DYNAMIC BEHAVIOR OF GROUP-PILE FOUNDATION BY THREE-DIMENSIONAL ELASTO-PLASTIC FINITE ELEMENT ANALYSES

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SUMMARY

It is known that in seismic evaluations of structures with pile foundations, because of the difficulty to conduct a three-dimensional nonlinear dynamic analysis on a full system, which consists of an upper structure, a foundation, and a ground, during a major earthquake, a separated method using a dynamic analysis on a simplified sway-rocking model (S-R model) and a responding displacement method are often used. In this paper, a finite element analytical code DGPILE-3D is developed for a static and dynamic three-dimensional elasto-plastic analysis on a full system. An elevated highway bridge with a group-pile foundation is investigated on a full scale. The nonlinear behavior of the ground and the structure is considered in the analysis. The soil is described by non-associated Drucker-Prager model and the piles and the pier are described by a tri-linear model considering hysteresis of moment-curvature relation. It is found that the nonlinearity of both the soil and the structures may greatly affect the mechanical behavior of the pile foundation subjected to cyclic lateral loading (earthquake loading) up to the ultimate state.

INTRODUCTION

After the Hyogoken-Nambu earthquake, the designing codes for highway and railway bridges have been revised to meet the needs of much higher levels of strength for structures (Design Codes for Foundations and Earth-Retaining Structures of Japan Railway, 1997; Design Codes of Japan Highway Bridge, 1997). In the revised railway code, a nonlinear dynamic analysis is strongly recommended for the design of bridges. It is commonly known, however, that the seismic behavior of a bridge is not only related to the upper structure of the bridge, but also to the foundation of the bridge and to the ground upon which the structure is built. It is recommended, therefore, that a full system, which consists of an upper structure, a foundation and a ground, be considered in the dynamic analysis. Dealing with the full system in a dynamic analysis is usually thought to be difficult when the nonlinearity of both the structure and the ground must be considered. Many studies have been done in this field through both experiments and numerical analyses. Taji et al. (1997) conducted a large-scale shaking-table test and a centrifuge model test to investigate the soil-pile-structure interaction in potentially liquefying sand. Fukutake (1997) conducted a seismic evaluation of a group-pile foundation surrounded by a frame wall using a 3-D nonlinear dynamic analysis. Taguchi et al. (1997) conducted a numerical simulation of soil-foundation interaction with a 3-D nonlinear dynamic analysis considering subsoil liquefaction. By developing the three-dimensional static and dynamic finite element analytical code DGPILE-3D (Kimura and Zhang, 1999), the authors try to discover whether or not it is possible and/or practical to conduct a three-dimensional nonlinear dynamic analysis on a structure-foundation-ground system.

In seismic evaluations of structures with pile foundations, two methods, that is, the separated method using a dynamic analysis on an S-R model and the responding displacement method, are often used. The advantage of the two methods is that the finite element analyses related to a ground can be conducted statically, which saves a lot of time. The disadvantage of the methods is that it is very difficult to evaluate the phase difference between the inertial force from an upper structure and the deformation of a ground during an earthquake. Furthermore, the interaction between the pile foundation and the ground is evaluated with static analysis, therefore, the question

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whether the dynamic interaction between the pile foundation and the ground can be properly described by the static analysis remains unresolved. A dynamic analysis on a full system is needed to solve this problem.

When a seismic evaluations of structures with pile foundations is conducted under the condition of a major earthquake, the most important point is to properly evaluate the nonlinear behavior of the ground and the structure, which may greatly affect the mechanical behavior of the pile foundation subjected to cyclic lateral loading up to the ultimate state. In this paper, a three-dimensional dynamic elasto-plastic finite element analysis (DGPILE-3D) is conducted. The main purpose is to provide an applicable numerical way to conduct the seismic evaluation of a pile foundation that can be used rather easily for highway and railway bridges. It should be pointed out that for simplicity, the numerical analysis in this paper is conducted under a total stress condition, which means that DGPILE-3D is not suitable to a potentially liquefying ground. For a potentially liquefying ground, a water-soil coupling analysis is necessary.

NONLINEAR BEHAVIOR OF GROUND AND PILES

Kimura et al. (1991) developed a three-dimensional finite element analysis program, GPILE-3D, in which the stress-strain relation of the ground is supposed to be elasto-plastic and an associated flow rule of Drucker-Prager's yielding criteria is adopted. Various improvements were later added to the program (Adachi et al., 1994; Kimura and Zhang, 1997).

