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# Dynamic behavior of group-pile foundation evaluated by simplified model and sophisticated model

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## SUMMARY

In this paper, a series of numerical analyses using dynamic finite element method are conducted to investigate the seismic behaviors of group-pile foundation. Two kinds of models for a ground-pile foundation-superstructure system are used. Model 1 (M1) is a simplified model, in which, the interaction between piles and ground is represented by springs and the ground-pile foundation-superstructure system is simplified to a frame-spring model. Model 2 (M2) is called as sophisticated model, in which, the system is modeled with three-dimensional finite elements without simplification. The purpose of the research is to verify the applicability of the dynamic analysis with M1 model that is commonly used in ordinary seismic design and to establish a way of simplification from M2 model to M1 model without losing too much accuracy of the analysis by M1 model.

## INTRODUCTION

It is known that during a strong earthquake, dynamic behavior of a group-pile foundation is related to not only the inertial force from superstructures but also the deformation of surrounding ground. During the Hyogoken-Nambu earthquake, it is found from field observations (Horikoshi et al., 1996) that even in the absence of a superstructure, piles failed because of the deformation of the surrounding ground. The moments developed due to the deformation of ground are usually referred to as kinematic moments. In evaluating the seismic behavior of a group-pile foundation, 3D dynamic finite element analysis using a full model, which consists of superstructures, a foundation and a ground, is regarded to be effective and executable nowadays when the nonlinearity of the superstructure, the piles and the soils is properly considered (Zhang & Kimura, 2002). In ordinary seismic design, however, commonly used model is a frame-spring model (denoted as M1 model) because dynamic analysis with three-dimensional finite elements (denoted as M2 model) is still too difficult to prosecute for engineers. Many simplified assumptions, however, have been included in the analysis with M1 model and therefore it is necessary to verify the accuracy of the analysis with M1 model.

Unfortunately, few researches were found in relation to the verification of the accuracy of the analysis with M1 model. Mori et al. (2002) conducted a series of analyses with different methods using M1 and

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M2 models to investigate the difference among the calculated results from different methods. Because the calculations were conducted by different researches, the calculating conditions for the same problem were not able to be consistent. Therefore, it is hard to say the results obtained in the research are applicable. Zhang et al. (2003) also conducted numerical analyses on group-pile foundation with M1 and M2 models with the same program named as DGPILE-3D (Zhang & Kimura, 2002) and concluded that without careful treatment on the simplification of M1 model, the results from M1 and M2 models may be totally different when considering the nonlinearity of both ground and piles.

The purpose of this research is to verify the applicability of the dynamic analysis with M1 model and to establish a way of simplification from M2 model to M1 model without losing too much, based on the assumption that the dynamic analysis with M2 model is an accurate method in seismic design of group-pile foundation. In the paper, dynamic analyses with M1 and M2 models in different types of ground, and different types of pile foundation are conducted and their results are carefully examined. All the numerical analyses are conducted with the same program DGPILE-3D.

## COMPARISON OF DYNAMIC ANALYSES ON DIFFERENT PILE FOUNDATION WITH M1 AND M2 MODEL

In order to make clear the difference between the dynamic analyses with M1 and M2 models, two types of ground, one is a 1-layer ground and another is a 6-layer ground, and two types of pile foundations, one is single pile and another is 9-pile foundation, are considered in the numerical analyses. For 9-pile foundation, a real elevated highway bridge with 3x3 cast-in-place reinforced concrete piles, in which the center-to-center distance of the group piles is 2.5D, is considered. Table 1 shows the physical and geometrical properties of single and group piles. As shown in Figure 1, the 6-layer ground is composed of sandy soils and clayey soils, while the 1-layer ground is made of clayey soil. The material parameters of both grounds are listed in Table 2 and Table 3, respectively.

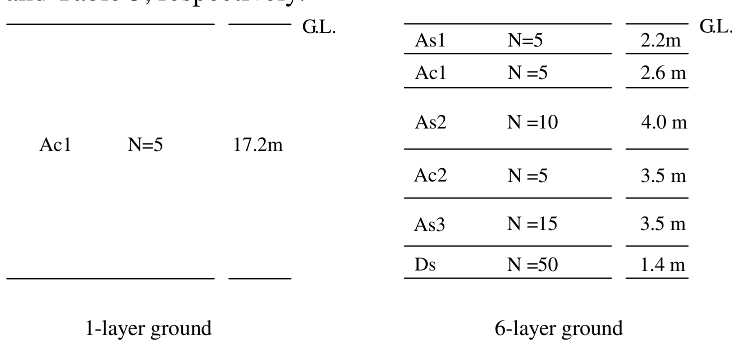



**Fig. 1 Geologic profiles of 1-layer ground and 6-layer ground**

**Table 1 Physical and geometrical properties of piles**

| Foundation type | Diameter | Length | Yield strength | Young's modulus (Reinforcement) | Young's modulus (Concrete) | Sectional moment |
|---|---|---|---|---|---|---|
| | D (m) | L (m) | $\sigma_y$ (MPa) | $E_S$ (kPa) | $E_C$ (kPa) | I ($m^4$) |
| 9-pile | 1.2 | 15.0 | 295 | 2.000E+08 | 2.500E+07 | 1.018E-01 |
| Single pile | 2.0 | 29.7* | 295 | 2.000E+08 | 2.500E+07 | 7.85E-01 |

*In single pile, the pile length is the length from the pile tip to the mass of superstructure.

**Table 2 Material parameters of soil in 1-layer ground**

| Layer | Thickness (m) | Density ($g/cm^3$) | Poisson's Ratio | Void ratio | Stress ratio at failure | Compression index | Swelling index |
|---|---|---|---|---|---|---|---|
| | $H$ | $\rho$ | $\nu$ | $e_0$ | $R_f$ | $\lambda$ | $\kappa$ |
| $A_{C1}$ | 17.2 | 1.7 | 0.40 | 0.88 | 3.5 | 0.0391 | 0.0224 |

**Table 3 Material parameters of soils in 6-layer ground**

| Layer | Thickness (m) | Density ($g/cm^3$) | Poisson's Ratio | Void ratio | Stress ratio at failure | Compression index | Swelling index |
|---|---|---|---|---|---|---|---|
| | $H$ | $\rho$ | $\nu$ | $e_0$ | $R_f$ | $\lambda$ | $\kappa$ |
| $A_{S1}$ | 2.2 | 1.7 | 0.30 | 0.93 | 4.5 | 0.0434 | 0.0240 |
| $A_{C1}$ | 2.6 | 1.7 | 0.40 | 0.88 | 3.5 | 0.0391 | 0.0224 |
| $A_{S2}$ | 4.0 | 1.7 | 0.30 | 0.93 | 4.6 | 0.0324 | 0.0192 |
| $A_{C2}$ | 3.5 | 1.7 | 0.40 | 0.88 | 3.5 | 0.0368 | 0.0197 |
| $A_{S3}$ | 3.5 | 1.9 | 0.30 | 0.87 | 4.7 | 0.0184 | 0.0160 |
| $D_S$ | 1.4 | 1.9 | 0.30 | 0.65 | --- | --- | --- |

For 9-pile foundation in M1 model, three piles in each row are combined to one so that the simplified model can be used in two-dimensional analysis. Figure 2 shows the simplified M1 model of the 9-pile foundation.

