EVALUATION OF SEISMIC FORCE OF PILE FOUNDATION INDUCED BY INERTIAL AND KINEMATIC INTERACTION

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SUMMARY

Characteristics of pile forces induced by earthquakes are discussed from the viewpoint of the inertial and kinematic force. In most seismic design codes, pile foundations are designed merely against inertial force. However, soil deformation caused by seismic waves generates curvature of piles and subsequently a bending moment along their whole length. In this paper, the dynamic response of laterally-vibrating pile foundation is investigated through numerical studies on a soil-pile-structure system with different ratios of the natural period of structure (*Ts*) to that of soil deposit (*Tg*). Both linear and non-linear seismic analyses have been performed for comparison. Then, a practical method to evaluate the pile force induced by not only inertial response but also by kinematic response is proposed.

INTRODUCTION

Pile foundation structure built in a soft ground receives both influences of the Kinematic interaction and the Inertial Interaction. As a result, pile forces are induced by these two interactions during earthquakes. Many of failures arose from the transmission onto the foundation of large inertia forces. However, in several cases, the location of pile failure is too deep to be caused by loading from the top [Mizuno, 1987]. This type of failure is caused by the soil deformations (the kinematic interaction). The potential of the latter failure has not received proper attention in the seismic design. In fact, engineers design the piled foundation merely against the inertia force. In order to establish a rational seismic design method of piled foundations, it is needed to elucidate the effects on the occurrence of pile forces of both the inertia force and the soil deformation.

We have studied about characteristics of pile forces induced by earthquakes and discussed from the viewpoint of the inertial force and the kinematic force by the analytical and the experimental methods [Murono *et al*, 1998; Nishimura and Murono *et al*, 1998]. This paper intends to propose a specific method to evaluate pile forces caused by these two effects based on results of more detailed parametric studies of linear and non-linear soil-pile-structure seismic analyses.

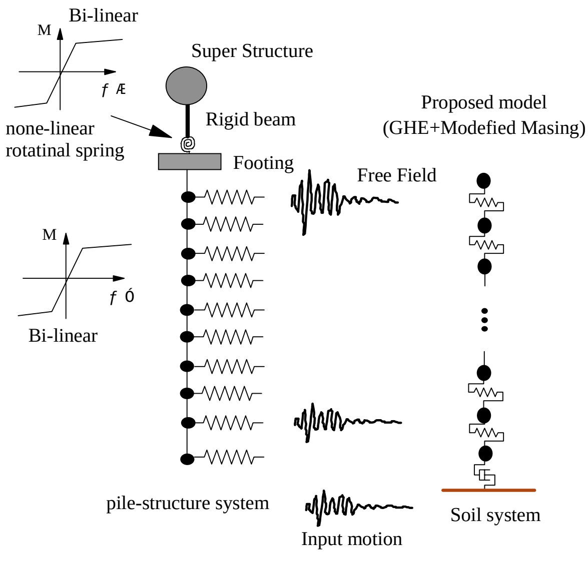

Figure 1: Simplified soil-pile-structure model

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ANALYTICAL MODEL

A typical pile-supported railroad bridge constructed in a soft ground is mainly discussed in this paper. The soil is assumed to be one-layer. The soil and structure conditions are defined in **Tab.1**. The initial predominant period of soil deposit Tg_0 is 0.5(s). A simplified soil-pile-structure model is described in **Fig.1**, in which the soil is represented by springs and dashpots distributed along the pile. The analysis is performed in two stages.

Table 1: Soil and structural propaties

			value
Soil	Shear Velocity	Vs(m/s)	100.0
	Layer Depth	H(m)	20.0
	Unit weight	γs(kN/m^3)	1.60
Structure	Super Structure	Wu(kN)	1968.5
	Footing	Wp(kN)	805.8
	Damping	hs	0.10
	Period	Ts(s)	0.6 □ 2.5
Pile	Diameter	D(m)	1.20
	Young modulus	E(kN/m^2)	2.5×10^7
	unit weight	γp(kN/m^3)	2.5
	Damping	hp	0.05

Free-Field Soil Analysis

At a large distance from pile foundation (so called the free-field), soils are less affected by the motions of these piles, and the one-dimensional wave propagation is adequately assumed for the behavior of layered soil deposits. The non-linear time history analysis is used to compute the non-linear free-field motions. A viscous boundary is considered at the bottom of the soil layer in order to absorb the reflection wave. A nonlinear shear stress-strain model of soil which was proposed by the authors [Nishimura and Murono, 1999] is used in the analysis. The backbone curve is constructed based on General Hyperbolic Equation [Tatsuoka and Shibuya, 1992] and the hysteresis loop is constructed based on the Masing's 2nd rule with some modifications.