2.1 nonlinearity of ground

It is known that in the case of Drucker-Prager's associated flow rule, the plastic potential is the same as the yielding function. Therefore, the plastic volumetric strain will always be negative, which means that swelling will always occur during shearing. The flow rule of Drucker-Prager model is

$$d\varepsilon_{ij}^{p} = \Lambda(\partial f_p / \partial \sigma_{ij}) df_y \tag{1}$$

where f_y is the yielding function, f_p is the plastic potential and Λ is a positive parameter. The yielding function is expressed as follows:

$$f_y = (J_2)^{1/2} - 3\alpha\sigma_m - \kappa_s = 0 \tag{2}$$

where J_2 is the second invariant of the deviatoric stress tensor and σ_m is the mean stress. α and κ_s are material constants which can be determined from ϕ , the internal frictional angle of the soil, and c, the cohesion of the soil, as

$$\alpha = 2\sin\phi / (3+\sin\phi)/\sqrt{3}\ ,\ \kappa_s = 6c\cos\phi / (3+\sin\phi)/\sqrt{3}\ . \tag{3}$$

The plastic potential in the theory is expressed as

$$f_p = (J_2)^{1/2} - 3\beta\sigma_m - \kappa_s = 0 \tag{4}$$

where β is the dilatancy parameter which can be computed with Equation 3 by substituting the frictional angle with dilatant angle β. If $\beta = 0$, it means no dilatancy. If $\beta = \alpha$, it means an associated flow rule. The loading and unloading conditions are expressed as

$$\begin{aligned} &f_y = 0,\ df_y > 0\text{: loading} \\ &f_y = 0,\ df_y = 0\text{: neutral} \\ &f_y = 0,\ df_y < 0\text{: unloading.} \end{aligned} \tag{5}$$

Figure 1 shows a theoretical stress-strain relation in a hollow cylindrical torsional test on sand. It is based on the associated and the non-associated flow rules of Drucker-Prager model, on the condition that the confining and axial pressures are kept constant. The sample is subjected to two and a half cycles of torsional shearing and the stress-strain relation remains the same during each cycle for different β values. The plastic volumetric strain is different, however, if β takes a different value. The larger the β value is, the larger the plastic volumetric strain will be. If the hollow cylindrical test is conducted under a condition in which the volumetric strain is restricted to zero, it is clear from Figure 2 that in the case of the associated flow rule, the shear strength increases during each loading cycle because of the increase in mean stress. In the case of the non-associated flow rule (β=0), the shear strength is kept constant.

It is commonly known that a swelling dilatancy does not usually occur in a soft clayey ground. For this reason, when a boundary value problem related to a pile foundation in soft ground is considered, the condition that β is equal to zero is preferred. The advantage of the model is that it is very simple and only four parameters need to be determined. In geotechnical engineering, Japanese engineers prefer to determine these parameters with N, the value of the standard penetration test (SPT), instead of with the laboratory tests. In this case, the following empirical formulae are recommended (Design Codes for Foundations and Earth-Retaining Structures of Japan Railway, 1997):

$$c = q_u / 2 = 1/160N \quad (\text{MPa}) \text{ for clay (Terzaghi - Peck)} \tag{6}$$

$$\phi = 1.85(N/(\sigma_v^{'}/\sigma_0^{'} + 0.7))^{0.6} + 26 \quad (°) \quad \text{for sand}, \sigma_0^{'} = 0.1\,\text{MPa} \tag{7}$$

$$E = 1.4 \text{ to } 2.8\, N \text{ (MPa)} \tag{8}$$

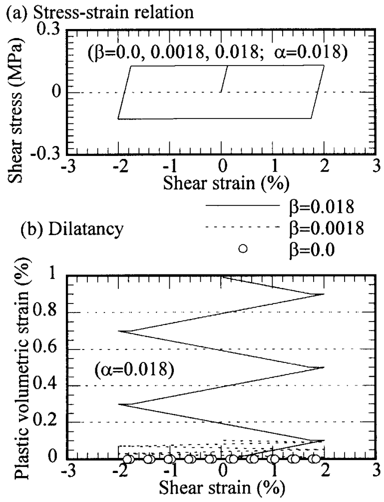


Fig. 1 Stress-strain and stress-dilatancy relations in hollow cylindrical test with Drucker-Prager model

