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SIMPLIFIED NUMERICAL METHOD FOR EVALUATION OF NON-LINEAR DYNAMIC PILE-SOIL-PILE INTERACTION

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SUMMARY

It is well known that the earthquake response of a pile-supported structure is strongly affected by the dynamic soil-pile interaction. When the structure is subjected to a severe ground motion, the soil behaves in nonlinear manner. Two kinds of the soil nonlinearity will affect the characteristics of the dynamic soil-pile interaction. The one is the nonlinearity that appears in the soil surrounding the pile and the other is the nonlinearity that appears in the free field. The nonlinearity of soil surrounding the pile is caused by the inertia force which is transmitted from the superstructure on the pile head (*i.e.*, the local-nonlinearity). The soil nonlinearity in the free field is caused by the transmitting shear wave (*i.e.*, the site-nonlinearity). Various tools based on the Sway-Rocking model have been developed to analyze the nonlinear dynamic soil-pile interaction. Most of these methods, however, take into account only the local-nonlinearity effects. Therefore, a methodology to evaluate both the effects of the local-nonlinearity and the site-nonlinearity are developed in the paper. The accuracy of this method is confirmed by comparing the simulated results and the experimental results.

INTRODUCTION

When a structure is subjected to a ground motion, it interacts with the soil and the foundation. It is well known that the dynamic soil-pile interaction should be taken into account in analyzing the response of a pile-supported structure such as a railway viaduct constructed in a soft ground. Thus, various studies have been conducted to deal with elastic dynamic soil-pile interaction. However, a reasonable scheme is desired recently to evaluate the nonlinear dynamic soil-pile interaction caused by a severe ground motion. The Hyogoken Nanbu Earthquake in 1997 caused serious damage to a number of pile-supported structures in Kobe and proved the importance of accounting for the nonlinear dynamic soil-pile interaction.

Various tools have been developed to analyze the nonlinear soil-pile interaction. These tools can be categorized into two groups. One group, treating the soil medium as a continuum, includes the "Finite Element Method (FEM)"[2]. The finite element method is a direct approach to analyze the soil-pile interaction. The direct method, however, requires a computer program that can treat the behavior of both piles and the surrounding soils with equal rigor. As a result, it requires quite a long time to conduct the

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analysis, especially in a 3-dimensional analysis. Hence, there still remains an important place for simple approaches even in these days. The another category includes rather simplified approaches such as the "Beam-on-Winkler-Springs" (so-called the "p-y method"). Winkler spring assumes that the soil reaction at a point of a pile depends only on the deformation of that particular point and not on the pile deflection above or below it. Therefore, the soil can be replaced with a set of discrete springs with a load-deflection characteristic curve (p-y curve). The nonlinearity of p-y curves is sometimes specified by empirical approach[3] or specified by a push-over analysis by using a 3-D FEM computer program[4]. These methods are not enough, however, to evaluate the effects of nonlinearity of soil profiles, because these approaches can consider only the effects of nonlinearity appeared in the soil surrounding the pile, which is caused by the inertia force (so-called the local-nonlinearity). It should be noted that as well as the local-nonlinearity, the effects of the nonlinearity which is caused by the transmitting shear wave in a free-field should be taken into account (so-called the site-nonlinearity).
In this paper, a simple and valid approach is developed to evaluate the local-nonlinearity and site-nonlinearity as well as the pile-group effects. The accuracy of this method is confirmed by comparing the simulated results and the experimental results.

EQUIVALENT SINGLE UPRIGHT BEAM

The greatest concern in this study is the response of a railway and expressway viaduct supported by a pile-group foundation. The viaduct is modeled by the three degree of freedom system shown in Fig. 1. In this model the soil-pile interaction system is simply expressed by the sway and rocking springs, so-called the Sway-Rocking model.

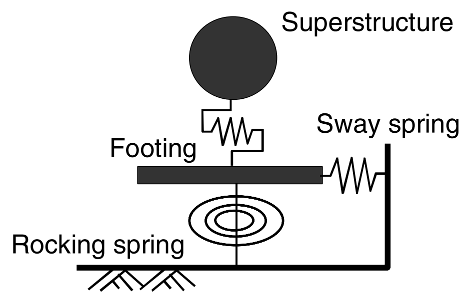


Fig. 1 Simplified analytical model for dynamic soil-pile interaction system
(so-called the sway-rocking model)

Formulation of equivalent single upright beam

The dynamic impedance of a groped pile is evaluated by an equivalent single upright beam method, which is first proposed by Konagai *et al*[5]. A brief description about the equivalent single upright beam analogy is given in this paper. See Ref. [5] for its details. The present single upright beam is a composite of n_p piles and the soil caught among them. Following the assumption of Thin-Layered Element Method[1] ("TLEM"), the soil deposit overlying its rigid bedrock should include a cylindrical hollow of radius R_0. The single upright beam is assumed to be embedded in this hollow. The cross-section πR_0^2 of this hollow is assumed to be identical to the cross-section of the beam A_G enclosed with the broken line circumscribing the outermost piles in the group (Fig. 2). The motion of the hollow is assumed to be compatible with that of the beam. The soil-pile composite together with its exterior soil is divided into n_L horizontal slices as shown in Fig. 2. The following assumptions are adopted to derive the stiffness matrix of the equivalent single beam.

