P k_{gl} / 4E_P I_P}$ for each soil layer and the function for the depth, z in Fig.8 which indicates a shape factor is defined as:

$$F_p(z) = exp(-\overline{\beta} z) cos(\overline{\beta} z) \tag{28}$$

where $\overline{\beta} = \frac{0.5\pi}{\overline{z}_{n'}}$, $z_0 = 0$ and $F_p(z_{n'}) \equiv F_p(\overline{z}_{n'})$. The $\overline{z}_{n'}$ shown in Fig.8 is given by

$$\overline{z}_{n'} = z_{n'-1} + \overline{d}_{n'} \qquad (\le L_p)$$

using the $\overline{d}_{n'}$ at a layer $n'$ which satisfies $\sum_{l=1}^{n'-1} \beta_l d_l + \beta_{n'} \overline{d}_{n'} = 0.5\pi$. For the imaginary part of Eq.(25), the $\zeta_g$ is obtained by

$$\varsigma_g = \sum_{l=1}^{n'} \left( \frac{\beta}{\beta_l} \right)^{0.75} \varsigma_{gl} \left\{ F_p(z_{l-1}) - F_p(z_l) \right\} \tag{29}$$

The coefficients, $k_g$ and $c_g$ in Eq.(25) are determined from : (1) inversely calculate $k_g$ by Eq.(23b) from $\beta$ in Eq.(27), (2) obtain $c_g$ in Eq,(26b) using $V_s$ from $E_s$ inversely calculated in Eq.(26a).

*Examination for validity*

The validity of the proposed method for pile foundations is examined here. At first, we examine on the modification method of spring constants, $K_{ps}$ for a multi-layered soil effect mainly described in Eq.(27). The conditions for numerical examples are 36 cases in each pile top condition of fixed or pinned support : 6 kinds of soil deposit shown in Fig.13, pile foundations with $D_P$ of 1m or 2m and $N_P$ of 1, 9 or 16. Results by the proposed approximate method are shown in Fig.14 in comparison with $K_{ps}$ obtained by the exact method for which the solution under the pile condition buried in 2m into base-rock is obtained by applying Eq.(21) in each layer and considering boundary conditions on displacements and stresses. The denominator $K_o$ is $N_P 4E_P I_P \beta_1^3$ in fixed support using the characteristic value, $\beta_1$ at the 1st soil layer. Figure 14 shows that the proposed method can estimate the exact solution within errors of 15%.

Next, impedance functions are examined under the condition of pile foundations with $D_P$ of 2m and single or $N_P$ of 2*2 or 4*4 in a soil deposit shown in Fig.9. Results on real and imaginary parts of