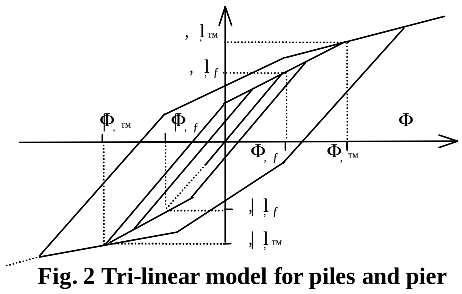
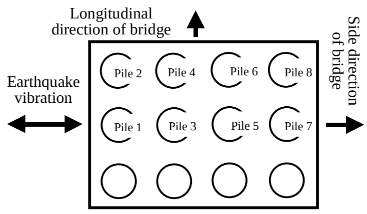
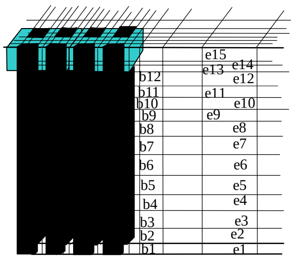


Fig. 2 Tri-linear model for piles and pier

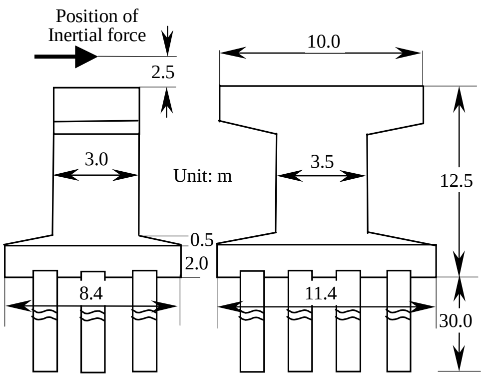


(a) Plane view of piles and footing



(b) Position of the elements under consideration

Fig. 3 Geometry of the elevated highway bridge



(a) Longitudinal direction of bridge

(b) Side direction of bridge

Fig.4 Setup of group-pile foundation

Table 1 Material properties of ground

Soil	N	Thickness (m)	γ (kN/m³)	E (MPa)	ν	K_0	c (MPa)	φ (
B	4	4	18.0	10	0.30	0.50	0.004	30
Ac1	2	10	17.0	5	0.42	0.80	0.0125	5
Ac2	8	14	18.0	20	0.40	0.67	0.050	5
As1	16	4	19.0	40	0.30	0.50	0.00	35
Dg	50	2	20.0	125	0.30	0.50	elastic	elastic

Table 2 Nonlinearity of piles

Position (m)	M_c (MN*m)	ϕ_c (1/cm)	M_y (MN*m)	ϕ_y (1/cm)	M_u (MN*m)	ϕ_u (1/cm)
0.0~10.0	0.545	0.0186	1.588	0.2126	2.476	2.639
10.0~30.0	0.509	0.0186	0.867	0.1914	1.319	2.122

$E_{concrete}$=2.87 ×10⁴ (MPa)

Table 3 Nonlinearity of pier

M_c (MN*m)	ϕ_c (1/cm)	M_y (MN*m)	ϕ_y (1/cm)	M_u (MN*m)	ϕ_u (1/cm)
30.13	0.00934	79.84	0.0785	103.37	0.681

$E_{concrete}$ =3.00 ×10⁴(MPa)

2.2 NONLINEARITY OF PILES

In DGPILE-3D, cyclic loadings should be considered, therefore, it is difficult to give a proper description of the influence of axial force on M-Φ relation. For this reason, the M-Φ relation of pile is simulated here by a trilinear relation, taking into consideration the hysteresis of loading and unloading, as shown in Figure 2. While the influence of axial force on M-Φ relation, is not considered In the relation, M_c, M_y, and M_u represent the cracking, the yielding and the ultimate moments, while Φ_c, Φ_y, and Φ_u represent the corresponding curvatures. These values can be easily determined based on the strength of the concrete and reinforcements. In simulating a pile, the usual way in the finite element analysis is to use a beam element. It is known that a beam element does not have volume. In the finite element analysis of a pile group, the pile-volume influence cannot be neglected. In particular, when the diameter of a cast-in-place reinforced concrete pile is large, the neglecting of the pile-volume due to the introduction of beam element may greatly affect the behavior of the piles. For this reason, a hybrid element that consists of a beam element and a few solid elements is introduced to simulate the pile. By introducing the hybrid element, the pile-volume influence can be properly considered.