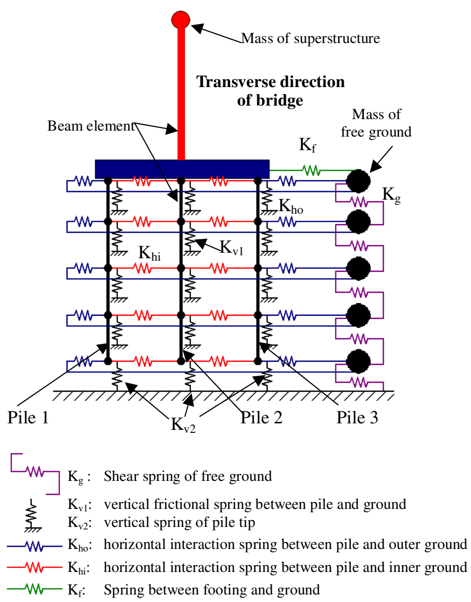



**Fig. 2 Simplified M1 model of ground-pile foundation-superstructure (3x3 piles)**

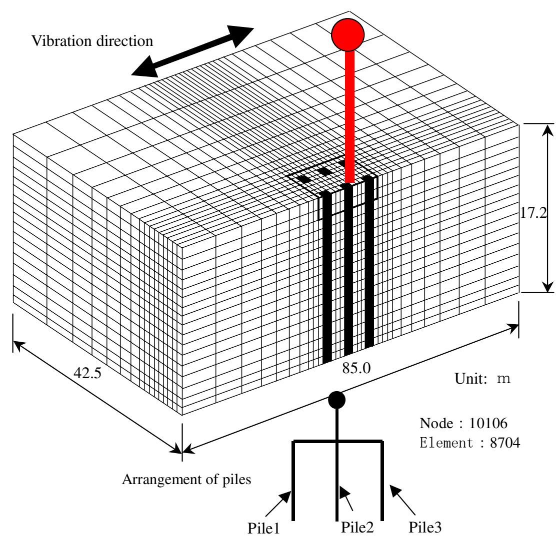



**Fig. 3 Finite element mesh (M2 model 3x3 piles)**

The springs representing the interaction between pile and surrounding ground in horizontal and vertical directions, the interaction between footing and the surrounding ground, and the shear spring representing the connection of free ground slices, are determined by Eq.1~2. And the variants in Eq. 1~2 are evaluated according to the Design Codes of Foundations and Earth-Retaining Structures of Japan Railway (Japan Ministry of Transportation, 1997). Detailed description about the evaluation of these equivalent springs can be referred to corresponding references, e.g., Mori, 1997 and the aforementioned design code.

For simplicity, earthquake wave is input in one direction. Therefore, in M2 model with 9-pile foundation, due to the symmetrical conditions, only six piles are considered, as shown in Figure 3. In order to make the comparison between the analyses using M1 and M2 models more easily, the first row of piles are called as Pile 1, and so on. A Newmark-$\beta$ direct integration method is used in all dynamic analyses. A Rayleigh type of damping is adopted and the damping factors of the structures and the ground are assumed as 2%. Figure 4 shows the input wave for all the dynamic analyses conducted in this paper.

Free ground spring $Kg$

$$Kg = \frac{A \cdot E_0}{2(1+\nu)} \cdot \frac{1}{t}$$

Lateral pile-ground interaction spring $K_{ho}$

$$K_{h0} = \frac{D}{2} \cdot (t_{i-1} \cdot k_h{}^{i-1} + t_i \cdot k_h{}^{i})$$

Vertical pile-ground interaction spring $K_{v1}$

$$K_{v1} = \frac{\pi D}{2} \cdot (t_{i-1} \cdot k_{v1}{}^{i-1} + t_i \cdot k_{v1}{}^{i})$$

Pile-to-pile interaction spring $K_{hi}$

$$K_{hi} = \lambda \cdot K_{ho}$$

Vertical spring in pile tip $K_{v2}$

$$K_{v2} = k_{v2} \cdot \frac{\pi D^2}{4} + \frac{1}{2} \cdot t_n \cdot \pi D \cdot k_{v1}{}^{n}$$

Footing-ground interaction spring $K_f$

$$K_f = k_{hf} \cdot A_h \qquad (1)$$

Yielding force of ground spring

$$P_y = X_f \cdot \sigma_m \cdot A$$

$X_f$ : stress ratio in critical state defined in tij model

$\sigma_m$ : mean stress of current ground slice

$A$ : Area of free ground, assumed as 100 times of the footing area in current study.

Yielding force of horizontal pile-ground interaction spring

$$P_{vy}{}^{j} = \frac{\pi D}{2}(t_{i-1} \cdot P_{ev}{}^{i-1} + t_i \cdot P_{ev}{}^{i})$$

Yielding force of lateral pile-ground interaction spring

$$P_{hy}{}^{j} = \frac{D}{2}(t_{i-1} \cdot P_{eh}{}^{i-1} + t_i \cdot P_{eh}{}^{i}) \qquad (2)$$

**Comparison of seismic responses of free grounds using M1 and M2 models**

*Under elastic condition*

First of all, seismic responses of free ground with M1 and M2 models are investigated. In M1 model, the free ground is simply modeled with a column of masses and springs as shown in the left side of Figure 5. In M2 model, the ground is modeled with a 3D finite element mesh without piles and pier as shown in the right side of Figure 5. Figures 6 and 7 show that under elastic condition, the responses of the accelerations and the displacements of 1-layer ground and 6-layer ground from both models are totally the same, implying that the simplification involved in M1 model is completely acceptable in seismic evaluation of ground under elastic condition.