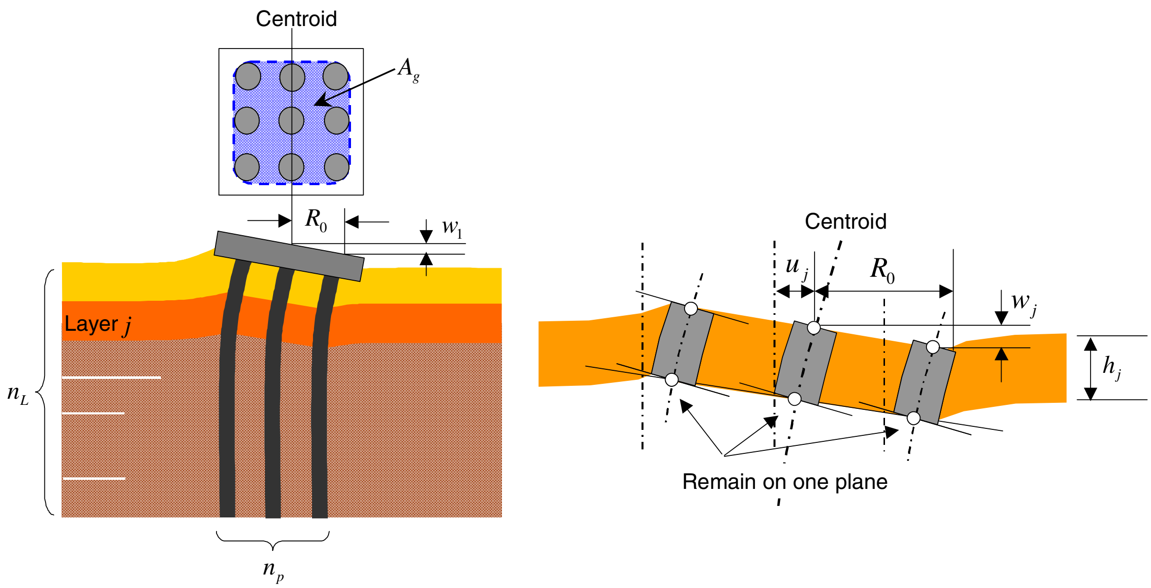


Fig.2 Assumption for evaluation of equivalent single beam

(a) Pile elements within a horizontal soil slice are all deformed at once keeping their intervals constant, and the soil caught among the piles moves with the piles.
(b) Frictional effects due to bending of piles are ignored.
(c) The top ends of piles are fixed to a rigid cap.
(d) All upper or lower ends of the sliced pile elements arranged on the cut-end of a soil slice remain on one plane.
With assumptions (a) and (d), there are only two degrees of freedom for each cut-end of all sliced of the soil-pile composite, namely sway and rocking motions, which are respectively designated as

$$\{\mathbf{u}\}=\begin{Bmatrix} u_1 \\ u_2 \\ \vdots \\ u_{n_L} \end{Bmatrix},\ \{\mathbf{w}\}=\begin{Bmatrix} w_1 \\ w_2 \\ \vdots \\ w_{n_L} \end{Bmatrix} \tag{1}$$

where n_L is the number of sliced soil-pile layers in the vertical direction. The rocking motions are expressed in terms of the anti-symmetric vertical motion $\{\mathbf{w}\}$ at the outermost edge ($r = R_0$) of the equivalent beam with respect to the centroid of the beam. In sway motions, all n_p piles are equally displaced (assumption (1)). This assumption makes the bending stiffness EI of the equivalent beam simply n_p times the bending stiffness of an individual pile. Assumptions (3) and (4) imply that axial motions of the piles control the overall anti-symmetric rocking motion of the equivalent beam just as the reinforcements in a concrete beam do. Therefore, another bending stiffness parameter EI^G is introduced to describe the rocking motion of the beam. This parameter EI^G is evaluated by following the same procedure as that used for the evaluation of bending stiffness of a reinforced concrete beam. The global stiffness matrix of equivalent single beam is finally described as

$$\begin{bmatrix} \{\mathbf{p}_x\} \\ \{\mathbf{M}/R_0\} \end{bmatrix} = [\mathbf{F}_H]\cdot\begin{bmatrix} \{\mathbf{u}\} \\ \{\mathbf{w}\} \end{bmatrix} \tag{2}$$

where $\{\mathbf{p}_x\}$ is the lateral external forces and $\{\mathbf{M}\}$ is the momens. The global equations of motion for the upright single beam are thus obtained as

$$\left([\mathbf{F}_H]-\omega^2[\mathbf{M}_H]\right)\begin{bmatrix}\{\mathbf{u}_x\}\\ \{\mathbf{w}\}\end{bmatrix}=\begin{bmatrix}\{\mathbf{P}_x\}\\ \{\mathbf{M}/R_0\}\end{bmatrix} \tag{3}$$

Motions of the surrounding soil

The Thin-Layered Element Method, a semi-analytical finite element method developed by Tajimi and Shimomura[1] is performed to describe the motion of the surrounding soil.

