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A STUDY OF DYNAMIC INTERACTION OF SOIL AND PILE WITH NON-UNIFORM SECTION

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SUMMARY

A number of response analyses of structures supported by pile foundations have been carried out using the 3-dimensional thin layer soil model, which is based on the Green's function method and is formulated in accordance with substructure technique. When we compose the complex flexibility matrix of soil, each pile considered is usually assumed as having an invariable constant cross section, respectively. In this study, the complex flexibility coefficients of soil derived using the ring exciting thin layer approach proposed by Tajimi is applied for piles with non-uniform cross sections, and their effectiveness is examined. As a numerical analysis, a simulation analysis of free vibration test of a new system foundation model that is composed of core-piles covered with an outer steel pipe and dampers is carried out. The test was carried out to confirm the possibility of the seismic base-isolation structures, which is constructed at soft ground sites. The numerical results agree with the test ones. Furthermore, it is verified that the new system foundation is appropriate for a base-isolation technique.

INTRODUCTION

The 3-dimensional thin layer soil model has been frequently utilized because of its sophistication and elegance for dynamic interaction analyses of structures embedded into layered soil mediums [Tajimi,1984].Especially, a considerable number of response analyses of structures supported on pile foundations have been carried out using the 3-dimensional thin layer soil model e.g. [Waas and Hartmann, 1984]. By applying this soil model to pile foundations, currently each section of pile considered is assumed as a constant diameter throughout. Actually, piles whose tip or head is provided with an expanded portion are employed to enhance their bearing or bending moment capacity. For the piles with the expanded tip or head portion, the complex flexibility matrix of soil becomes asymmetric, because the displacements at the center of piles due to a disk or ring load by the 3-dimensional thin layer approach are not satisfied with the reciprocity theorem. Therefore, when we estimate the flexibility matrix for the piles whose sections are non-uniform, the piles have been dealt as having equivalent invariable uniform sections.

In this study, the flexibility coefficients of soil derived using the ring exciting thin layer approach proposed by Tajimi [Tajimi, 1994] is presented for piles having non-uniform sections, and their applicability is examined. The ring exciting thin layer approach was developed to conduct the dynamic interaction analysis between soil and structure embedded in the layered soil medium [Hanazato *et al*. 1996]. By evaluating the displacement averaged on the circumference of each pile generated by the Green's functions, it is found that the flexibility matrix of soil for piles with non-uniform section is satisfied with Maxwell-Betti's reciprocity law. As a numerical analysis, simulation analyses for free vibration tests of a new system foundation model composed of core-piles covered with a steel pipe and dampers are carried out. The test was carried out to confirm the possibility of the seismic base-isolation structures, which is constructed at soft ground sites [Ishimaru *et al*.(1999)]. The numerical results agree with the test ones. It is confirmed that the new system foundation is favorable for a base-isolation construction technique. Furthermore, the proposed flexibility coefficient of soil for the piles whose sections are non-uniform is appropriate for evaluating more detailed analyses of structure-pile-soil interaction problems.

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FLEXIBILITY COEFFICIENT OF SOIL

Outline of Computational Method

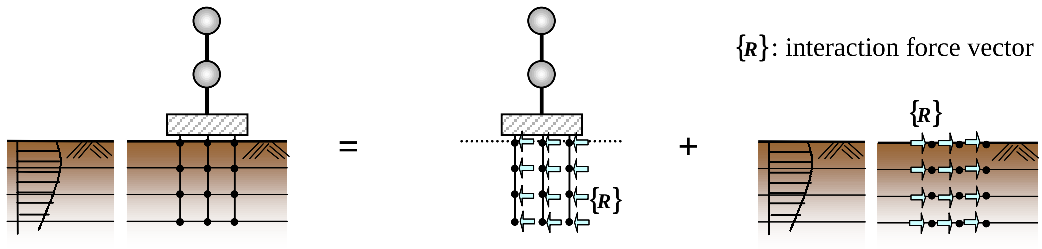
The computational method employed in this paper is based on the Green's function method and is formulated in accordance with the substructure technique. First, the total system (a) is divided into the two sub-systems, *i.e.* Structure-Pile system (b) and Ground system (c), as shown in Fig.1. By employing the flexible volume method for the pile columns and the soil with excavation, the steady state equation of motion for Structure-Pile system is finally formulated as

$$\left(\begin{bmatrix}[K_{SS}] & [K_{SP}] \\ [K_{PS}] & [K_{PP}]-[K_{PP}^{G}]+[A(i\omega)]^{-1}\end{bmatrix}-\omega^2\begin{bmatrix}[M_S] & \\ & [M_P]-[M_P^G]\end{bmatrix}\right)\begin{Bmatrix}\{u_S\} \\ \{u_P\}\end{Bmatrix}=\begin{bmatrix}\{0\} \\ \{F(i\omega)\}\end{bmatrix} \tag{1}$$

where $\{F(i\omega)\}$ represents the driving force vector, $[K]$ and $[M]$ represent static stiffness and mass matrices, respectively, subscript "*S*" and "*P*" denote Structure and Pile respectively, and superscript "*G*" denotes the excavated soil columns. The pile-soil-pile interaction can be appropriately included in the complex flexibility matrix of the soil system without excavated columns, $[A(i\omega)]$, by using the Green's function derived from the thin layer formulation. In the above and followings, the time function $e^{i\omega t}$ is omitted.