Pile-Structure System Analysis

The soil reaction to the motion of piles is described by the soil springs and dashpots distributed along the pile. The soil spring stiffness is evaluated by the standard design code for railway bridges. The shear modulus used to calculate it is reduced due to the non-linearity of soil properties caused by the shear strain during S-wave propagation, which is evaluated form the free-field analysis. The soil springs are treated as linear in this paper.

The periods of structures are varied from 0.6 to 2.0(sec) by some adjustment of pier height. Four cases are examined as shown in **Tab. 2**. Because a plasticity hinge occurs at the pier bottom end, the bridge pier is modeled as a rigid beam and a non-linear rotational spring arranged at the pier bottom. The non-linear rotational spring is idealized as the bi-linear bending moment□rotation relation (M-θ), and the ratio of post-yield stiffness to elastic stiffness is 5 %. The yielding seismic coefficient k_{hys} is assumed to be 0.3,0.5 and 0.8. About a pile, the non-linear model is expressed with a bending moment□curvature relation (M-φ). The yielding seismic coefficient k_{hyf} is assumed to be 0.4.

Table 2: Four cases for analyses

	Free-Field	Pier	Pile	Input
Case001	Linear*	Linear	Linear	Sine wave
Case101	Non-Linear	Linear	Linear	Earthquake wave
Case102	Non-Linear	Non-Linear	Linear	Earthquake wave
Case103	Non-Linear	Linear	Non-Linear	Earthquake wave

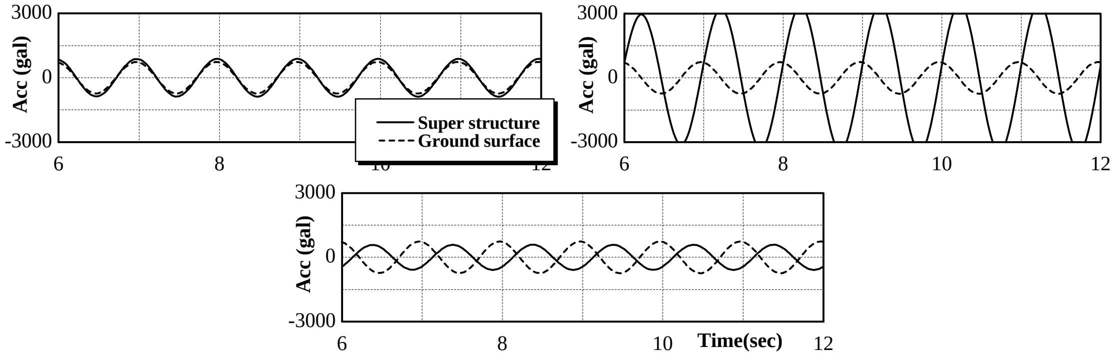
* The shear velocity used in the analysis is reduced from the initial shear velocity (=0.5*V_{s0})