In the analysis, a foundation with a circular cross-section is assumed to be embedded upright in the cylindrical soil hollow. The wall of the cylindrical hollow and the foundation (single upright beam) are assumed to be completely connected to each other. Thus, the force-displacement relationship is to be obtained on the wall of the cylindrical hollow. The following layer boundary force-displacement relation is finally obtained:

$$\begin{bmatrix}\{\mathbf{P}_X\}\\ \left\{\dfrac{\mathbf{M}_y}{R_0}\right\}\end{bmatrix}=[\mathbf{R}_H]\cdot\begin{bmatrix}\{\mathbf{V}_r\}\\ \{\mathbf{V}_z\}\end{bmatrix} \tag{4}$$

in which $\{\mathbf{V}_r\}$ and $\{\mathbf{V}_z\}$ are the displacement vectors for radial and vertical components, respectively.

Motions of the entire soil-foundation system

When the following boundary conditions on the upright cylindrical hollow is taken into account,

$$\{\mathbf{V}_r\}=\{\mathbf{u}\},\ \{\mathbf{V}_z\}=\{\mathbf{w}\} \tag{5}$$

the equation of motion for the entire soil-foundation system are obtained by combining the eq. (3) with eq. (4) as

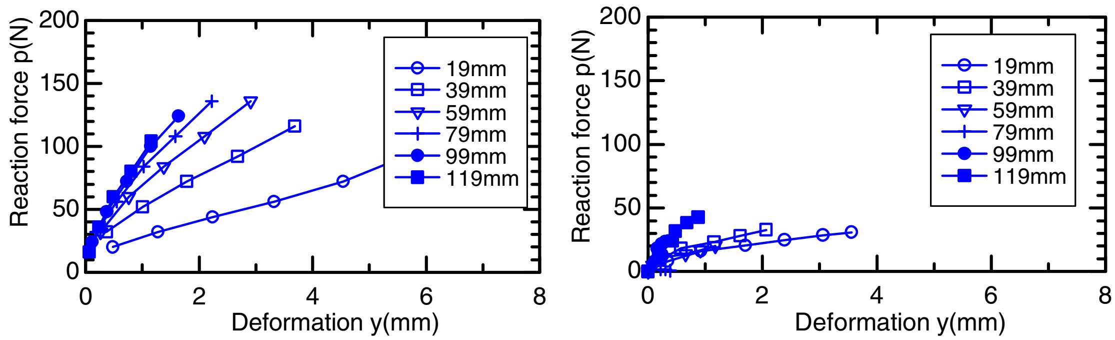
$$\left([\mathbf{R}_H]+[\mathbf{F}_H]-\omega^2[\mathbf{M}_H]\right)\begin{bmatrix}\{\mathbf{V}_x\}\\ \{\mathbf{V}_z\}\end{bmatrix}=\begin{bmatrix}\{\mathbf{P}_x\}\\ \{\mathbf{M}/R_0\}\end{bmatrix} \tag{6}$$

NON-LINEARLITY OF SOIL PROPERTIES

It is well kwon that the dynamic interaction between the soil and the pile will significantly be affected by the soil nonlinearity when a pile-supported structure is subjected to a severe ground motion such as the Hyogoken-Nanbu earthquake. Thus, it should be a very important issue how to evaluate both the effects of the local-nonlinearity (LN) and the site-nonlinearity (SN) in discussing the dynamic soil-pile interaction.

The authors have conducted a soil-deformation loading test and a lateral pile head loading test of a miniature pile model embedded in a sand box by using a centrifuge test system. In the lateral pile head loading test, the lateral force loaded statically on the pile head deforms the piles embedded in the soil. On the other hand, in the soil-deformation loading test, the side-walls of the lamina shear box are moved statically to provide soil deformation on the piles. See Ref. [6] for the details of these tests.

The relationships between the soil reaction force *p* and the relative pile deflection to the ground were measured. Fig. 3 shows the typical *p-y* curves obtained from these tests. The *p-y* curves obtained by the soil-deformation loading tests (including both the local- and site-nonlinearity) are much smaller than those by the lateral pile head loading tests (including only the local-nonlinearity). These figures suggest that the site-nonlinearity makes the *p-y* curves more smaller.



(a) Pile head loading test (b) Soil deformation test

Fig.3 *p-y* curves at each depth along the pile

In this study, the nonlinear models for sway-rocking springs at the pile head are developed based on the above results, which can be used in a non-linear earthquake response analysis of pile-supported structures. The process is summarized schematically in Fig. 4.

(a) The local nonlinearity of the soil surrounding the pile (*i.e*. the local nonlinearity) is incorporated into the sway-rocking spring by using a hyperbolic equation model as a backbone curve.

$$P = \frac{K_0}{1/y + 1/y_r} \tag{7}$$

where P is the pile head force; y is the displacement of pile cap relative to the soil surface; K_0 is the initial pile stiffness; and y_r is the reference displacement defined as

$$y_r = \frac{P_{\max}}{K_0} \tag{8}$$

where $P_{\max}$ is the ultimate resistance force at the pile cap. The Massing's 2nd rule is adopted for the histerisys loop. The initial stiffness K_0 is determined by the upright equivalent single beam method (see Chapter 2).