Flexibility Coefficient of Soil

The complex flexibility matrix of soil which can account for pile-soil-pile interaction is formed by the superposition of the complex flexibility coefficient between nodes of soil columns, i.e. harmonic point, disk and ring load solutions [Tajimi, 1980].



(a) Structure-Piles-Soil system **(b) Structure-Piles system** **(c) Soil system**

Figure 1: Two sub-systems separated by substructure method

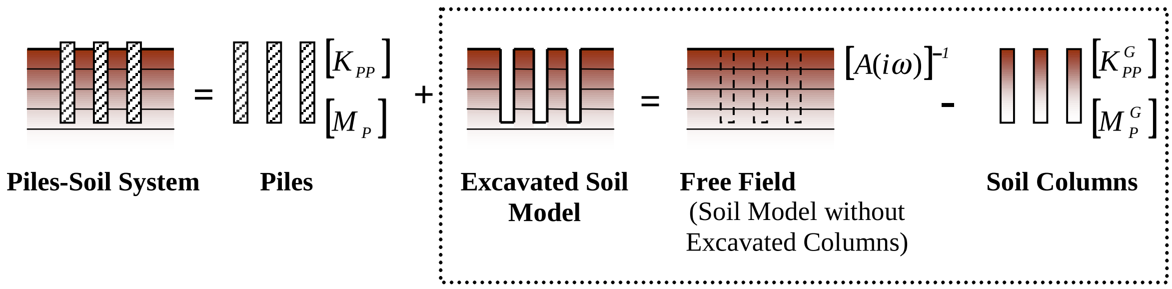


Figure 2: Flexible volume method for calculating flexibility coefficient of soil

Point load solution

The displacement solution of the *R-th* node of the *I-th* pile due to the steady state unit point load acting at the *S-th* node of the *J-th* pile, $1e^{i\omega t}$, is employed as the complex flexibility coefficient between these two nodes for the case of $I \neq J$ as shown in Fig.3. These displacement solutions can be expressed as followings [Tajimi, 1980],

$$\begin{Bmatrix} u_x \\ u_y \\ u_z \end{Bmatrix}_R = \begin{bmatrix} V_1 \cos 2\theta + V_2 & V_1 \sin 2\theta & V_4 \cos\theta \\ V_1 \sin 2\theta & -V_1 \cos 2\theta + V_2 & V_4 \sin\theta \\ V_3 \cos\theta & V_3 \sin\theta & V_5 \end{bmatrix} \begin{Bmatrix} P_x \\ P_y \\ P_z \end{Bmatrix}_S \tag{2}$$

where

$$V_1 = -\frac{1}{2\pi}\sum_{k=1}^{2N}\frac{X_{Rk}X_{Sk}}{D_{\alpha k}}\alpha_k^2 F_1(\alpha_k r) + \frac{1}{4\pi}\sum_{k=1}^{N}\frac{Y_{Rk}Y_{Sk}}{D_{\beta k}}F_1(\beta_k r)$$

$$V_2 = \frac{1}{2\pi}\sum_{k=1}^{2N}\frac{X_{Rk}X_{Sk}}{D_{\alpha k}}\alpha_k^2 F_2(\alpha_k r) + \frac{1}{4\pi}\sum_{k=1}^{N}\frac{Y_{Rk}Y_{Sk}}{D_{\beta k}}F_2(\beta_k r)$$

$$V_3 = -\frac{1}{\pi}\sum_{k=1}^{2N}\frac{Z_{Rk}X_{Sk}}{D_{\alpha k}}\alpha_k^2 F_3(\alpha_k r),\ V_4 = \frac{1}{\pi}\sum_{k=1}^{2N}\frac{X_{Rk}Z_{Sk}}{D_{\alpha k}}\alpha_k^2 F_3(\alpha_k r),\ V_5 = \frac{1}{\pi}\sum_{k=1}^{2N}\frac{Z_{Rk}Z_{Sk}}{D_{\alpha k}}\alpha_k^2 F_2(\alpha_k r)$$