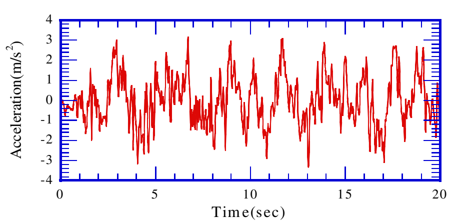



**Fig. 4 Input earthquake wave**

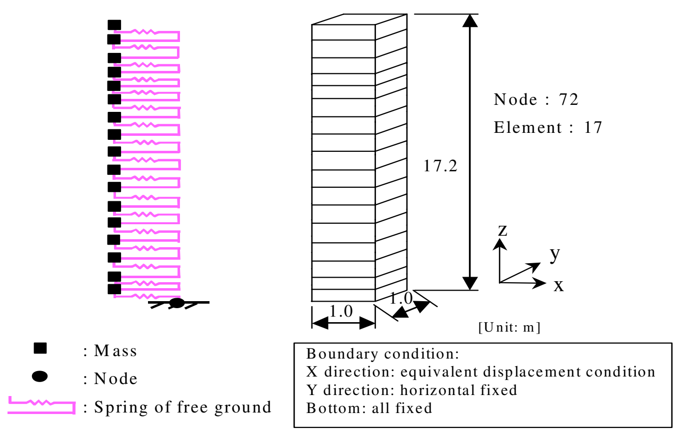



**Fig. 5 M1 and M2 models of free ground**

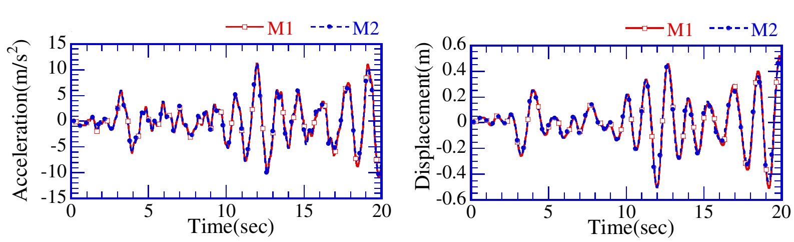



**Fig.6 Seismic response at ground surface from M1 and M2 models (1-layer ground, elastic)**

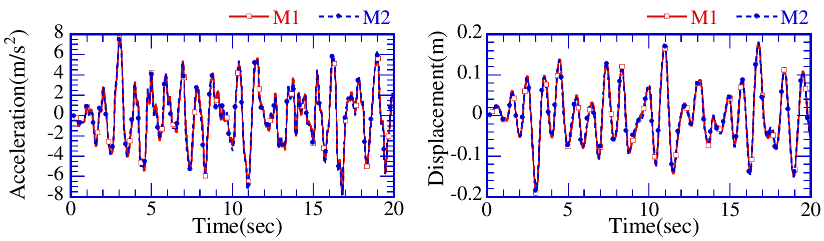



**Fig.7 Seismic response at ground surface from M1 and M2 models (6-layer ground, elastic)**

*Under elastoplastic condition*

If the ground is considered under elastoplastic condition, subloading tij clay (Chowhury, 1999) and tij sand (Nakai, 1989) models are used in M2 model, and Ramberg-Osgood (R-O) model is used for ground spring in M1 model. By transforming the stress-strain relation defined by R-O model in such a way that $P$=(shear stress τ)*(Area of free ground) and $\delta$=(shear strain ε) *(thickness of ground slice), the force-displacement relation can be obtained, as shown in Eq. 3. The calibration of parameters in R-O model should obey such a rule that by adjusting the material parameters involved in the model, the stress-strain relation should be kept as close to tij models as possible, as shown in Figure 8.

$$\delta = \frac{P}{K_{g0}}\left(1+\alpha\left|\frac{P}{P_y}\right|^{r-1}\right) \tag{3}$$

$K_{g0}$: initial stiffness,
$\delta$: displacement of spring,
$P$: force imposed on spring,
$P_y$: yielding force at failure state,
$\alpha$, $r$: material parameters.

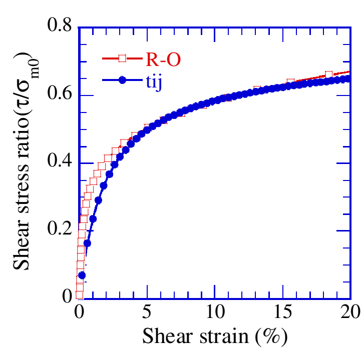



**Fig.8 Calibration of parameters involved in R-O model for $A_{s1}$ soil**

Figures 9 and 10 show the responses of the accelerations and the displacements at the surface of the ground from M1 and M2 models for 1-layer ground and 6-layer ground under elastoplastic condition. Since the R-O model cannot reflect the mechanical ratcheting phenomenon in geomaterials, the amplitude of response acceleration predicted by M1 model did not decrease during the earthquake, as shown in Figure 9 and 10. Therefore, the results from M1 and M2 models coincided with each other at the first several vibrations and then error accumulated to an annoying level. Also it can be seen that the results in 1-layer ground are better that those in 6-layer ground. One thing, however, can be confirmed that the maximum values of accelerations and displacements in both grounds are almost in the same level. These imply that the simplification involved in M1 model can be acceptable only under certain restricted aspects in seismic evaluation of ground under elastoplastic condition.

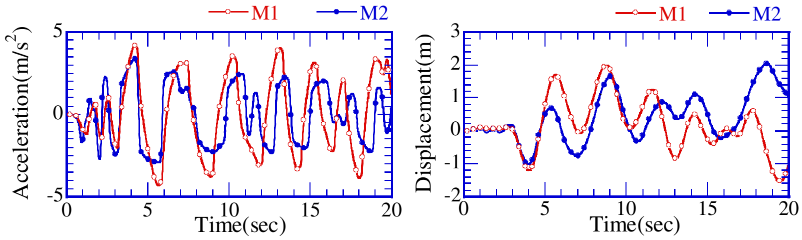



**Fig.9 Seismic responses at ground surface from M1 and M2 models (1-layer ground, elastoplastic)**

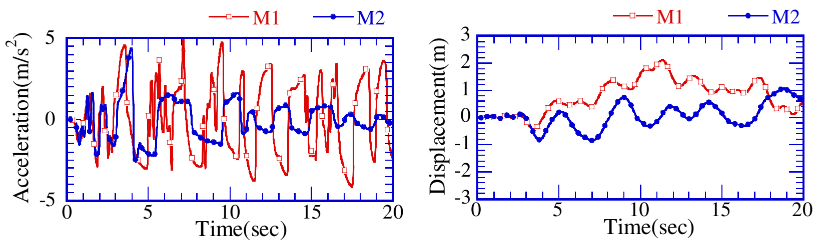



**Fig.10 Seismic responses at ground surface from M1 and M2 models (6-layer ground, elastoplastic)**

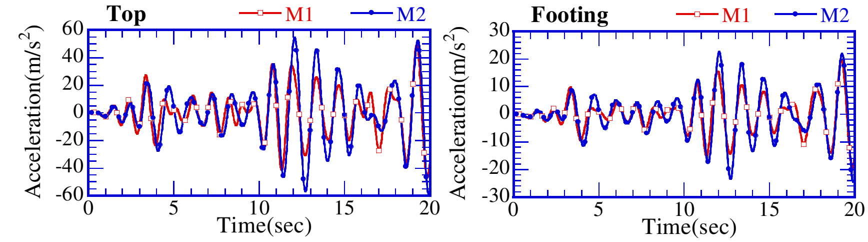
**Comparison of seismic responses of ground, pile foundation, superstructure using M1 and M2 models**