(d) The site-nonlinearity is incorporated into the sway-rocking spring by assuming the initial stiffness K_0 as the time dependent value. A site response analysis (*i.e*., nonlinear dynamic response analysis of free field) is conducted first to yield stress-strain histories of all layers. At each reversal point of the stress-strain history, a corresponding secant shear modulus was determined. The time dependent stiffness function $K_0(t)$ can be determined by using TLEM with the secant shear modulus.

(e) Since the ultimate resistant force $P_{\max}$ is strongly controlled by soil parameters such as the shear resistance angle ϕ or the cohesion c, $P_{\max}$ is assumed to be constant for the duration of the excitation.

(f) Finally, the proposed model yields the following equation.

$$P(t_i) = \frac{K_0(t_i)}{1/y(t_i) + 1/y_r(t_i)} \tag{9}$$

where $P(t_i)$ is the pile cap force at a particular time instance $t = t_i$; $y(t_i)$ is the relative deformation at the pile cap to the ground surface; and $K_0(t_i)$ is the pile cap stiffness at a particular time instance $t = t_i$. $y_r(t_i)$ is the reference displacement defined as

$$y_r(t_i) = \frac{P_{\max}}{K_0(t_i)} \tag{10}$$

Note that the ultimate resistance force at the pile cap $P_{\max}$ is a constant parameter, which is independent of shear stain.

Since the backbone curve of *p*-*y* relationship is changed step by step, a high frequency noise may occur in integrating the equation of motions. The high frequency noises often cause a serious problem in the computation of dynamic system, because they can trigger instability in the computation process. In such analyses, elimination of high frequency noise is essential. In order to eliminate the high frequency noise, a digital filtering should be introduced into the time history of shear modulus. The shear modulus time history smoothed by the digital filtering technique makes the integration stable.

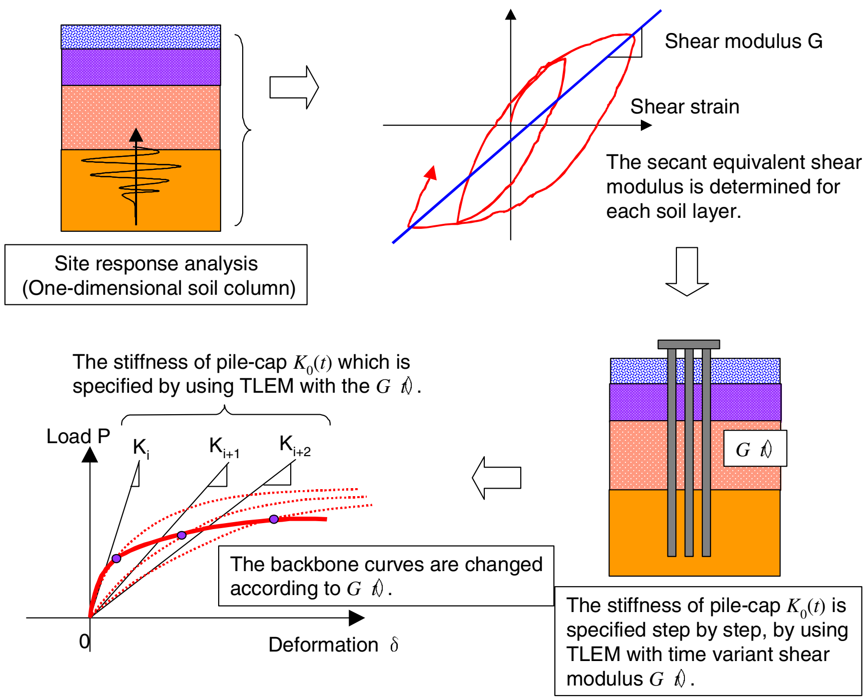


Fig.4 Schematic figure to specify the stiffness at the pile-head considering both effects of local-nonlinearity and site-nonlinearity

NUMERICAL ANALYSIS AND DISCUSSIONS

Large scale soil-pile shaking table tests have been carried out by the Railway Technology Research Institute (Tokyo, Japan)[7] and the results are used to verify the effectiveness of the newly developed analogy.

Outline of large-scale soil-pile shaking table test

Fig. 5 shows a cross-sectional view of the experimental device. Steel pipe piles and dry sand were used in the experiment. The configuration of pile group is 2×2. Piles were pinned at the bottom to the base of shear box. The pile head is fixed by steel footing. Two types of footing weight were used, *i.e.*, W=150 (kN) and 450 (kN). The pile-to-pile spacing for all groups was about 6D (D: diameter of the pile). The ground was constructed artificially with the Kasumigaura Sand, which was dropped from a certain height by a hopper. The white noises with different peak acceleration were used as the base excitations shown in Fig. 6. The detailed information can be found in Ref. [7]. Before the shaking table tests, the natural frequency of the soil-pile model was measured to be about 12.5 [Hz].