$$F_1(z) = -\frac{2}{z^2} - i\frac{\pi}{2}H_1^{(2)}(z) + i\frac{\pi}{2}H_0^{(2)}(z),\ F_2(z) = -i\frac{\pi}{2}H_0^{(2)}(z),\ F_3(z) = -i\frac{\pi}{2}H_1^{(2)}(z)$$

in which X_{ik}, Z_{ik} =the *i-th* components of modal vectors of the order k for Rayleigh wave equation, α_k =the corresponding eigen value, $\mathrm{Im}(\alpha_k) < 0$, $D_{\alpha k}$ =the corresponding normalizing factor, Y_{ik} =the *i-th* component of nodal vector of the order *k* for Love wave equation, β_k = the corresponding eigen value, $\mathrm{Im}(\beta_k) < 0$, $D_{\beta k}$ = the corresponding normalizing factor, $H_\nu^{(2)}(*)$ =Hankel function of the 2nd kind of the ν–th order, and *N* denotes the total number of the discrete layers of the thin layer model.

Disk solution and ring solution

Flexibility coefficients of the soil for a self-excited pile generally evaluated by the center point displacement due to the disk or ring excitation of the uniformly distribution. The horizontal and vertical disk load solutions of the *I-th* pile a_{RS}^H, a_{RS}^V due to the corresponding unit load can be written as

$$a_{RS}^H = V_6\big|_{r=r_I},\ a_{RS}^V = V_7\big|_{r=r_I} \tag{3}$$

where

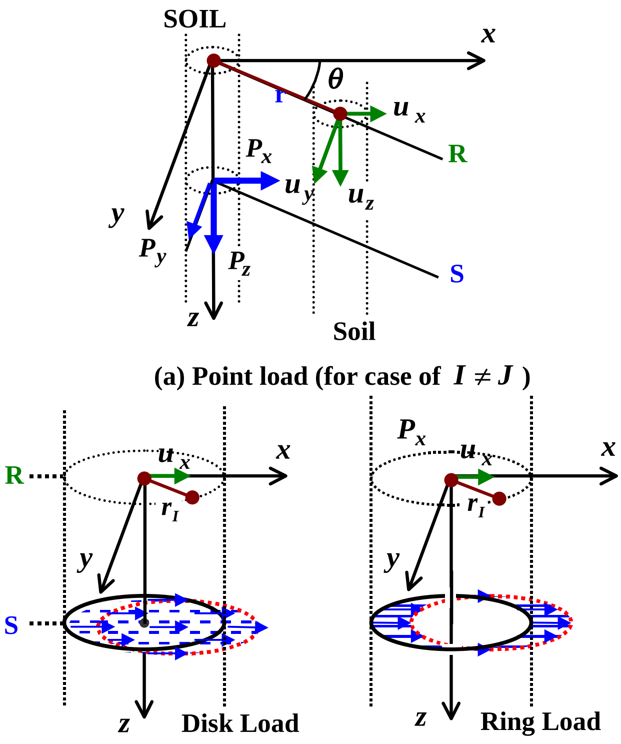
$$V_6 = \frac{1}{2\pi}\sum_{k=1}^{2N}\frac{X_{Rk}X_{Sk}}{D_{\alpha k}}\alpha_k^2 F_4(\alpha_k r) + \frac{1}{4\pi}\sum_{k=1}^{N}\frac{Y_{Rk}Y_{Sk}}{D_{\beta k}}F_4(\beta_k r)$$

$$V_7 = \frac{1}{\pi}\sum_{k=1}^{2N}\frac{Z_{Rk}Z_{Sk}}{D_{\alpha k}}\alpha_k^2 F_4(\alpha_k r)$$

$$F_4(z) = -\frac{2}{z^2} - i\frac{\pi}{z}H_1^{(2)}(z)$$

Similarly, the ring load solutions are given as following.

$$a_{RS}^H = V_2\big|_{r=r_I},\ a_{RS}^V = V_5\big|_{r=r_I} \tag{4}$$



(a) Point load (for case of $I \neq J$)

(b) Disk load and ring load (for case of $I = J$)

Figure 3: Relationship between nodal force and displacement of soil system

DISPLACEMENT FUNCTION OF SELF-EXCITED PILE WITH VARIABLE CROSS SECTION

New System of Core-pile Covered with Steel Pipe

Recently, a lot of studies on isolation systems and active or passive control systems have been carried out as the utilities of the isolation systems were verified through the experience of the Hyougoken Nanbu earthquake etc. The authors' group has been executing a study on the feasibility of a seismic base-isolation, which uses a new soil-pile-structure system [Ishimaru *et al*., 1999]. The new system is composed of core-piles covered with a steel pipe and dampers. The complex flexibility coefficients of soil for such a pile have to be computed in consideration of the variation of radius of the cross section along the longitudinal axis. The computing method is derived in the following section.