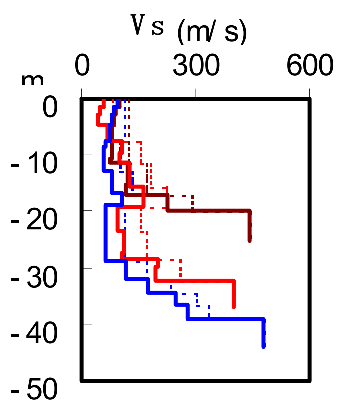



Fig.13 examples of soil

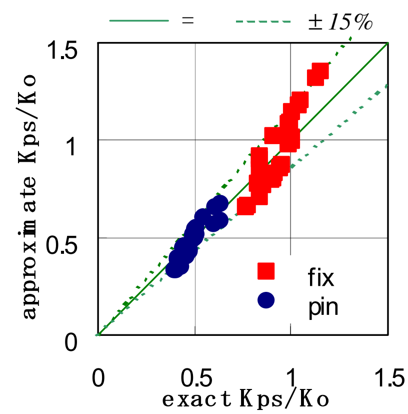
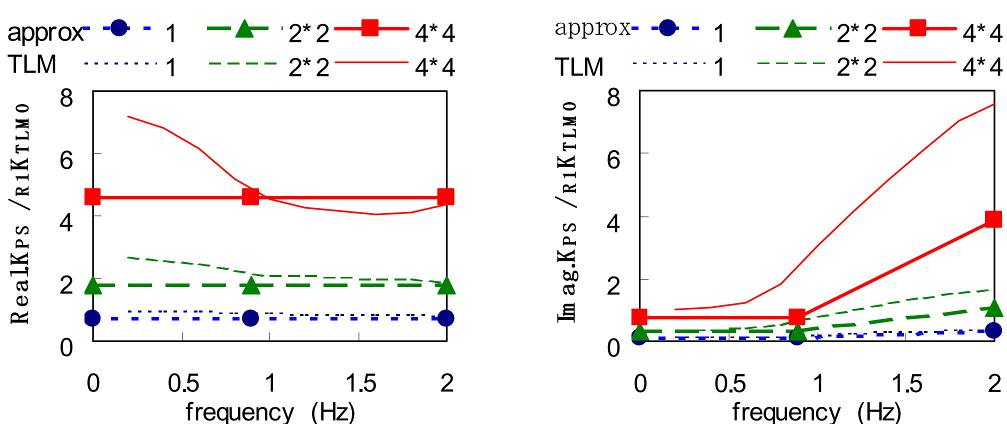



Fig.14 Sway spring constants of piles




a) Real part

b) Imaginary part

Fig.15 Examples of impedance functions of piles

$K_{ps} + iK'_{ps}$ are compared in Fig.15 with results by TLM in which pile spacing is 6m. Any impedance values are normalized by the real part of single pile impedance at 0.2Hz due to TLM. It may be recognized that the proposed method is practically applied although spring constants is a little lower at low frequency and damping coefficients a little lower at high frequency.

## FOUNDATION INPUT MOTIONS

As for the kinematic problem of SSI, the basic concept for foundation input motions of a massless embedded foundation, $U_{fh}$ shown in Fig.16 is derived from averaging free field motions, $U_g(z)$ with weighting by soil stiffness, that is, releasing the driving force due to the restriction by the existence of foundation:

$$U_{fh} = \frac{K_{bs} U_g(d) + K_{ws} \overline{U}_{gs}}{K_{bs} + K_{ws}}$$

where $\overline{U}_{gs} = \frac{1}{d}\int_0^d U_g(z)dz$. Therefore it is recognized from the relationship between $K_{bs}$ and $K_{ws}$ in Eq.(17a) that $\eta = d/\sqrt{A_f}$ is a key parameter [Tohdo et al. 1986]. Here, parametric studies for $U_{fh}$ are

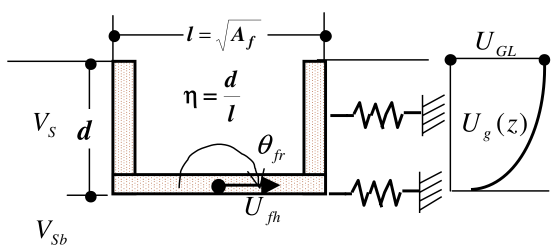



Fig.16 Schematic view for foundation input motion

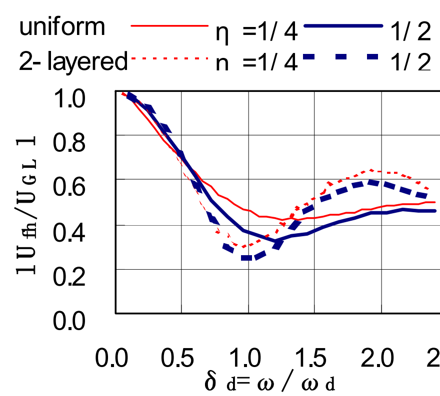
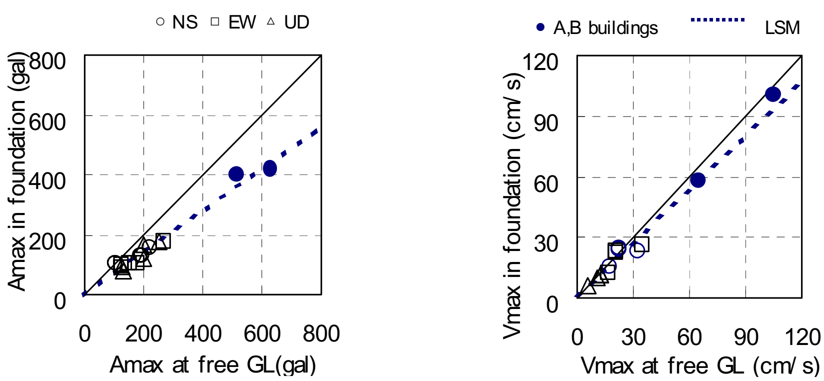