DYNAMIC SIMULATION OF AN ELEVATED BRIDGE WITH A GROUP-PILE FOUNDATION

Based on the above discussion, an elevated highway bridge with a group-pile foundation is considered in a seismic evaluation. The bridge is supported by a group-pile foundation made of 3×4 cast-in-place reinforced concrete piles, 1.2 meters in diameter (D) and 30 meters in length, as shown in Figure 3. The distance between the centers of the two piles is 2.5 D. The ground is composed of five layers. The surface layer of the ground is a sand layer, followed by a very soft alluvial clayed layer, 10 meter in thickness, with a small *N* value for SPT. The third layer is also alluvial clayed soil and fourth layer is an alluvial sandy soil. The pile group is laid on fifth layer, a diluvial gravel. The bottom layer of the ground is supposed to be an elastic material in the numerical analysis. The material properties of the grounds are listed in Table 1. The material properties of the piles and the pier are shown in Tables 2 and 3. Figure 4 shows the setup of the group-pile foundation.

A dynamic analysis of the elevated highway bridge using a full system is conducted to simulate the mechanical behavior in earthquake vibration. A direct integration method of is adopted. The finite element mesh for the ground is shown in Figure 5. Figure 6 shows the input earthquake wave used in the analysis, which is an artificial earthquake wave specified for the seismic design of railway bridges in Japan. It has a maximum acceleration of 722 gal in a horizontal direction. A Rayleigh type of attenuation is adopted and the attenuation constants of the structures and the ground are assumed as 5% and 10%, respectively, in the dynamic analysis of a full system. Although the stiffness of the ground, the piles, and the pier may change because of the nonlinearity of these materials, the viscous matrix calculated from the Rayleigh type of attenuation is assumed to be constant in spite of the changes in the stiffness matrix. In order to calculate the viscous matrix, an eigenvalue analysis for the full system is conducted to evaluate the first two eigenvalues. The eigenvalue analysis shows that the first two eigen periods are 2.361 and 1.663 sec., respectively. The eigenvalue analysis is conducted with a hybrid of Jacobian and subspace methods. In the three-dimensional finite element dynamic analysis, the time interval of the integration is 0.01 sec. In order to investigate the influence of the nonlinearity of the ground and the structure, two kinds of analyses, that is, elastic analysis and plastic analysis are conducted.