*Under elastic condition*

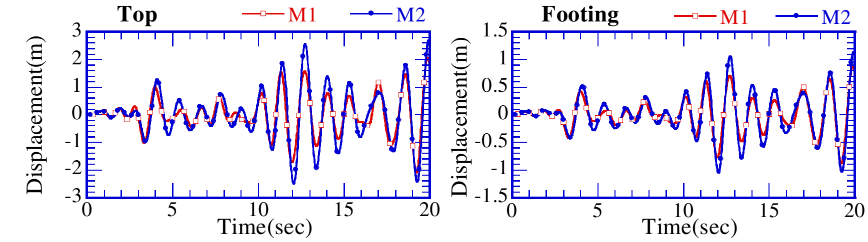
In the former research (Zhang et al., 2003), it was found that if the nonlinearity of the ground, the piles and the superstructure (full model) is fully considered, the results from the dynamic analyses of the ground-pile foundation-superstructure system using M1 and M2 models may be greatly affected by the modeling of M1 model. For this reason, as a first step, dynamic analyses on the full model with M1 and M2 models under elastic condition are conducted for two types of grounds, that is, 1-layer ground and 6-layer ground, in order to verify the applicability of M1 model under elastic condition. Four cases of calculations are conducted and are listed as below:

Case A-1: single pile foundation in 1-layer ground
Case A-2: single pile foundation in 6-layer ground
Case A-3: 9-pile foundation in 1-layer ground
Case A-4: 9-pile foundation in 6-layer ground

Figures 11-13 show the comparisons of the responses of accelerations and displacements at the top of pier and the footing, sectional forces of the pier and the single pile, and distribution of sectional forces of the single pile at the time when the maximum bending moment is reached, in Case A-1. Figures 14-16 show the same comparisons of the responses in Case A-2. In both cases, results from M1 and M2 models agree well with each other, with the only exception that the amplitudes of the responses in Case A-1 and Case A-2 are slightly different to each other.




(a) Accelerations of top and footing




(b) Displacements of top and footing

**Fig.11 Accelerations and displacements in Case A-1**

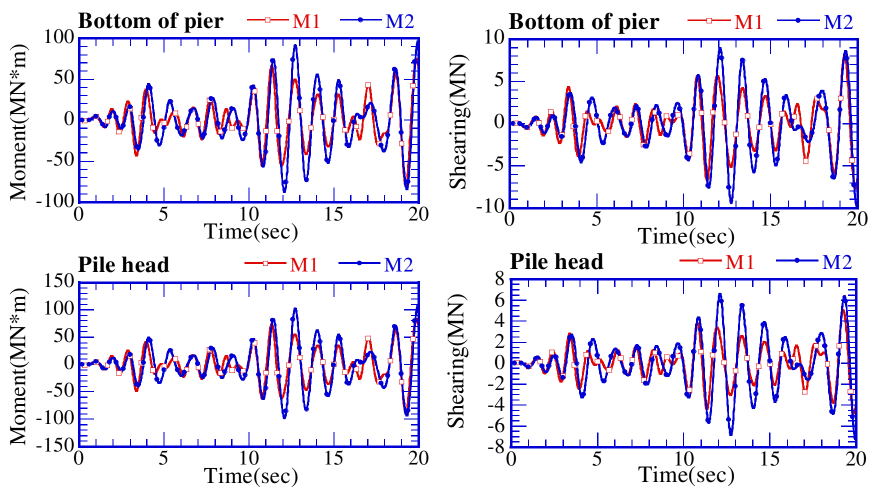



**Fig.12 Sectional forces of pier and single pile in Case A-1**

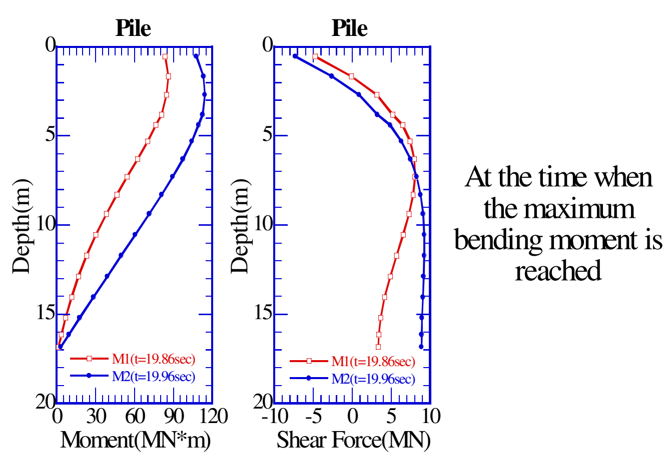
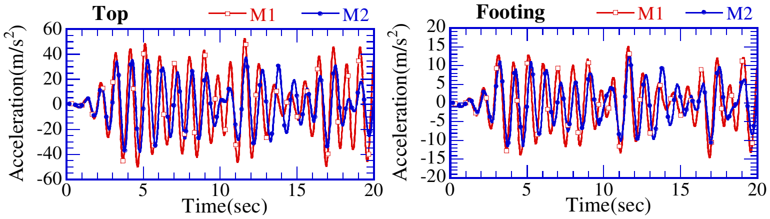
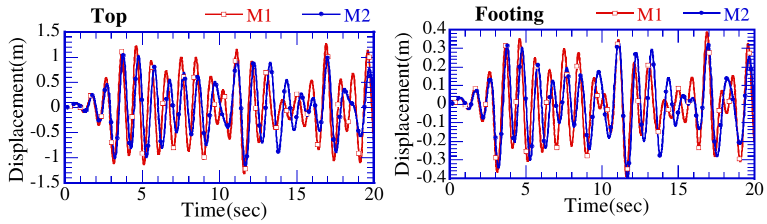