Flexibility Coefficients for Non-uniform Section Piles

According to the method of both disk and ring solutions presented in the previous chapter, the displacement of the center on the *R-th* nodal surface due to the unit excitation load on the *S-th* nodal surface that is independent on the radius of the *R-th* surface, r_R, can be written as follow,

$$a_{RS} = \sum_{k=1}^{2N} A_k F(\alpha_k r_S) + \sum_{k=1}^{N} B_k F(\beta_k r_S) \tag{5}$$

where r_S = the radius on the *S-th* surface, A_k and B_k are the normalizing coefficients for the *k-th* modes, F(*)=functions depending on the *k-th* mode and the radius of the cross section. When the radius on the *S-th* surface is different from that on the *R-th* one, $r_R \neq r_S$, the reciprocity law, $a_{SR} = a_{RS}$, is not satisfied in Eq. (5). Therefore, the complex flexibility matrix becomes asymmetry.

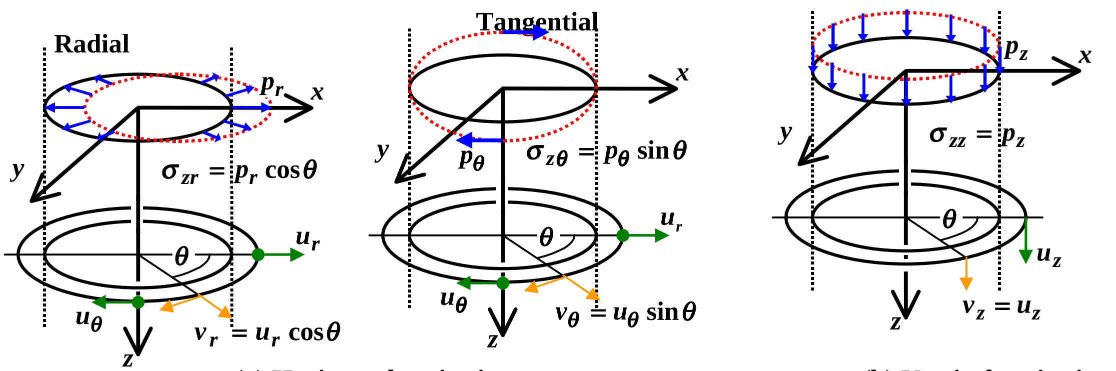
Tajimi proposed the Green's function for (r, θ, z) components in the concentric circle for the ring loads[Tajimi, 1994]. Omitting the vertical displacement due to the horizontal excitation and the horizontal displacements due to the vertical excitation, Fig.4 illustrates the distribution of ring loads and the corresponding displacements for sway and vertical modes, respectively. By using the Green's function for the ring loads and evaluating the complex flexibility coefficient as the average displacement on the circumference, the reciprocity law of the coefficients can be satisfied. Though the details of derivation are omitted, the horizontal and vertical complex flexibility coefficients can be finally written as follows.

$$a^H_{RS} = \frac{1}{2\pi}\sum_{k=1}^{2N}\frac{X_{Rk}X_{Sk}}{D_{\alpha k}}\alpha_k^2 G(\alpha_k, R_a, R_b) + \frac{1}{4\pi}\sum_{k=1}^{N}\frac{Y_{Rk}Y_{Sk}}{D_{\beta k}}G(\beta_k, R_a, R_b) \tag{6}$$

$$a^V_{RS} = \frac{1}{2\pi}\sum_{k=1}^{2N}\frac{Z_{Rk}Z_{Sk}}{D_{\alpha k}}\alpha_k^2 G(\alpha_k, R_a, R_b) \tag{7}$$

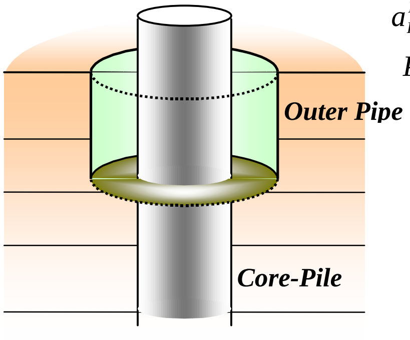
where $R_a = Min.(r_R, r_S)$, $R_b = Max.(r_R, r_S)$, $G(z, r, R) = -i\frac{\pi}{2}J_0(zr)H_0^{(2)}(zR)$

Therefore, Eqs. (6) and (7) yield $a^H_{RS} = a^H_{SR}$ and $a^V_{RS} = a^V_{SR}$ respectively, and these obviously satisfy the reciprocity law.