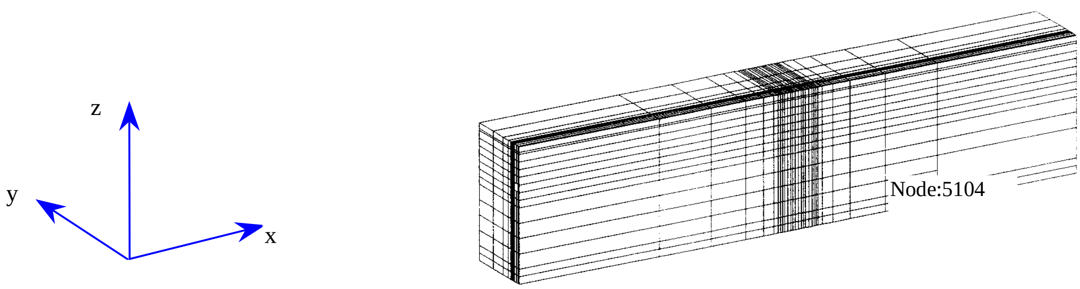


Fig.5 Finite element mesh

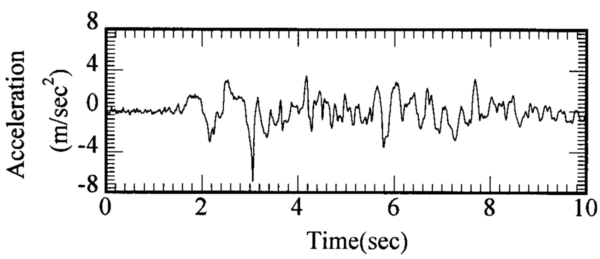


Fig.6 Input wave

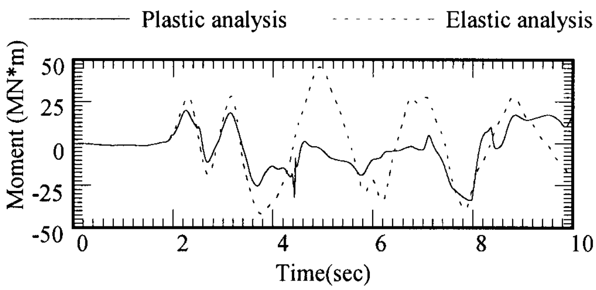
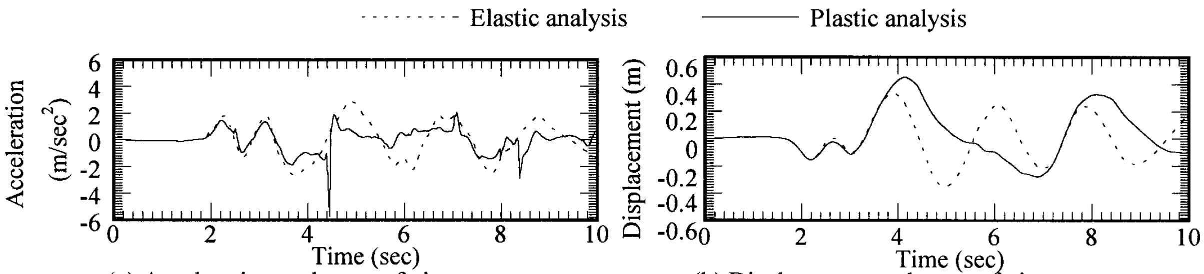


Fig.7 Time history of bending moment at the



(a) Acceleration at the top of pier (b) Displacement at the top of pier

Fig.8 Comparison of responding displacement and acceleration at the top of pier

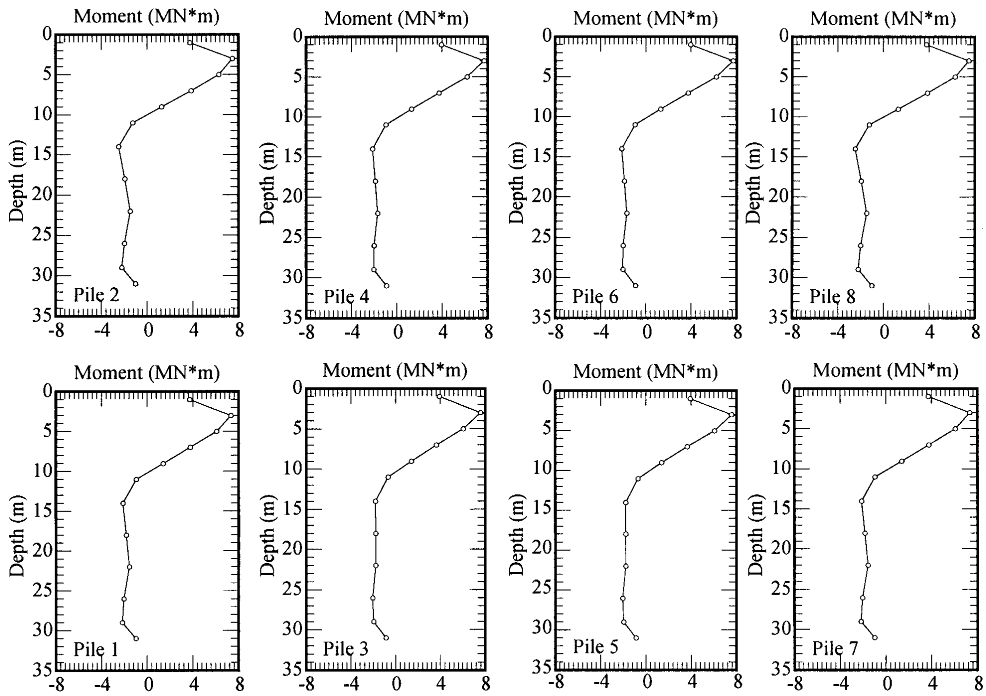


Fig.9 Distribution of the moment in the piles (Elastic analysis, t=3.90)