RESULTS OF ANALYSIS

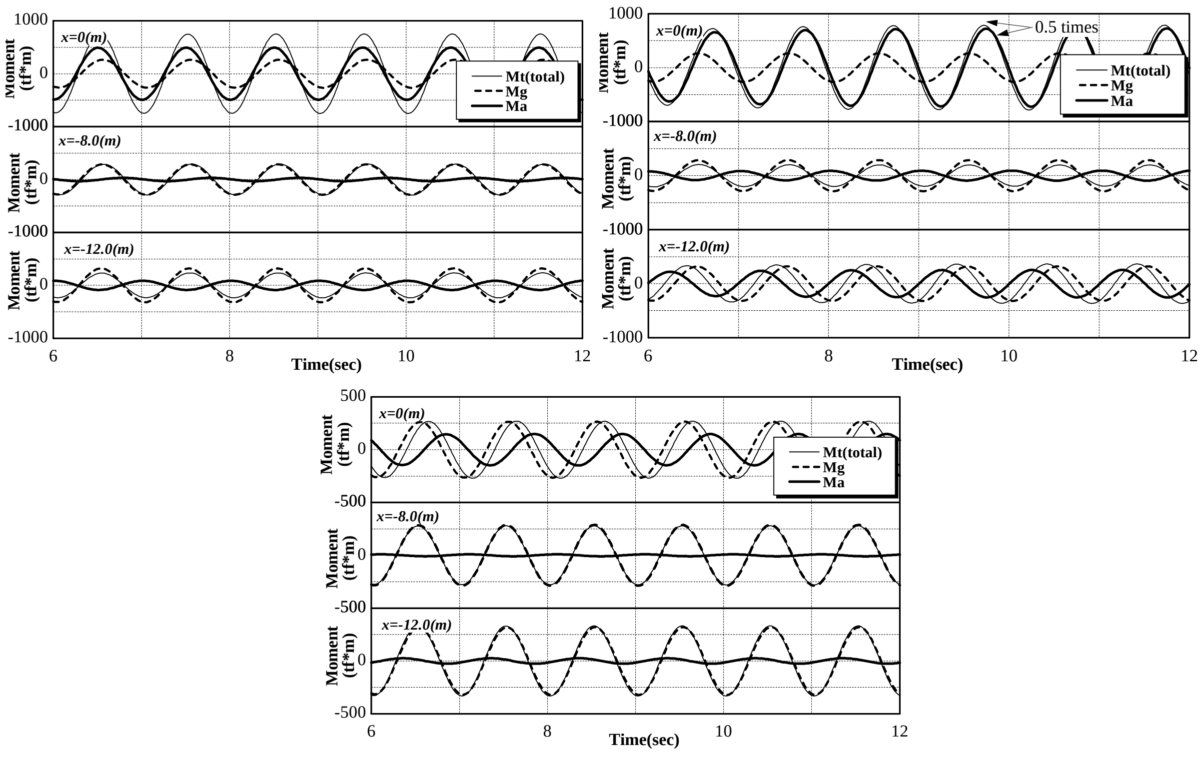
Linear Seismic Response Characteristics of Soil-Pile-Structure System

Sine wave input (Case001)

Fig. 2(a) is the pier top and soil surface acceleration time histories when structures are excited by a sine wave whose period is the same as the ground predominant period Tg. The soil and the structure vibrate with the same phase in the case of $Ts<Tg$, with a 90-degree difference in the case of $Ts=Tg$, and with the inverse phase in the case of $Ts>Tg$. **Fig. 2(b)** is the bending moment time histories at different depths along the pile. Ma and Mg represent the bending moments caused by the inertial force and the soil deformation, respectively. Mt represents the total bending moment. *Mg* is computed by the model whose mass is set to zero. *Ma* is defined as the value of *Mt* from which *Mg* is subtracted (*Ma* =*Mt*- *Mg*). 1) At the pile head (x=0(m)); Both *Ma* and *Mg* arise with the same phase in the case of $Ts<Tg$. In the case of $Ts=Tg$, *Ma* and *Mg* arise with a 90-degree difference and *Ma* is much predominant because the structure vibrates resonating with the soil layer. In the case of $Ts>Tg$, *Ma* and *Mg* arise with the inverse phase and



(a) Acceleration time histories of pier top and ground surface



(b) Bending moment time histories at different depths

Figure 2: Response time histories of linear models excited by sine wave input

Mg becomes more predominant than *Ma* because the response acceleration of structure becomes small. 2) At x=-8(m) and –12(m) depth: In all cases *Ma* becomes suddenly small with depth, and the phase of *Ma* turns over in the pile head and the deep position. But *Mg* changes smoothly in depth direction, the influence of *Mg* becomes predominate relatively at this deep position (*Mt* is nearly equal to *Mg*). When only the inertial force is considered in a seismic design of piled foundations, it is impossible to evaluate the moment occurred at this depth. In the seismic design, it was cleared that both of *Mg* and *Ma* must be considered adequately.