(a) Horizontal excitation **(b) Vertical excitation**

Figure 4: Distribution of ring loads and corresponding displacements



$$a_{RS}^{H} = \frac{1}{2\pi}\int (v_r \cos\theta - v_\theta \sin\theta)d\theta$$
$$P_S = \int (\sigma_{zr}\cos\theta - \sigma_{z\theta}\sin\theta)d\theta = 1e^{i\omega t}$$

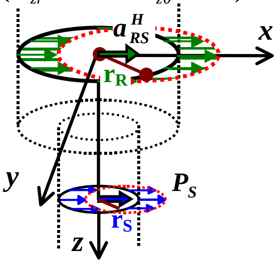


Table 1: Soil profile

20 m	*20@1m*	$V_s = 100 m/s$
15 m	*10@1.5m*	$\rho = 1.6\ t/m^3$
40 m	*20@2m*	
dashpot -mat		

Figure 5: Schematic illustration of core-pile covered with steel pipe and proposed method for calculating flexibility coefficients of soil for pile with non-uniform cross section

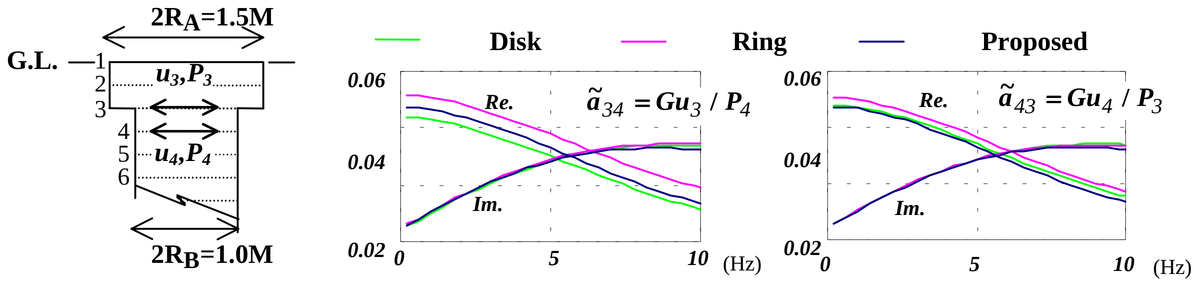


Figure 6: Examples of flexibility coefficients of soil for non-uniform cross section pile

Numerical Example of Flexibility Coefficients

Fig. 6 shows the numerical example of the three kinds of the complex flexibility coefficients for the self-excited pile, i.e. disk solution, ring solution and the proposed method. The numerical analysis is carried out for the pile whose top is expanded is buried into the homogeneous soil modeled by the thin layer model shown in Table1 1. The longitudinal axis of Fig. 6 is represented as $\tilde{a}_{jk}^{H} = Ga_{jk}^{H}$ ($G =$ shear rigidity of soil).

VIBRATION TEST AND SIMULATION ANALYSIS OF NEW BASE-ISOLATION SYSTEM

New System of Seismic Base-Isolation at Soft Ground Sites

In general, it is well known that structures and equipment, etc adopted isolation systems prevent not only human damage but also breakage of structures during earthquakes. However, the isolation structures can take advantages against disaster only when they are constructed in fine sites. Many cities in Japan are located at the inappropriate sites like as reclaimed lands or deep alluvium deposits. To construct the isolation structures at such a site, the site condition has to be improved by using the ground improvement works or pile supports. From the above reason, it is considered that a parallel movement of both the superstructure and the basement structure cannot be expected without soil improvement. Accordingly, the effect of the device such as the laminated rubber bearings decreases due to shortage of the stiffness of the base etc. However, doing construction of the base-isolated structures, which have the ability of the parallel movement, has mainly two problems. The first problem is the cost of this ground improvement work, and another one is difficulty of the design implying the uncertainty of the improvement method.

From the above viewpoint, the authors' group has dealt with a study on the feasibility of the seismic base-isolation system, which is constructed at the soft ground site [Ishimaru *et al.*, 1999]. Fig. 7 shows an illustration of the system concept. To verify the possibility of the seismic base-isolation structures constructed at such a site, the group made the experimental model, which is composed of double steel pipe piles, a partial ground improvement body and dampers. Furthermore, the double steel pipe pile is composed of a core pile and its outer pipe which covers with the upper part of the core pile, to enhance the deformability of the core pile head. The damper is installed between the lower side of the foundation and the upper part of the improvement body that is separated from piles. The horizontal stiffness becomes lower so that the pile may act as a long column, and it is possible to function as an isolator like the laminated rubber bearing. Because the phase and the frequency trends of the improvement body-soil system and the structure-soil system are different, the total system can possess an energy absorbing device by setting up dampers between these two systems. It is expected that the structural system can efficiently control the earthquake response.