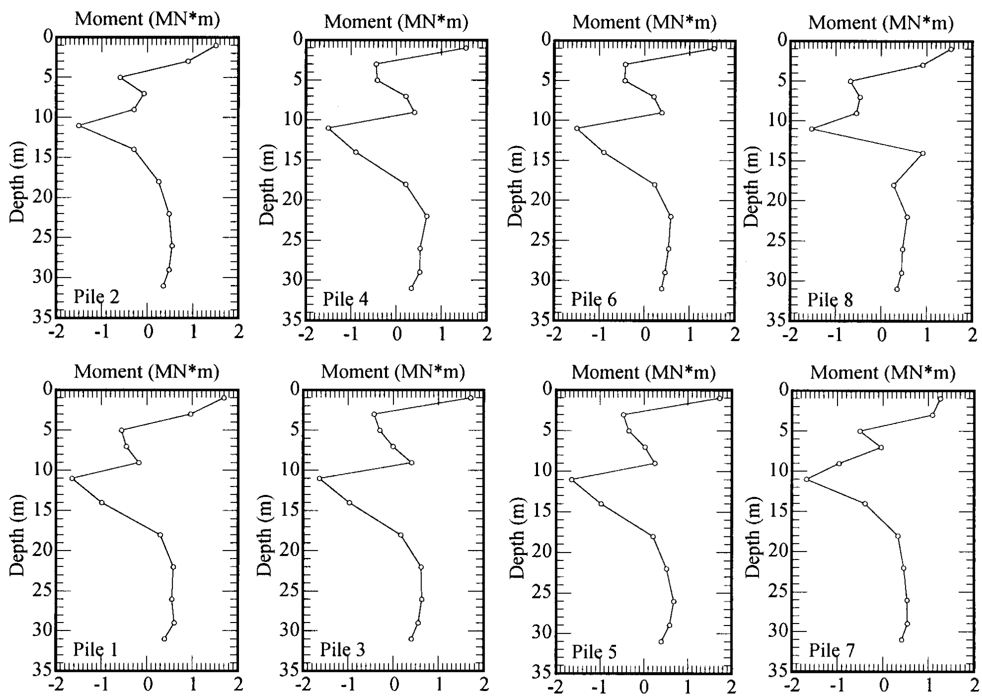


Fig.10 Distribution of the moment in the piles (Plastic analysis, t=4.40)

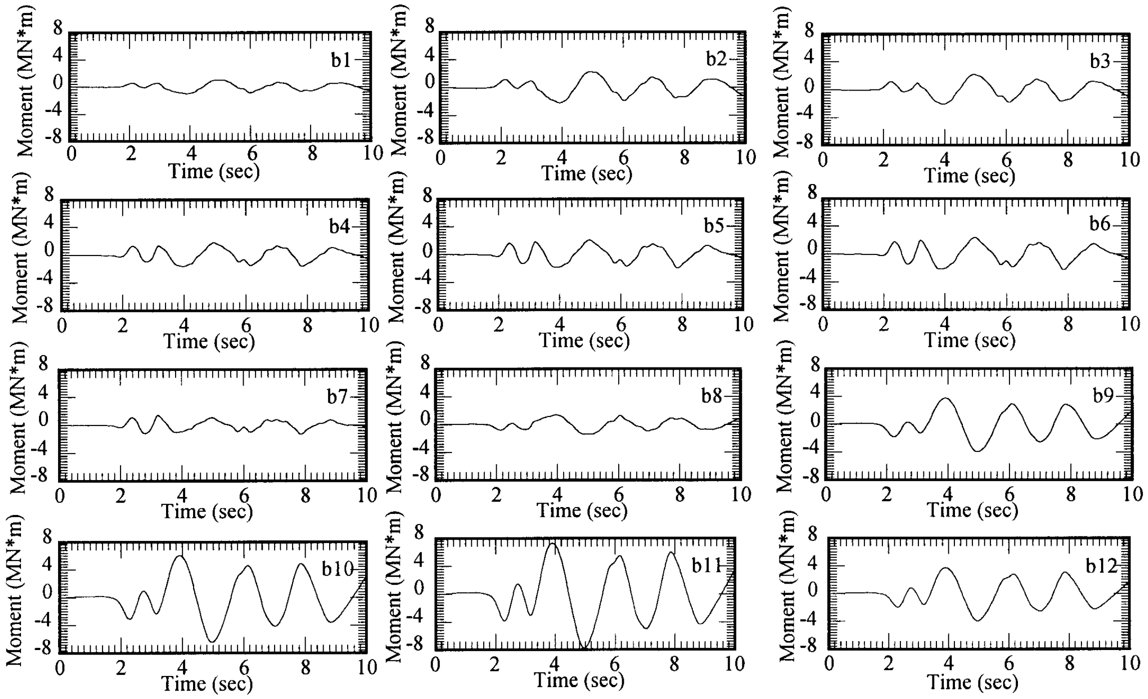


Fig.11 Time history of the moments in the piles (Elastic analysis)