Fig. 3 shows the relationship between the normalized soil surface displacement $\delta(t)$ and the normalized pier top acceleration $a_{cc}(t)$, which are defined as follows.

$$\delta(t)=\delta(t)/|\delta_{\max}| \quad \text{and} \quad a_{cc}(t)=a_{cc}(t)/|acc_{\max}| \tag{1}$$

in which $\delta_{\max}$ is the maximum response displacement at ground surface, and acc_{max} is the maximum response acceleration at pier top. The sign $\triangledown$ and $\blacktriangledown$ show the point which mean that $\delta(t)=\delta(t)/|\delta_{\max}|=\pm1$ and $a_{cc}(t)/|acc_{max}|=\pm1$, respectively.

In the case of sine wave input, the orbit describes a narrow ellipse inclined by about 135 degrees. This means that the soil deformation and the inertia force act on the pile in the same direction. And the inertia force and the soil deformation become largest value at the same time. As *Ts* becomes near *Tg*, the orbit begins to trace a wider ellipse (almost a circle). This shows that the phase difference between the ground and the structure behavior is nearly 90 degrees. As a result, when one of the two effects becomes maximum, other one becomes roughly zero. In the case of *Ts*>*Tg*, the orbit becomes an ellipse about 45 degrees inclined. This means that the inertia force and the soil deformation act on the pile in the reverse direction.

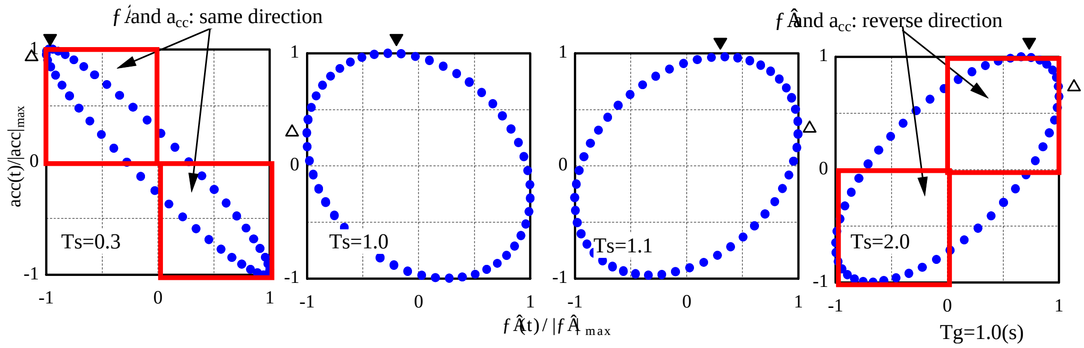


Figure 3: Relationship between soil and structure behaviour for a linear model (sine wave input)

Earthquake wave input (Case101)

Fig. 4 shows the input earthquake motion at base rock used in the analyses. This earthquake motion is the standard design earthquake of dislocation neighborhood used in Japanese railway facilities, which is simulated based on the rupture process [Sato, Murono *et al* ,1999].

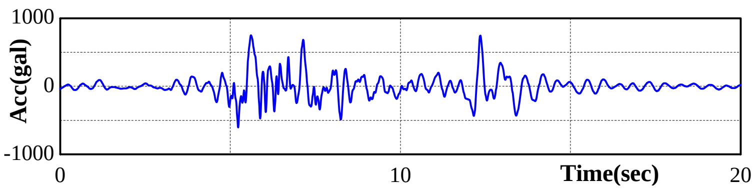


Figure 4: Input base ground motion

The response characteristics for the earthquake input are much complicated, but the $\delta(t)-a_{cc}(t)$ relations are almost the same as those for sine wave input (see **Fig. 5**). In this figure, the sign and show the occurrence time of maximum values of pile moments at *x*=0 and -8(m) depth. It can be said that the pile head moment takes a maximum value when the inertia force becomes max value and that the pile moment at -8(m) depth takes a maximum value when the soil deformation becomes max value. As a result, there is a time lag between the times when those two moments become the maximum. It is necessary for piles to resist these two seismic effects. These results suggest that a two-step design method is needed.