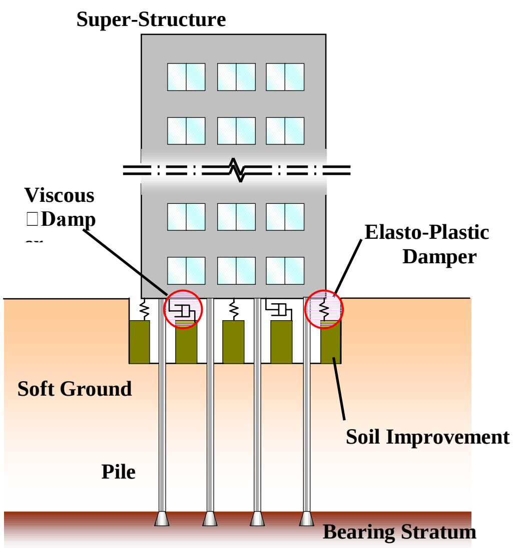
Experimental Models

The two types of experimental models (reinforced concrete block supported by pile foundation) illustrated in Fig. 8 were made for trial purposes, one was a conventional foundation model and another was a new system foundation model. Table 2 shows the physical properties of the test models. The concrete block of the conventional model was directly supported on core piles without outer pipe, and the soil adjacent the model was not improved. On the other hand, the upper region (G.L.-G.L.-2m) of the core pile on the new type model was covered with an outer steel pipe. Both models had the same reinforced concrete block. The blocks rested on four piles driven to the bearing stratum at the depth of G.L.-6.2m. Furthermore, these pile tops and blocks were united through reinforcing steel bars.

Simulation Analysis of Free Vibration Test

The verification of the proposed computational method is confirmed by the simulation analysis of free vibration tests of both the conventional and the new system foundation models. Owing to an approximate analysis, let assume that the influence of the ground improvement and foundation-soil-foundation interaction is neglected. It is considered that one of the joint points of a damper is located at the surface of the free field. The connection between the pile head and the concrete block is represented as a pin joint.

The numerical results of impedance functions for horizontal and rotational components of both the conventional type and the new type model (before dampers are set up) are shown in Fig. 9. The soil profile utilized in the calculations is indicated in Table 3. Because the site condition is unsuitable for the purpose of the actual experiment, the soil at the shallower part than 1.5m (backfilled macadam) was backfilled again after being excavated once. Therefore, S-wave velocity of presumed from the investigation beforehand is considerably decreased referring to the result of a preliminary calculation. The horizontal rigidity of the new type model decreases dramatically when it is compared with that of the conventional type model. This is originated to the transformation performance of the piles of the new type model. On the other hand, a remarkable difference is not seen in the rotational rigidity of the both models, because of the fact that the axis force, which acts on the pile head, reaches to the pile tip.



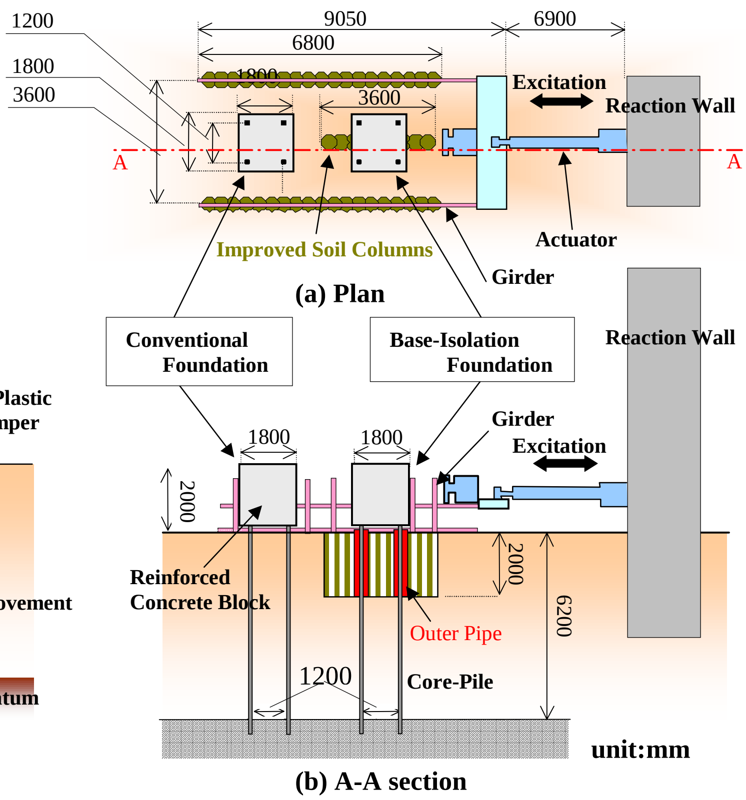


Figure 7: Concept illustration of the proposed base-isolation system

Figure 8: Test models (reinforced concrete block supported by pile foundation)