**Fig.13 Distribution of sectional forces of single pile in Case A-1**




(a) Accelerations of top and footing




(b) Displacements of top and footing

**Fig.14 Accelerations and displacements in Case A-2**

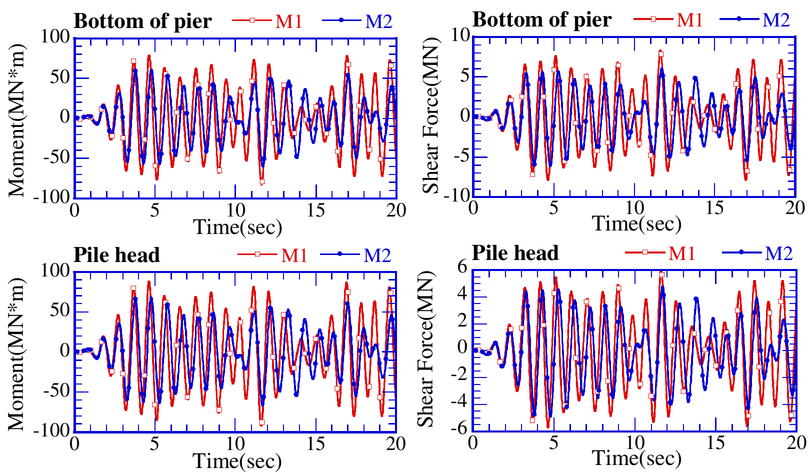
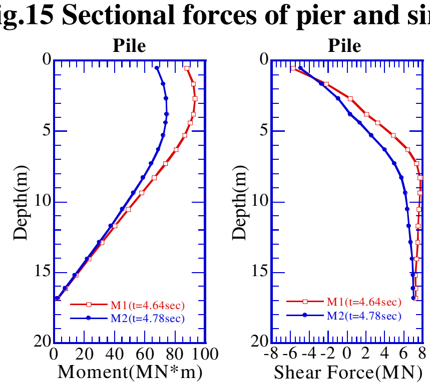


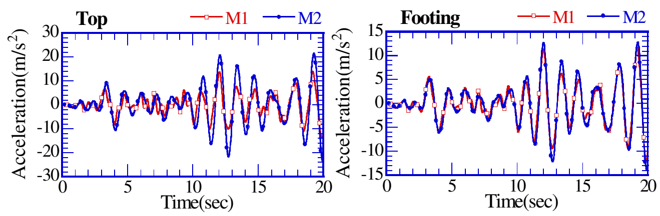

**Fig.15 Sectional forces of pier and single pile in Case A-2**



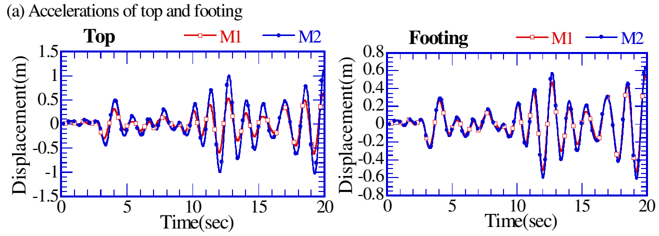

At the time when the maximum bending moment is reached

**Fig.16 Distribution of sectional forces of single pile in Case A-2**

Figures 17-20 show the comparisons of the responses of accelerations and displacements at the top of pier and the footing, sectional forces at the bottom of the pier, sectional forces of pile head at the group piles, and distribution of sectional forces of the group piles at the time when the maximum bending moment is reached, in Case A-3. Figures 21-24 show the same comparisons of the responses in Case A-4. In both cases, results from M1 and M2 models agree well with each other, with the only exception that the amplitudes of the responses in Case A-3 and Case A-4 are slightly different to each other. From both cases, it is found that the dynamic responses of footing obtained by M1 and M2 models are almost the same, but those of superstructure are different in amplitudes due to a discrepancy in the stiffness of footings in two models. That indicates that to obtain a correctly estimation of the response of superstructure, accurate modeling of the footing is very important.




(a) Accelerations of top and footing




(b) Displacements of top and footing

**Fig.17 Accelerations and displacements in Case A-3**

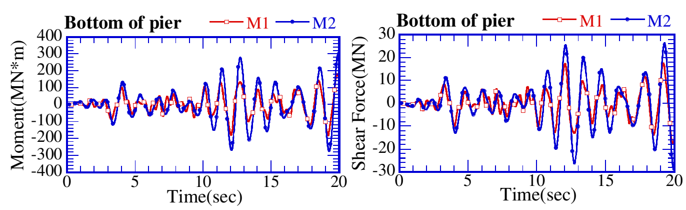



**Fig.18 Sectional forces at the bottom of pier in Case A-3**

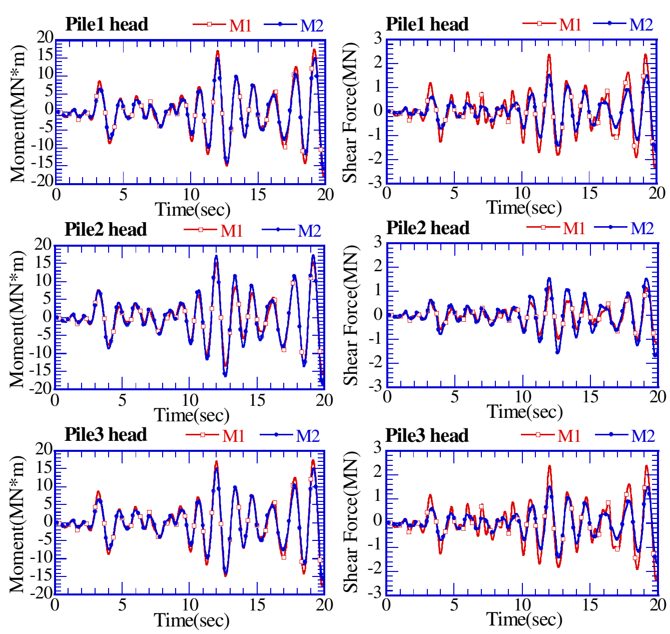
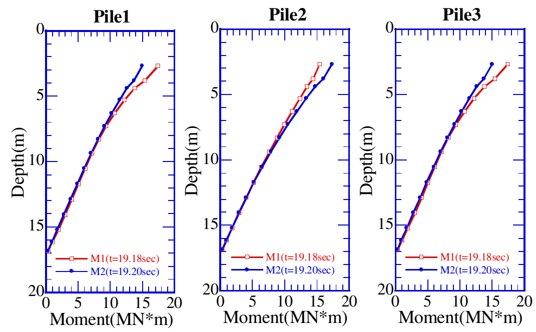
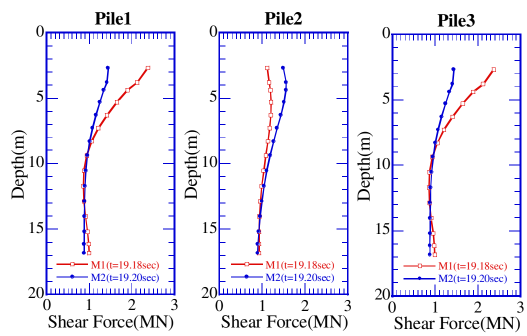