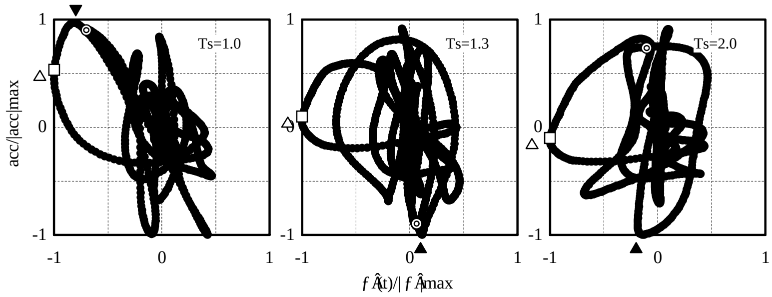
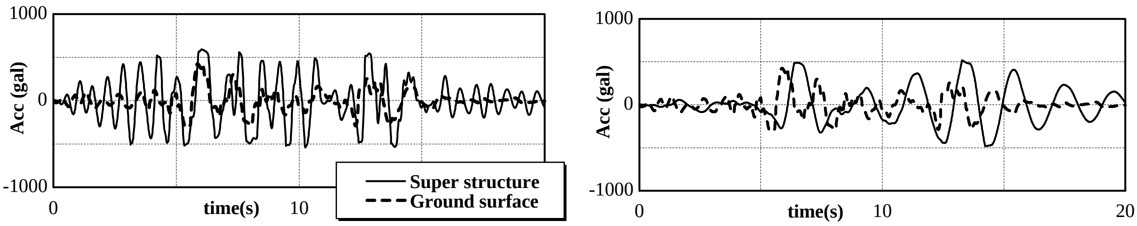


Figure 5: The relationship between soil and structure behaviour for linear model (Earthquake input)

Non-linear Seismic Response Characteristics of Soil-Pile-Structure System

Effects of pier non-linearity (Case102)

The results of non-linear earthquake response analyses of soil-pile-structure systems whose yielding seismic coefficient k_{hys} 0.5 are discussed as an example here. The response acceleration time histories at the pier top and the ground surface are compared in **Fig. 6**. The phase difference between the structure and the ground behavior varies with the relationship of their periods. This trend is the same as that of linear model. But the maximum response acceleration of the pier top is reduced greatly due to the pier yielding and the shape of response wave becomes flat at the yielding seismic coefficient (about 500(gal)).



(a) Ts=0.6(s), Tg=1.3(s) (b) Ts=2.0(s), Tg=1.3(s)

Figure 6 : Response time histories at pier top compared with ground surface

Fig. 7 shows the relationship between normalized displacement $\delta(t)$ and acceleration $a_{cc}(t)$. Response ductility values μ of piers are also shown in the Figs. Even if the bridge pier yields, the general tendency of the $\delta(t)-a_{cc}(t)$ relation is the same as that of the linear model but there is one significant difference. The orbit for the linear model is tangent to the cross-axis and the vertical-axis only at one point (**Fig. 5**). On the other hand, the orbit for the non-linear model is tangent to these axis on some lines.

Effects of pile non-linearity (Case103)

The influence of pile yielding is examined. **Fig.8** shows the $\delta(t)-a_{cc}(t)$ relationship for the pile yielding model. The $\delta(t)-a_{cc}(t)$ relationship is very similar to that of the linear model even though the pile yields. This is due to the model assumptions that the soil springs were constrained to be linear and the rotation of footing was fixed.

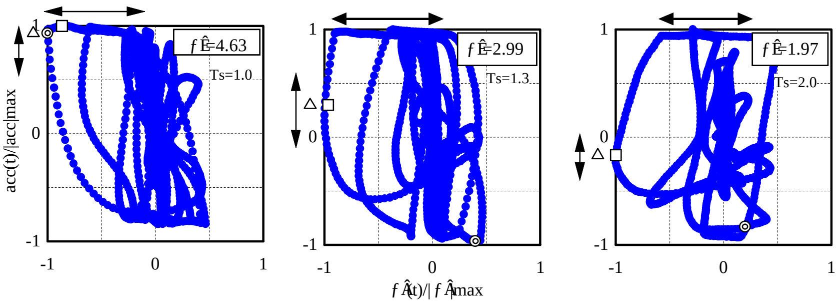


Figure 7: The $\delta(t)-a_{cc}(t)$ relation of non-linear mode (Pier yielding).