The measured and analytical results of free vibration of both models are shown in Figs .10 and 11, respectively. The numerical analysis is carried out by using the impedance functions shown in Fig. 9. These figures show the dimensionless horizontal displacement of the bottom of the concrete block due to the corresponding initial displacement. The result using the frequency dependent dynamic stiffness, which is evaluated only from the impedance function, estimates attenuation values lower than the test ones. As this might be caused by nonlinear behaviors of the soil adjacent the model and the condition of the pin connection of the pile head, etc., these effects are approximately represented as a viscous damping. As a result, it assumed that the minimum value of the viscous damping excluding the radiation damping is about 0.05, as shown in Fig. 11. Further Fig. 12 shows the test results of the free vibration of the new type model, which installed the lead dampers or friction ones. Similarly, Fig. 13 shows the analytical results of the model. Here, the influence of the dampers and the pin joint connections is approximated as a viscous damping. Finally, the damping ratios for the cases of lead dampers and friction ones can be approximately estimated as $h \approx 0.10 - 0.15$ and $h \approx 0.20 - 0.25$, respectively.

CONCLUSION

The calculation method of complex flexibility coefficients based on the thin layer model by which the influence of the non-uniformed cross section of the pile considered is presented in this paper. The proposed flexibility coefficients satisfy the reciprocity law. The simulation analysis of the free vibration test of the seismic base-isolation foundation model and the conventional model is demonstrated as an applied example. As a result, the analytical result that approximates the influence of the attenuation as a viscous damping agrees pretty well with the measurement one. Moreover, it is confirmed that the proposed new type base-isolation structure is excellent in the deformability and attenuation and is appropriate as a method of making of the base-isolation for the soft ground sites. A more detailed analysis that considers the influence of the ground improvement body, the adjacent foundation and etc. is necessary to achieve the proposed type base-isolation in future.

ACKNOWLEDGEMENT

The experiment on the foundation used in this paper is the one executed by the study committee of Seismic Isolation by Use of a Soil-Pile-Structure System (Chief: Prof. Ishimaru, Members: Coll. of Sci. & Tech., Nihon Univ., Kume Sekkei Co. Ltd., Oiles Co., Kawada Industries Inc., Tokyo Soil Research Co. Ltd., Asahi Chemical Construction Materials Co. Ltd., Tenox Co., Tomoe Co., Mitsubishi Materials Co., Soil Design Inc. and Nippon Stud Welding Co. Ltd.). The authors wish to express their appreciation to all members of this committee'.

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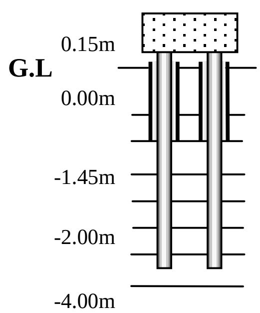
Table 2: Physical properties of the models

dimension :	DxDxH=1.8mx1.8mx2.0m	
weight W :	0.152MN	
Piles	Core-pile	Outer pipe
diameter :	0.1143m	0.2163m
wall thickness :	6mm	4.5mm
flexural rigidity :	0.618MNm^2	3.457MNm^2
length :	6.2m	2m

Table 3: Soil profile for numerical analysis



ρ (ton/m^3)	V_S (m/s)	ν	h
2.0	30 (125)	0.179	0.02
1.6	73	0.486	0.02
1.6	73	0.486	0.02
1.8	110	0.495	0.02
1.8	150	0.491	0.02
2.0	270	0.492	0.02
2.0	270	0.429	0.02
2.0	360	0.487	0.02