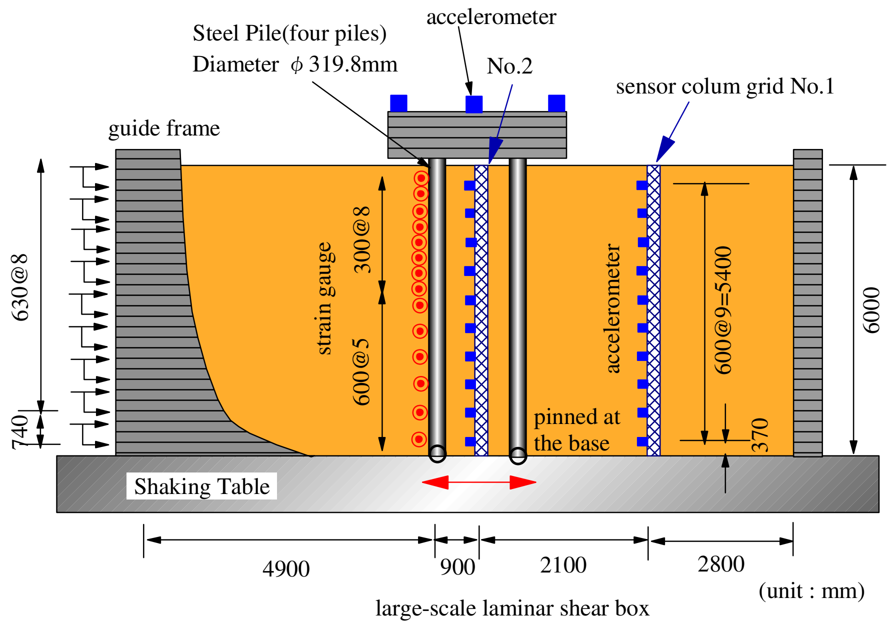
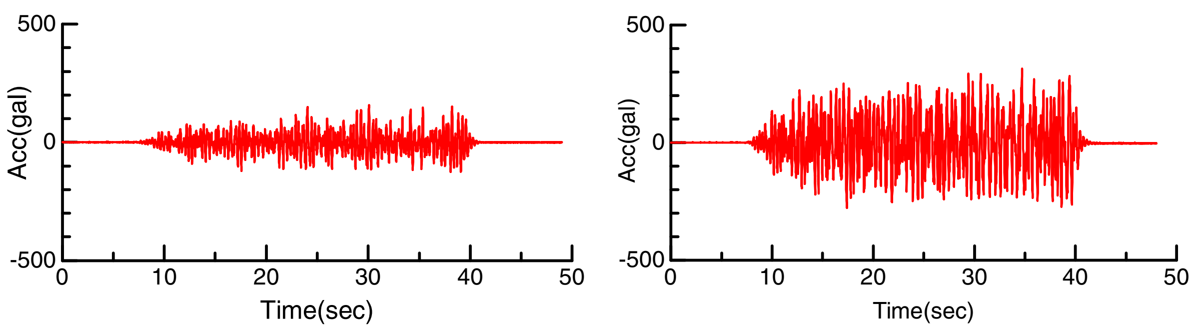


Fig. 5 Outline of experimental device



(a) 100(gal) type (b) 400 (gal) type

Fig. 6 Samples of base input acceleration

Stiffness of pile cap

The initial stiffness of pile cap $K_0(t=0)$, which is provided by using the equivalent upright single beam analogy mentioned in Section 3, is shown in Fig. 7. There is a slight space of 300mm between the footing and the ground surface. Thus, in order to take into account the effects of the space in the TLEM, a dummy thin layer whose stiffness is extremely small is added to the top of the original soil layer.

The shear modulus of soil is calculated by taking into account both the dependency of consolidation pressure and the effect of anisotropy characteristic. The Yong's modulus of soil for in the vertical direction cab be expressed by[8]

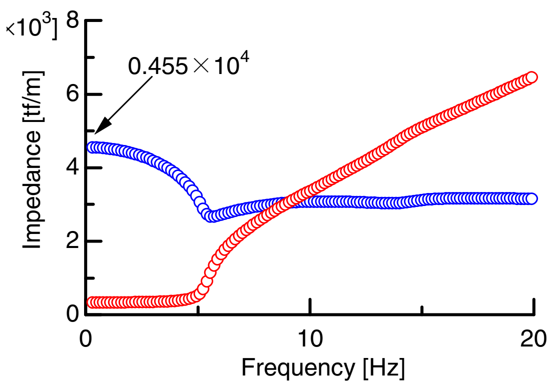


Fig. 7 Variation of stiffness parameter for sway motion of pile group

$$\frac{E_{\max v}}{p_r} = A \cdot f(e) \cdot \left(\frac{\sigma_v'}{p_r} \right)^m \tag{11}$$

where p_r is a reference stress; A and m are parameters decided by laboratory tests, *i.e.*, $A = 2.08 \times 10^4$ and $m = 0.5$; and $f(e)$ is the void ratio factor, which is provided by Eq. (12) [9].