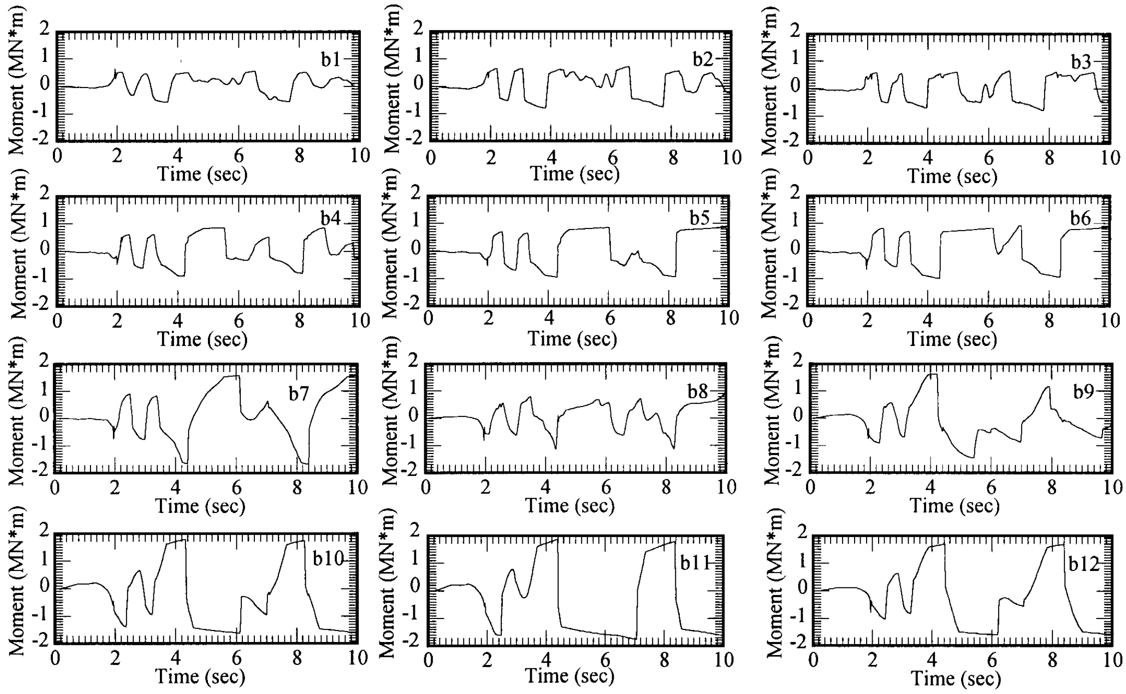


Fig.12 Time history of the moments in the piles (Plastic analysis)

Figure 7 shows the time responses of the bending moment at the bottom of the pier obtained from the elastic and the plastic analyses. It is known form the figure that the maximum bending moments obtained from the two analyses exceeded cracking moment M_c, and the plastic one is much less than elastic one but with a much longer period.

Figure 8 shows the comparison of the responding displacement and acceleration at the top of pier obtained from the elastic and the plastic analyses. The responding acceleration does not change too much in amplitude for different analyses but the eigen period becomes longer in the plastic analysis, changing from 2.3 sec. to about 4.0 sec. The same phenomenon can be observed in the responding displacement. This is thought to be the reason that in the plastic analysis, if the ground reaches the failure line, the stiffness of the ground will decrease greatly, resulting in the elongation of the eigen period.

Figure 9 and 10 show the distribution of the bending moments in the piles obtained from the elastic and the plastic analyses, respectively. The distribution is plotted at the time of 3.90 sec. in elastic analysis and 4.40 sec. in plastic analysis, at the times when the maximum bending moments occur. The maximum moment in elastic analysis is about 7.80 MN*m, being four times as large as the one obtained from the plastic analysis. Unlike the elastic analysis in which the distributions of the moments in piles are almost the same, different distributions of the moments in piles are observed. A dramatic change in the distributions of the moments of the piles, which occurs at the boundary of the two different soil layers, is also observed. The maximum moment occurs at the place near the ground surface in the elastic analysis while it occurs at the boundary of the two different soil layers in the plastic analysis. The maximum moments do not exceed the ultimate moment M_u in the plastic analysis. From these figures, it is clear that the nonlinearity of the ground and the structures may greatly affect the mechanical behavior of piles.