Fig.17 examples of foundation input motion




a) Peak acceleration

b) Peak velocity

Fig.18 The relationship between motions in building foundations and free field motions observed during the Kobe earthquake in 1995 carried out using the Thin Layer method (TLM), which have the $\eta$ of 1/4 or 1/2 and the supporting soil of $V_s = V_{sb}$ or $V_s = 0.5V_{sb}$ .The results of $\left|U_{fh} / U_{GL}\right|$ are shown in Fig.17 in which the circular frequency $\omega$ of the horizontal axis is normalized by the ground circular frequency of $\omega_d = \pi V_s / 2d$ . It is found from Fig.17 that $U_{fh}$ decreases as $\omega$ is higher and has the nodal frequency at $\omega_d$ to be $U_g(d) = 0$ .

The input motions of buildings subjected to Hyogoken Nanbu earthquake in 1995 had been investigated by a special project group of AIJ using observed earthquake motions [Yasui 1997]. Figure 18 shows the comparison of peak accelerations, $A_{max}$ and peak velocities, $V_{max}$ of observed motions between foundations and free surface ground in each site. The $\eta$ of A and B buildings are about 0.3 and 0.5, respectively. The average ratios of input motions in foundations against $U_{GL}$ from the least square method (LSM) had been obtained to be 0.7 for $A_{max}$ and 0.9 for $V_{max}$.

Based on these investigations, the AIJ's draft proposes the following formulae for the modification factor on input motions into embedded foundations.

$$\left|H_{emb}\right| = \left|\frac{U_{fh}}{U_{GL}}\right| = \left(1 + 2\eta\delta_d^2\right)^{-0.5} \quad for \quad \delta_d \leq 1$$

$$= \left(1 + 2\eta\right)^{-0.5} \quad for \quad \delta_d > 1 \qquad (30)$$

in which $\eta = d / \sqrt{A_f}$ , $\delta_d = \omega_1 / \omega_d$ and $\omega_d = \pi V_s / 2d$ where $V_s$ is the average shear wave velocity in soil surrounding side-wall.

## CONCLUSIONS

A practical method to evaluate seismic loads in the draft of AIJ recommendations is proposed, which considers herein soil-structure interaction (SSI) effects. The contents are summarized as follows.

One is to obtain story shear forces using an equivalent SDOF model for a super-structure connecting with sway-rocking spring and dashpot. In numerical examples, natural periods, modal dampings and effective masses by the proposed formula are estimated well the exact ones based on MDOF model.

Next, we propose the formula to evaluate spring and dashpot constants for flat, embedded and pile foundations, the later of which are based on impedance functions in terms of frequency. These formula include multi-layered soil effects, in addition pile-group effects. The validity to estimate is verified through numerical examples by the more rigorous method such as the 3D-Thin Layer method.

In the last, a semi-empirical formulae to evaluate foundation input motions into embedded foundations is recommended on the basis of analytical considerations and actual phenomena in observed earthquake motions during the 1995 Kobe earthquake.

We conclude this paper that the proposals are available for the evaluation of seismic loads taking into account SSI effects in a sense of practical seismic design.

## ACKNOWLEDGEMENT

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