$$f(e) = \frac{(2.17 - e)^2}{1 + e} \tag{12}$$

in which e is the void ratio. Tatsuoka and Kohata[8] pointed out that the Young's modulus of ground which is made artificially should have an anisotropy property. Thus, the young's modulus in the lateral direction $E_{\max,h}$ is given by reducing the young's modulus in the vertical direction $E_{\max,v}$ [8].

$$E_{\max,h} = \frac{1}{2} E_{\max,v} \tag{13}$$

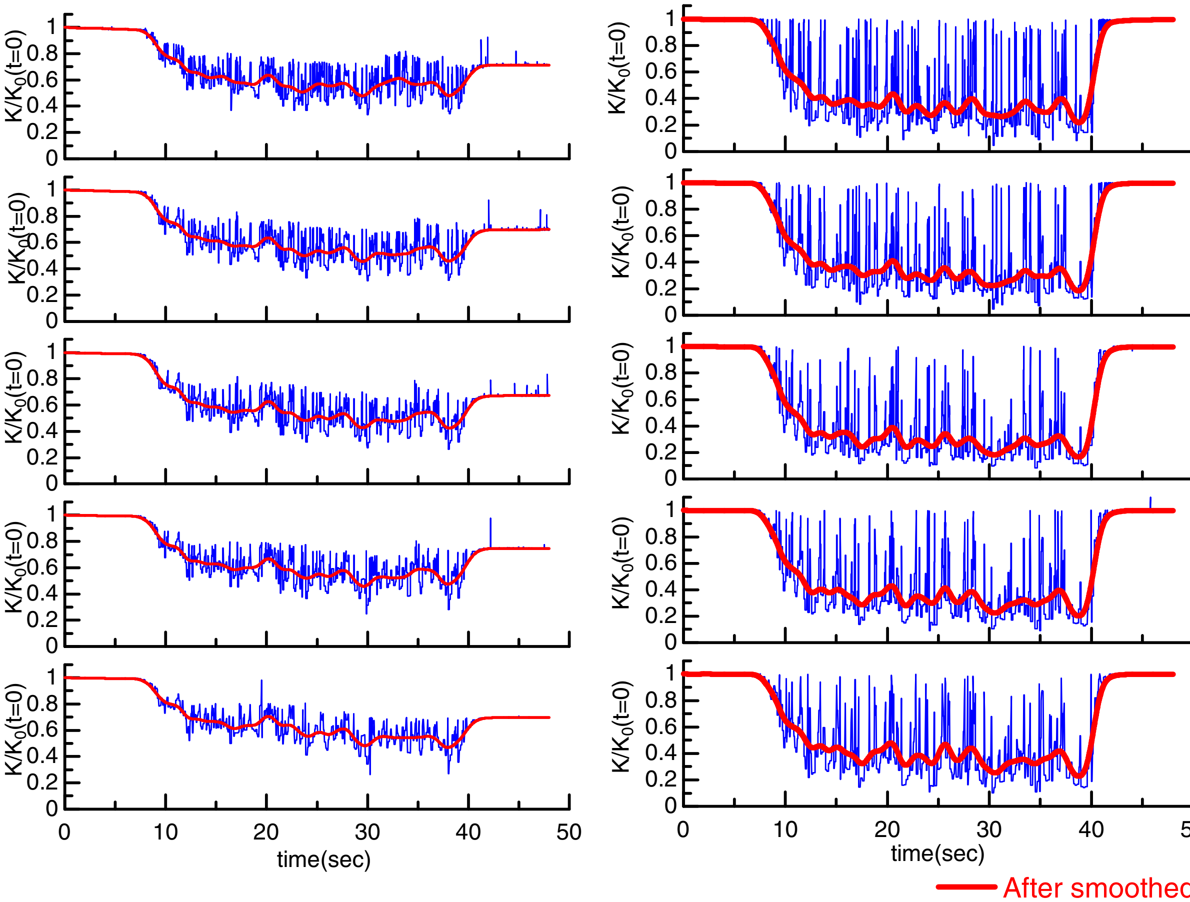
Because the ground is made artificially after piles were struck, the surrounding soil near the piles cannot completely be compacted. Thus, the young's modulus should be reduced more by the reducing factor α as[6], [10]

$$E_h = \alpha \times E_{\max h} \tag{14}$$

From the recent experimental study, the reducing coefficient will be 1/3-1/5. In this paper, the value of the parameter is determined as 1/3.

Simulation of free filed ground motion

The one-dimensinal non-linear dynamic response analysis is conducted to obtain ground responses. To express the nonlinear stress-strain relationship of each layer of the soil, the modified GHE model[11] is adopted. The modified GHE model was proposed by the authors, which satisfies both deformation and peak strength characteristics of soil. At each reversal point of the stress-strain history, the corresponding secant shear modulus is determined for each soil layer. Fig. 8 shows the variations of the secant shear modulus for different layers with time. The shear modulus is normalized by its initial stiffness. The time histories of the secant shear modulus contain quite high frequency components, which often cause serious problems in the computation of dynamic system, because they can trigger instability in the computation process. In order to avoid this trouble digital filtering is introduced into the time history of shear modulus. The smoothed time histories of shear modulus are also shown in Fig. 8.