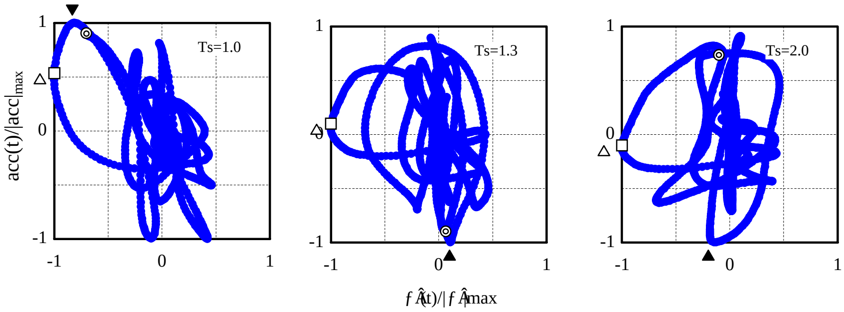


Figure 8: The $\delta(t) - a_{cc}(t)$ relation of non-linear mode (Pile yielding)

A PROPOSAL OF A SEISMIC DESIGN METHOD

"The Seismic Deformation Method" is the equivalent static analysis that takes into consideration the pile bending moment caused by the soil deformation [JSCE, 1988]. In this method, the surrounding soil deformation is applied statically along a pile whole length through the soil springs. The schematic of the seismic deformation method is shown in **Fig. 9**. The combination of the inertia force and the soil deformation (kinemacic force) becomes a very important problem in applying this method to a pile design.

Combination of the Soil Deformation and the Inertial Force

A proposal of two-step seismic design method

The two types of seismic effects, such as the inertia force the soil deformation, must be considered in the seismic design. This is expressed with by the following equation.

$$R_t = \beta \times R_a + \gamma \times R_g \tag{2}$$

in which *Rt* is the seismic effects to be considered; *Ra* is the inertia force; *Rt* is the soil deformation and β, γ are coefficients to combine *Ra* and *Rt*. The combination of β and γ changes at every moment during an earthquake, but the severest combination for the pile stress must be considered. In order to design pile foundations against an earthquake, the following "two-step design" concept is proposed.

Step1; The design mainly for the inertia force.

$$R_t = 1.0 \times R_a + \gamma \times R_g \tag{3}$$

Step2; The design mainly for the soil deformation.

$$R_t = \beta \times R_a + 1.0 \times R_g \tag{4}$$

Based on the analytical results of Chapter 3, the values of these combination coefficient β and γ are decided.

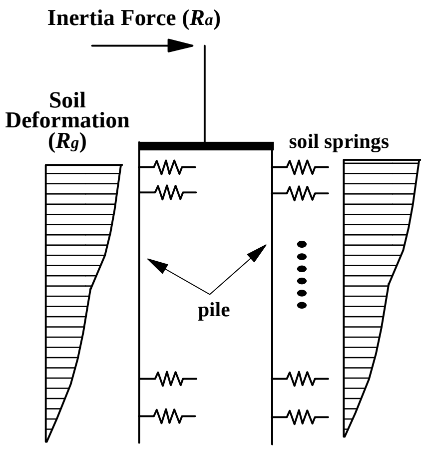


Figure 9: Schematic of the seismic deformation method

The combination coefficient

From the results of dynamic analyses, the value β and γ is decided as follows.

$$\beta = a_{cc}(t_g)/a_{cc_{\max}}, \quad \gamma = \delta(t_a)/\delta_{\max} \tag{5}$$

where t_g and t_a represent the occurrence time of maximum value of the soil deformation and that of structure, respectively. **Fig. 10** shows the results of all analysis cases. In this figure, the former work [murono *at al*, 1998] is described too. The cross axis is the ratio α of structure period *Ts* and ground period *Tg*. *Tg* represents a predominant period of soil, and *Ts* represents a period of soil-pile-structure system. Though there are some dispersions due to the differences in the conditions, the values of β and γ reduce as α increases for the linear models. This tendency indicates the following fact; as the period of structure becomes long, the phase difference between the behavior of the soil layer and the structure becomes greater as mentioned in chapter 3. The values of β and γ are almost equal to those of the linear model even if the pile member yields. On the other hand, the values of β and γ tend to approach 1.0 and do not depend on the period ratio α when pier yields. This is because the acceleration wave shape becomes flat due to the pile yielding (see Chapter 3). Based on these results, the upper limit value ν_U and the bottom value ν_L are proposed like the solid lines in the figure to include the dispersion of analytical results.