**Fig.19 Sectional forces at the heads of piles in Case A-3**

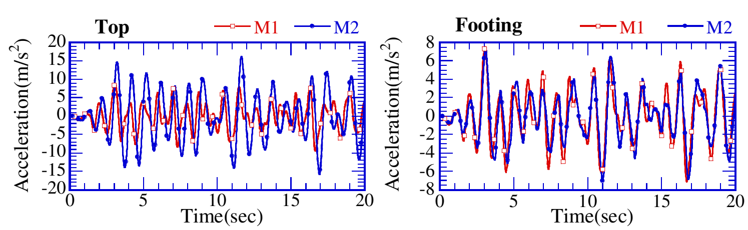



(a) Bending moment

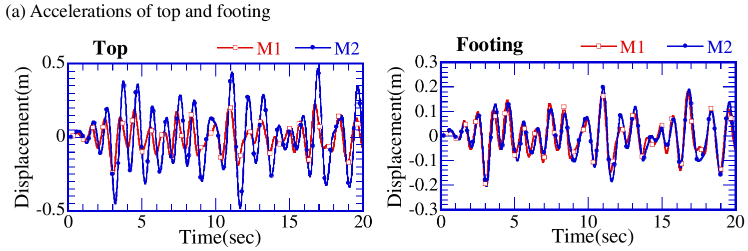



(b) Shear force

**Fig.20 Distribution of sectional forces of piles in Case A-3 (the moment when the maximum bending moment is reached)**




(a) Accelerations of top and footing




(b) Displacements of top and footing

**Fig.21 Acceleration and displacement in Case A-4**

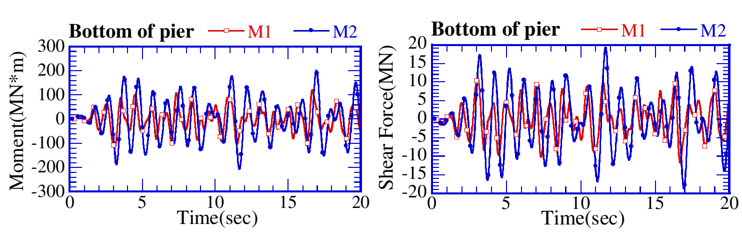



**Fig.22 Sectional forces at the bottom of pier in Case A-4**

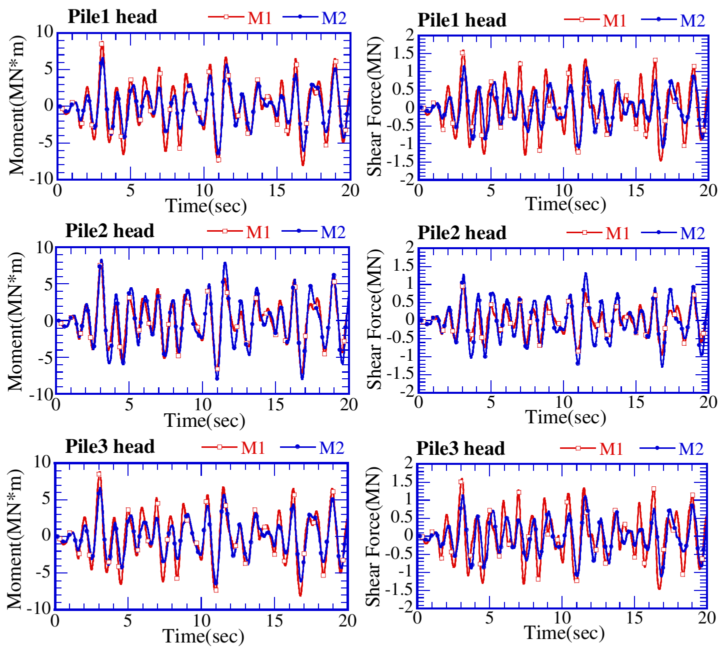
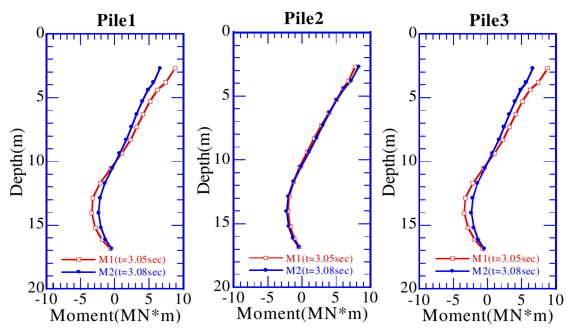
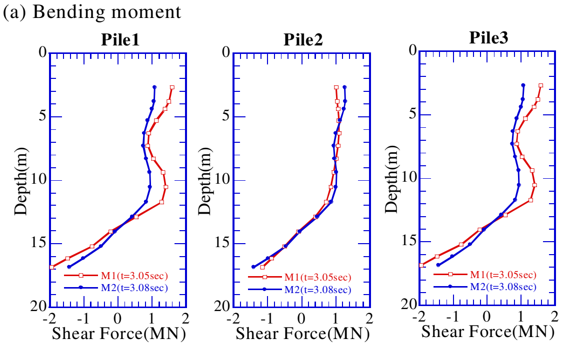



**Fig.23 Sectional forces at the heads of piles in Case A-4**




(a) Bending moment




(b) Shear force

**Fig.24 Distribution of sectional forces of piles in Case A-4 (the moment when the maximum bending moment is reached)**

*Under elastoplastic condition*

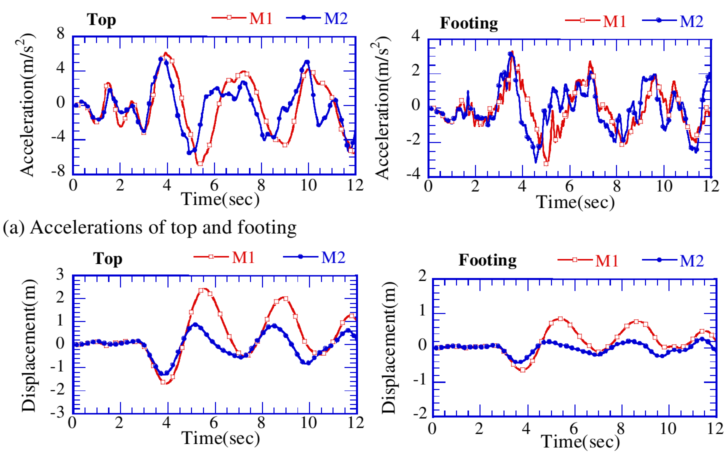
After confirming the validity of M1 model under elastic condition, further investigation is then extended to elastoplastic condition. The ground is simulated by springs whose force-displacement relation is described by R-O model, instead of trilinear model that was used in the former research (Zhang et al., 2003).