(a) 100 gal input (b) 400 gal input

Fig. 8 Variation of secant shear modulus for each soil layer

Simulation results

The rocking motion was negligible because the bottoms of piles are pinned at the base of sand box. The effective input motion at the top end of the foundation is evaluated by multiplying the kinematic displacement factor by the free-field surface response. The kinmatic displacement factor is specified for the equivalent upright single beam (see Chapter 2-3).

The variation of the acceleration and the displacement at the footing are shown in Fig. 9 and Fig. 10. In this paper, the following three types of analysis are conducted,

(a) Analysis to take into account only the local-nonlinearity effects (*i.e.*, the response analysis with the normal hyperbolic p-y curve, LN-model).

(b) Analysis to take into account only the site-nonlinearity effects (*i.e.*, the response analysis with the linear p-y curve whose stiffness is varied step by step with the secant shear modulus of free-field, SN-model).

(c) Analysis to take into account both the local- and site-nonlinearity by using the newly developed method mentioned in Chapter 3.

Case for W=150 [kN]

The simulated response acceleration and displacement time historyies agree quite well as the measured response results. It is proved that the newly developed method has a high accuracy. The analysis with the

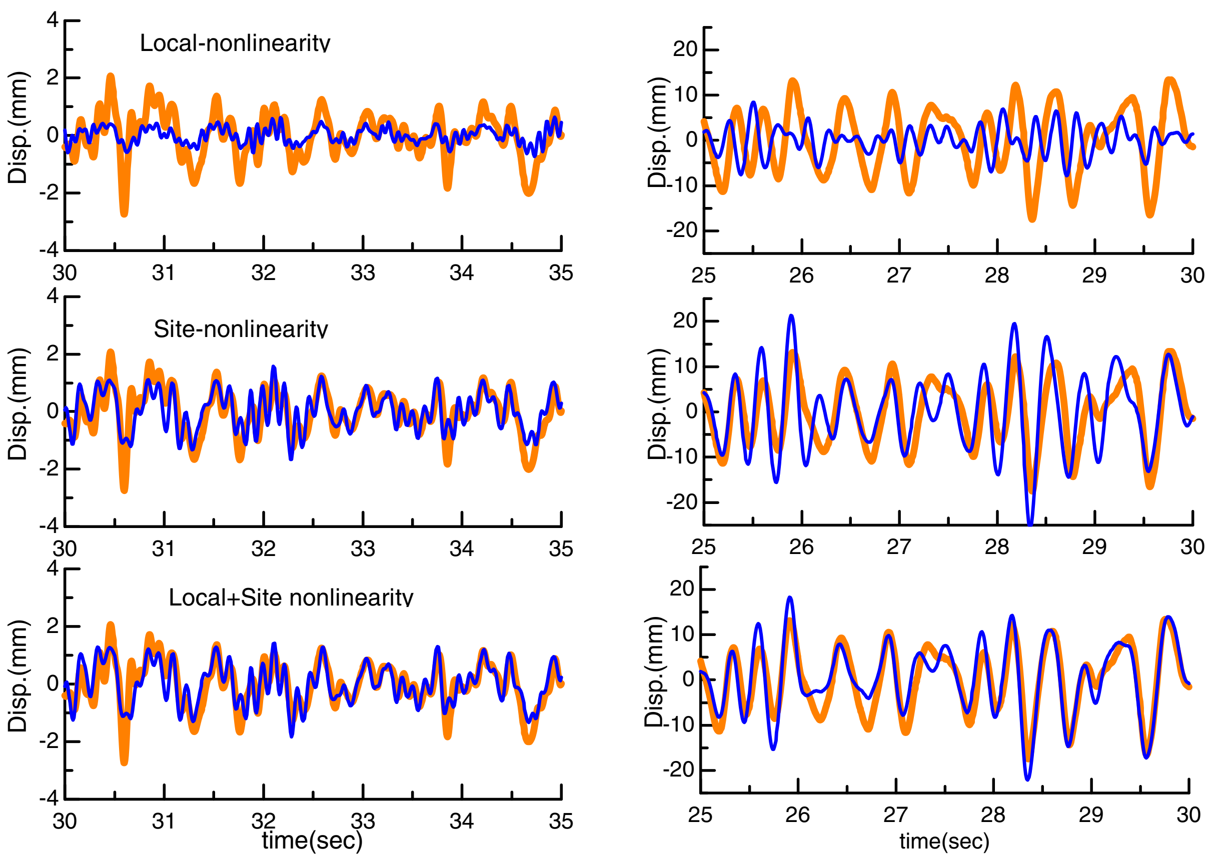
SN-model can also simulate the experimental results reasonably. The results from the LN-model, however, are quite different from the experimental results. These facts suggest that the site-nonlinearity effects predominate when the mass of the structure is small and the inertia force transmitted from superstructure is small.