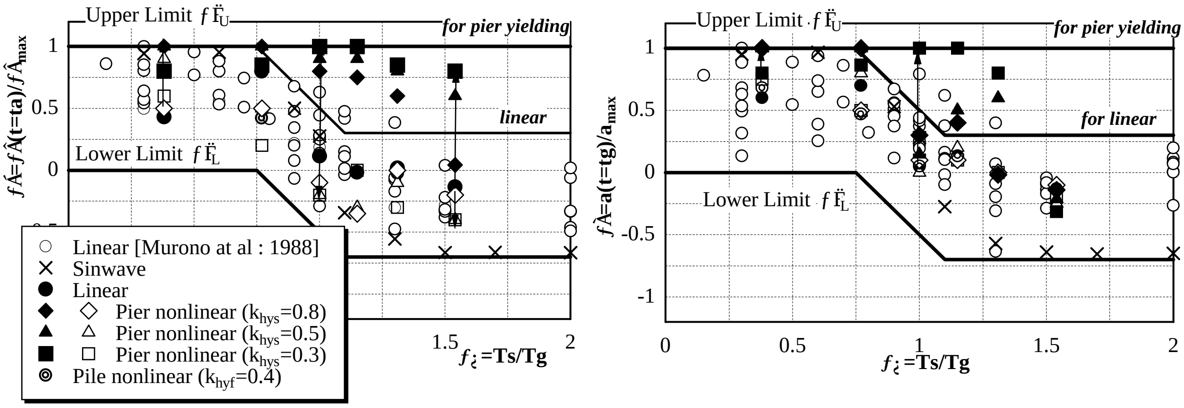
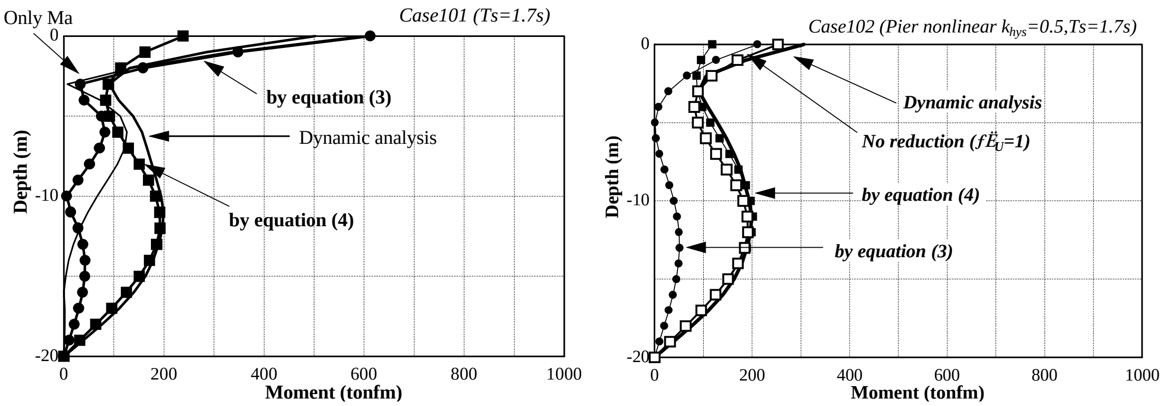


Figure 10: The combination coefficients β and γ

Fig.11 shows the bending moments of the pile computed statically by the proposed method (the two-step seismic deformation method), which are compared with the results of dynamic analyses. The results of the proposed method agreed with those of dynamic analyses very well, and it was verified that the proposal method is very effective in the seismic design of pile foundations.



(a) Linear model (*Ts*=1.7s) (b) Non-linear model (*Ts*=1.7s)

Figure11: Bending moments computed by the proposed method compared with those of dynamic analyses

CONCLUSION

Characteristics of pile forces induced by earthquakes were discussed from the viewpoint of a inertial and a kinematic force (a soil deformation). Major conclusion derived from this study may be summarized as follows;

1) For the linear model: The seismic response of soil-pile-structure system is much dependent on the relationship between the period of structure *Ts* and soil deposit *Tg*. If $Ts<Tg$, the inertia and the kinematic loading will act on the pile with nearly the same phase, while $Ts>Tg$, the delay in phase between them will be very large.

2) For the pier yielding model: Though inertial force itself is reduced due to the pier yielding, the possibility becomes high that the soil displacement and the inertial force take maximum values simultaneously, because the acceleration wave pattern has a flat shape.

3) The practical seismic design method for piled foundation (the two-step soil deformation method) was proposed base on the characteristics mentioned above.

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