Furthermore, the analyses with M1 and M2 models under elastoplastic condition are performed on the single pile and 9-pile foundation in 6-layer ground. In the M1 model, R-O model is used for ground spring, and trilinear model is used for the springs that represent the interaction between ground and piles. In M2 model, tij models, which can well predict the stress-strain-dilatancy relation of soils in general stress condition, are used for soils. Axial force dependent model (AFD), (Zhang and Kimura, 2002), is used to simulate the nonlinear behavior of the piles in M1 and M2 models.

Two cases of calculations are conducted and are listed as below:

Case B-1: single pile foundation in 6-layer ground
Case B-2: 9-pile foundation in 6-layer ground

Figures 25-27 show the comparisons of the responses of accelerations and displacements at the top of pier and the footing, sectional forces of the pier and the single pile, and the distribution of sectional forces of the single pile at the time when the maximum bending moment is reached, in Case B-1. In the calculation, the main vibration of 12 seconds was calculated. It can be seen that results from M1 and M2 models agree well with each other in the 12 seconds, except a little discrepancy in the response displacements of top and footing, as shown in Figure 25.




(a) Accelerations of top and footing

(b) Displacements of top and footing

**Fig.25 Acceleration and displacement in Case B-1**

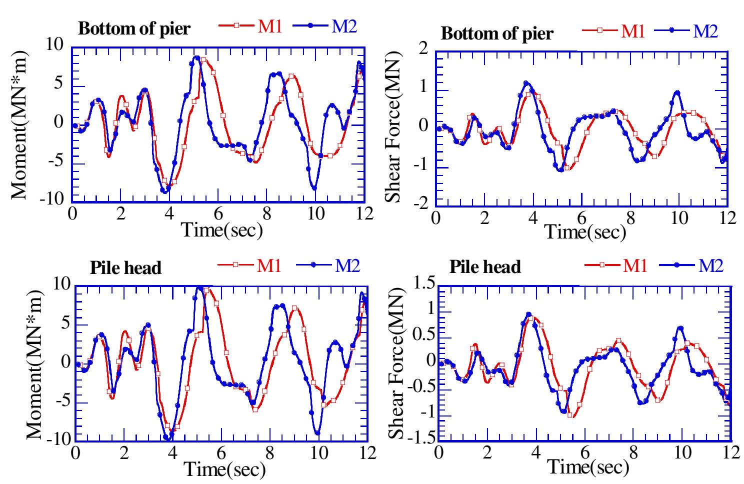



**Fig.26 Sectional forces at the heads of piles in Case B-1**

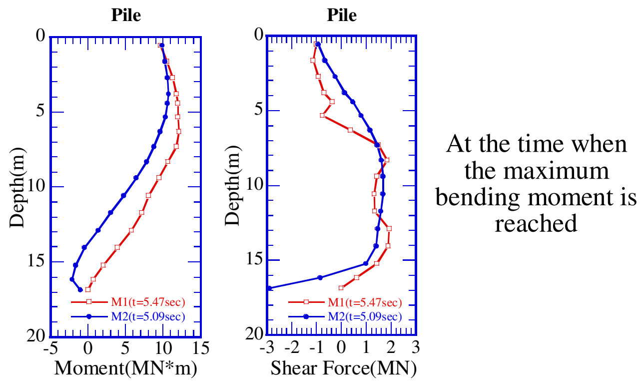
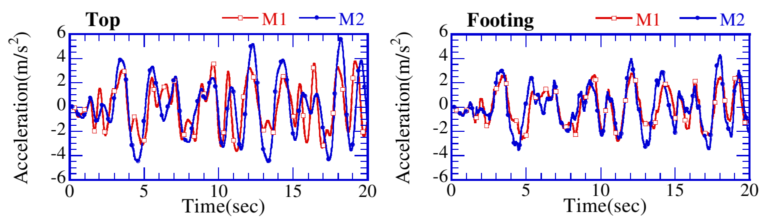
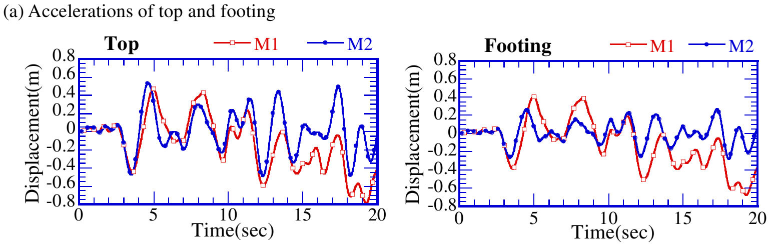



**Fig.27 Distribution of sectional forces of single pile in Case B-1**

Figures 28-31 show the comparisons of the responses of accelerations and displacements at the top of pier and the footing, sectional forces at the bottom of the pier, sectional forces of pile head at the group piles, and distribution of sectional forces of the group piles at the time when the maximum bending moment is reached, in Case B-2. The results from M1 and M2 models agree with each other to an acceptable degree in some aspects. For instance, the response accelerations predicted by two models are almost the same, not only in the amplitude, but also the response vibration phases, which are different in free ground calculation as shown in Figure 10. From Figure 30, however, it can be seen that a large difference in amplitude existed in the shear forces of Pile2 and Pile3. This implies that the evaluation of the equivalent horizontal springs connecting piles should be improved.