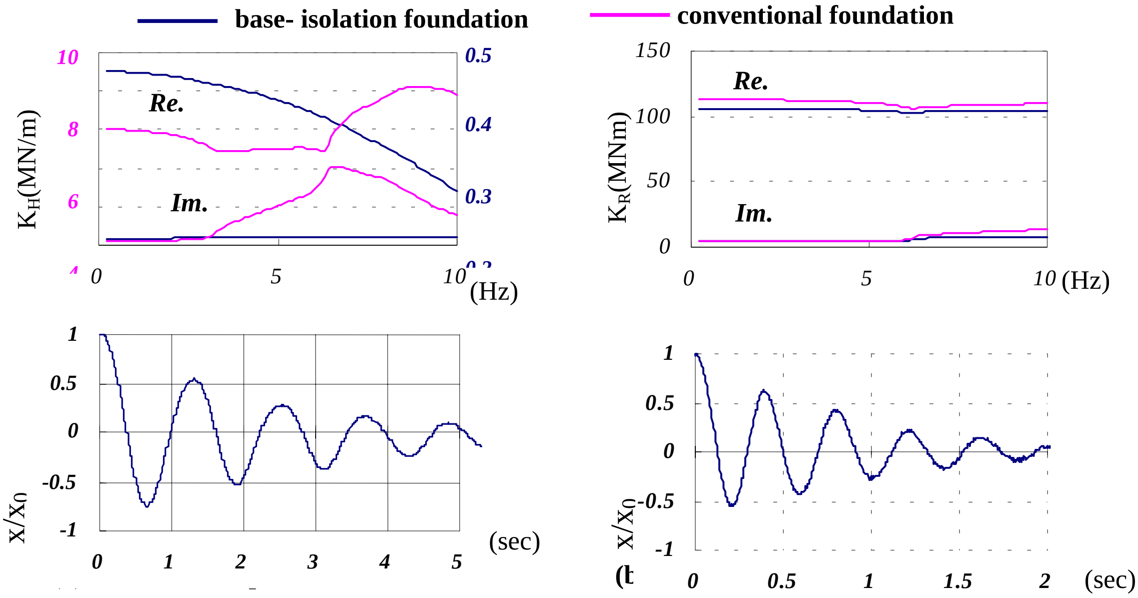
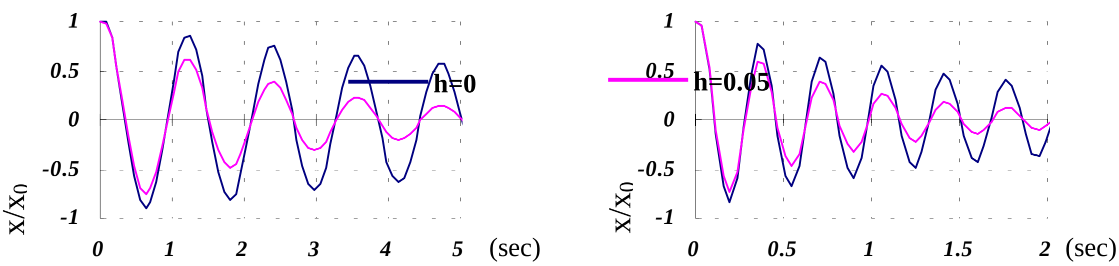
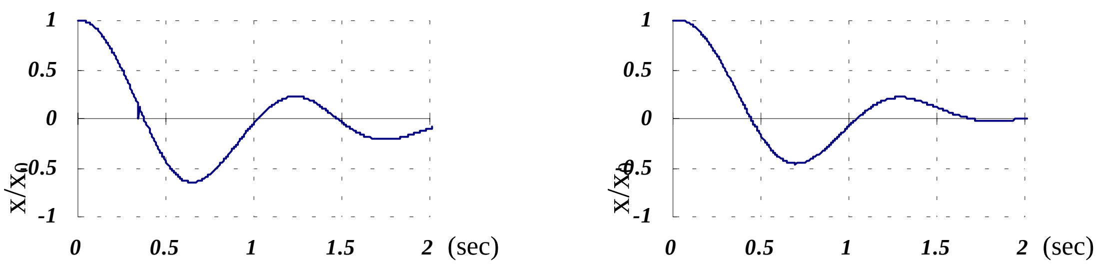


Figure 9: Impedance functions for the two models



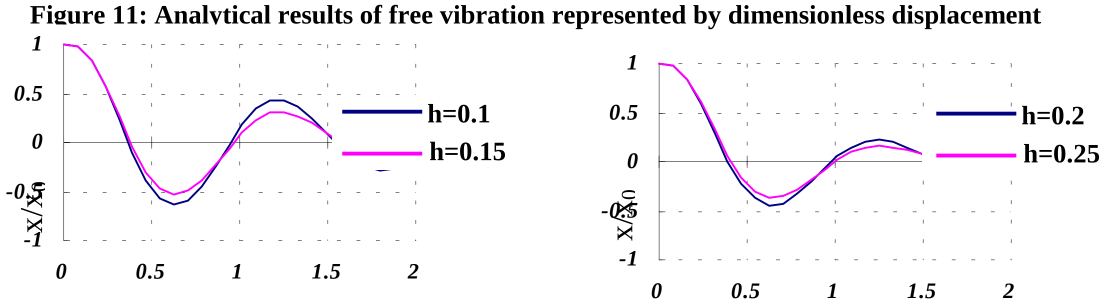
(a) Base-isolation type model (without damper) **(b) Conventional type model**

Figure 10: Test results of free vibration represented by dimensionless displacement



(a) Base-isolation type model (without damper) **(b) Conventional type model**

Figure 11: Analytical results of free vibration represented by dimensionless displacement



(a) Lead damper (sec) **(b) Friction damper** (sec)

Figure 12: Test results of free vibration of the base-isolation model with dampers

(a) Lead damper(b) Friction damper

Figure 13: Analytical results of free vibration of the base-isolation model with dampers