Case for W=450 [kN]

The simulated results from the proposed model agree quite well also in the cases with W=450 (kN). The result from the SN-model, which provide good agreement for the case of W=150 (kN), does not work well for the case of W=45 (kN). This fact suggests that the local-nonlinearity, as well as the local-nonlinearity, affect the response of pile foundation during the excitation because the weigh is larger. The characteristics of nonlinear soil-pile interaction can be summarized as follows.

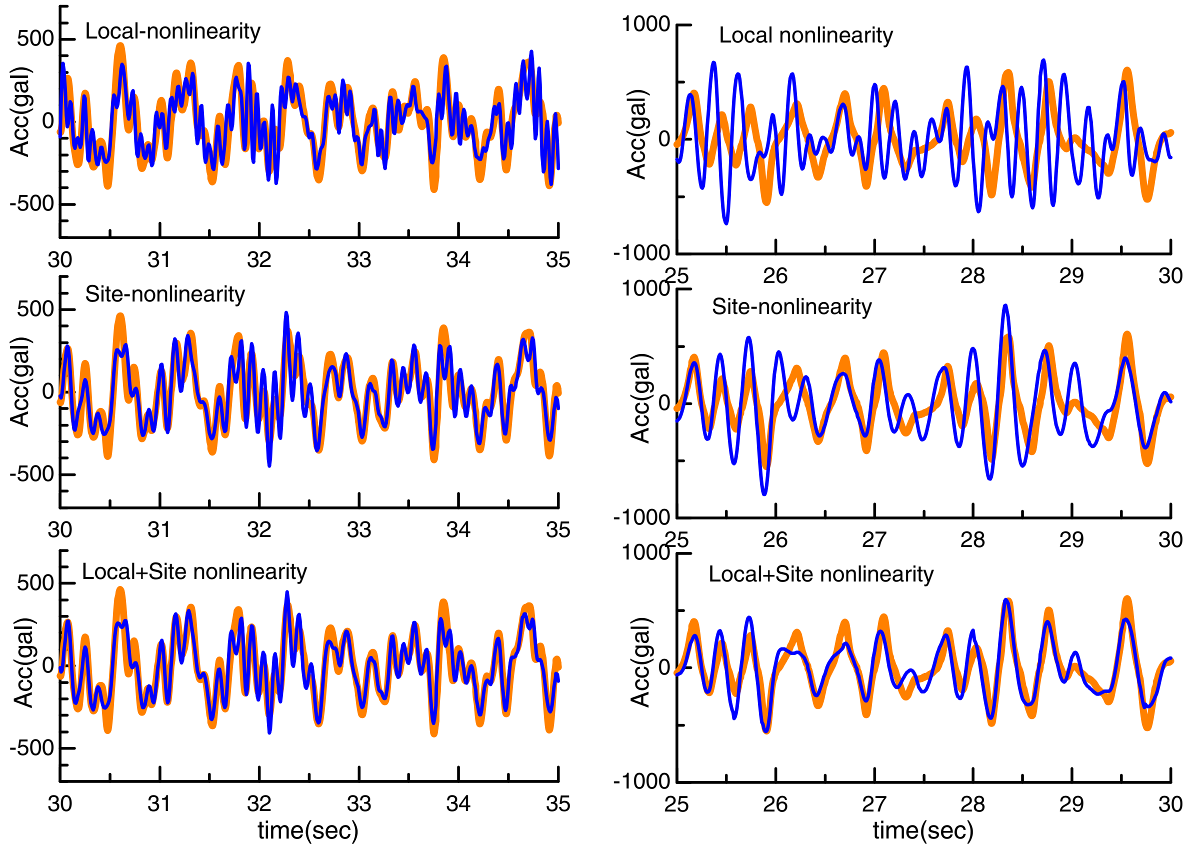
1) If the weight of superstructure is small, the inertia force loaded on the pile-cap is small. As a result, the soil around the piles is little deformed and the local-nonlinearity does not appeared. The effect of the site-nonlinearity is larger than that of the local-nonlinearity.

2) If the weight of superstructure is large, the inertia force loaded on the pile-cap becomes large. As a result, the soil around the piles is deformed largely and the surrounding soil will behave in a nonlinear manner (*i.e.* the local-nonlinearity). Thus, the response of the structure can not be specified by taking into account only the site-nonlinearity.



(a) W=150 (kN) type (b) W=400 (kN) type

Fig.9 Simulated response displacement compared with the test result (400gal input)



(a) W=150 (kN) type (b) W=400 (kN) type

Fig.10 Simulated response acceleration compared with the test result (400gal input)

CONCLUSIONS

A simple and valid approach is developed to evaluate the nonlinear dynamic soil-pile which can be used in analyzing an earthquake response of pile-supported structure.

(a) A hyperbolic equation is incorporated into the *p-y* curves in order to take into account the effect of the local-nonlinearity. In order to evaluate the site-nonlinearity, the Thin Layer Element Method (TLEM) is adopted step by step to determine the stiffness function $K_0(t)$ which is varied by the shear strain at each time step.

(b) An accuracy of this method is confirmed by comparing the simulated results and the experimental results.

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