(a) Accelerations of top and footing




(b) Displacements of top and footing

**Fig.28 Acceleration and displacement in Case B-2**

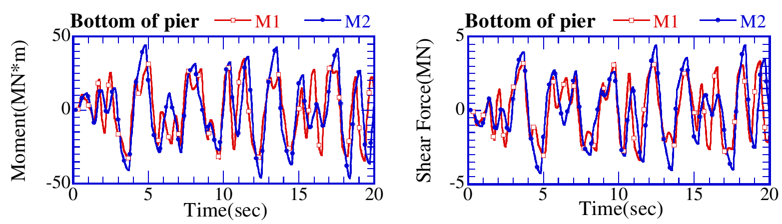



**Fig.29 Sectional forces at the bottom of pier Case B-2**

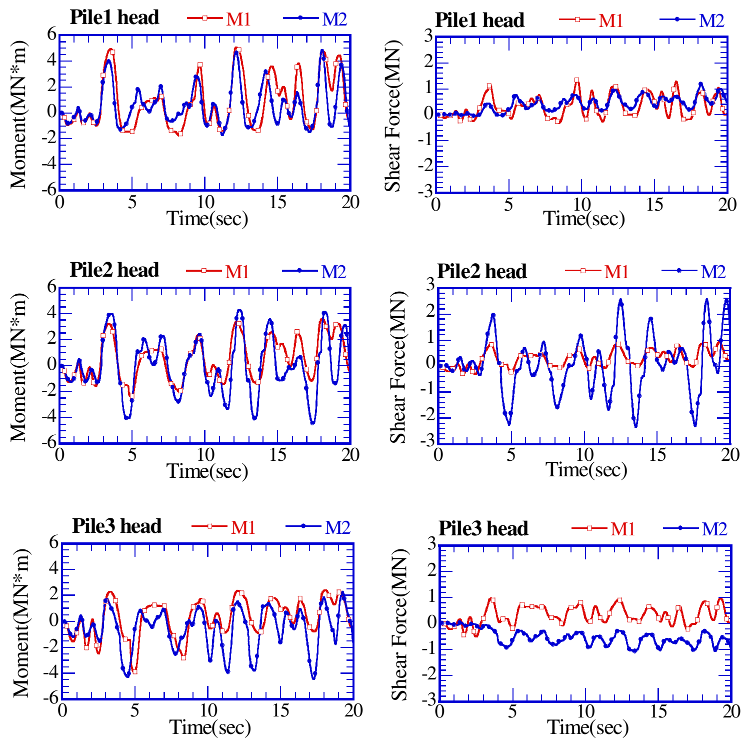
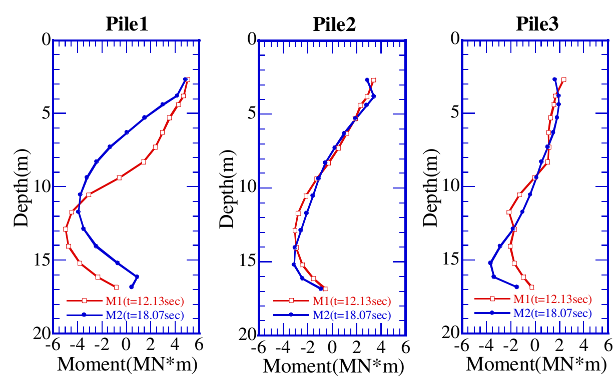
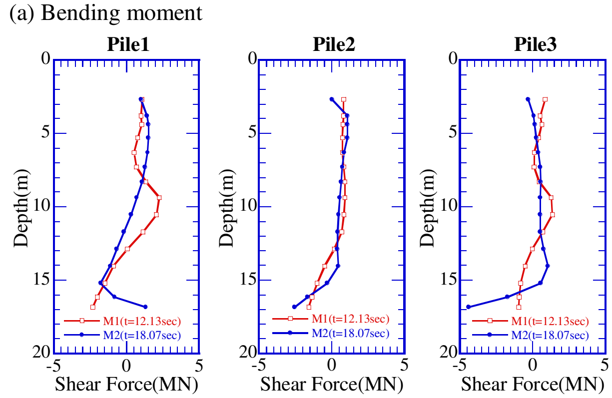



**Fig.30 Sectional forces at the heads of piles in Case B-2**




(a) Bending moment




(b) Shear force

**Fig.31 Distribution of sectional forces of piles in Case B-2 (the moment when the maximum bending moment is reached)**

## CONCLUSIONS

The following conclusions can be given:

Under elastic condition, the seismic behaviors of a free ground predicted by M1 and M2 models are the same. And the seismic behaviors of a full system containing a ground, a pile foundation and a superstructure, predicted by M1 and M2 models coincide well with each, only with a slight difference in the amplitudes of the responses, which indicates that M1 model is suitable for elastic seismic analysis with satisfactory accuracy.

Under elastoplastic condition, the seismic responses of the free ground obtained from M1 and M2 models can give similar predictions in the first several vibrations of the responses but then diverse in the subsequent vibration. And there is a relative large difference in their phases. Only the amplitudes of the response are in the same order.

As to the prediction of the seismic behaviors of a full system, by comparing to the results from M2 model, M1 model can also give a satisfactory result, only with a large discrepancy in the shear force of piles. Therefore, in the future study, the evaluation of the equivalent horizontal springs in M1 model should be improved so that it can be applied to ordinary seismic design.

## REFERENCES


1. Chowdhury, E. Q., Nakai, T., Tawada, M. and Yamada, S. 1999. A model for clay using modified stress under various loading conditions with the application of subloading concept, Soils and Foundations, Vol. 39, No.6, pp.103-116.
2. Design Codes of Foundations and Earth-Retaining Structures of Japan Railway, 1997, Japan Ministry of Transportation, ISBN 4-621-04315-3 C 3051, Maruzen Print Co. Ltd. (in Japanese).
3. Horikoshi, K., Ohtsu, H., Kimura, M. and Oka, F. 1996. Investigation of piles damages by the 1995 Hyogoken-Nambu Earthquake, Tsuchi-to-Kiso, Vol. 44, No.11, 27-29 (in Japanese).
4. Mori, S. 1997. Research on seismic technologies of pile foundation in liquefied ground, Doctoral Thesis, Kyoto University, Japan (in Japanese)
5. Mori, S., Miura, F., Zhang, F., Kishisita, T., Iwagami, N. and Sunazaka, Y., 2002. Seismic response analyses of pile-foundationed structure by various methods, Journal of Structural Engineering, Vol.48A, 457-468.
6. Nakai. T. 1989. An isotropic hardening elastoplastic model for sand considering the stress path dependency in three-dimensional stresses, Soils and Foundations, Vol. 29, No.1, 119-137.
7. Zhang, F. and Kimura, M. 2002. Numerical prediction of the dynamic behaviors of an RC group-pile foundation, Soils and Foundations, Vol. 42, No.3, 77-92.
8. Zhang, F., Yashima, A. and Noda, Y., 2003. Dynamic behavior of group piles with simplified model and full model, Geotechnical Engineering in Urban Construction, Proc. of the Sino-Japanese symposium on geotechnical engineering, Yu and Akagi (eds), Tsinghua University Press, 80-87.