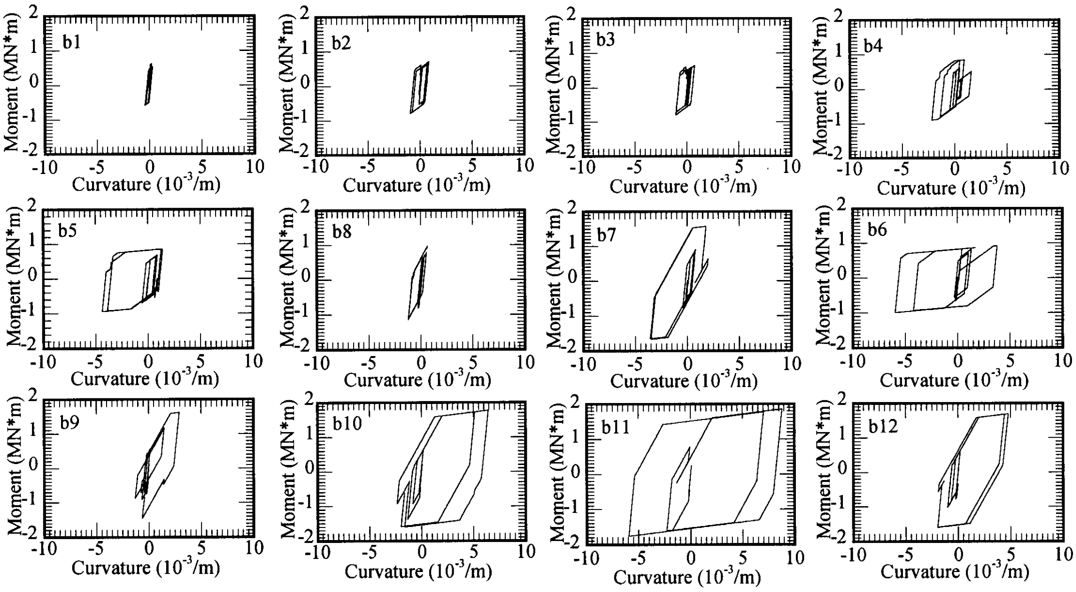


Fig.13 Hysteresis of moment-curvature relation obtained from plastic analysis

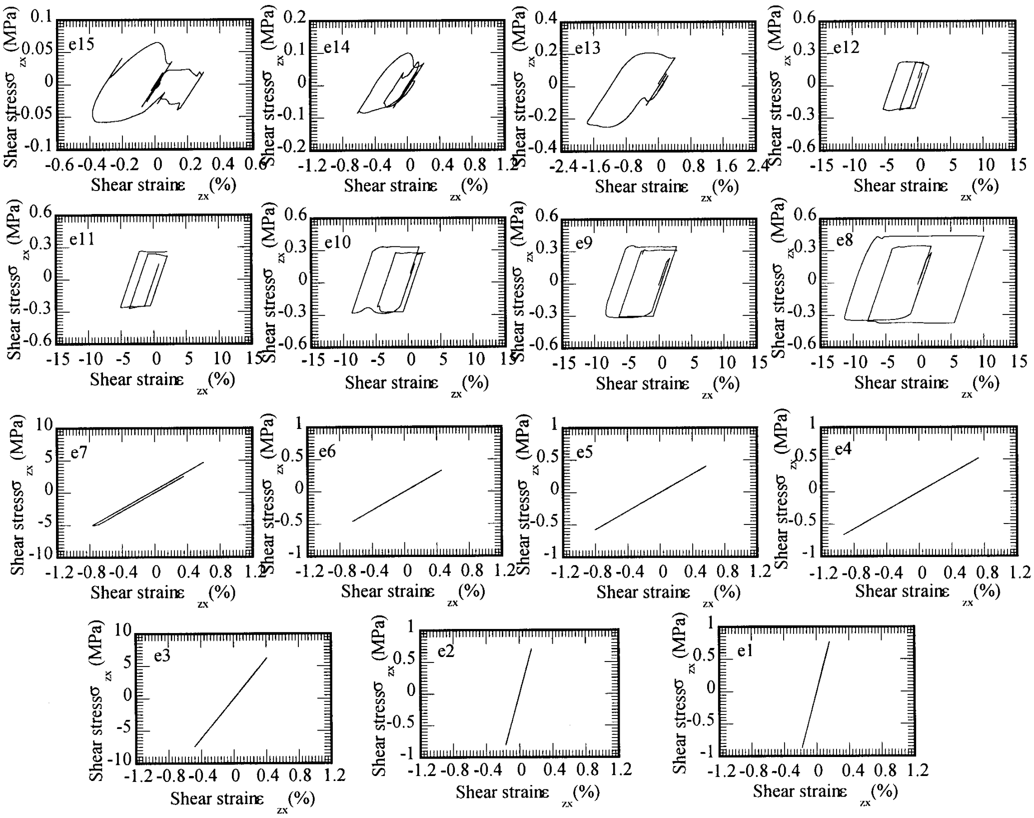


Fig.14 Stress-strain relation of soils obtained from plastic analysis

Figure 11 and 12 show the responding time histories of the moments in the piles, whose positions are shown in Figure 4, obtained from the elastic and the plastic analyses, respectively. It is clear from the figures that due to the yielding of the ground and the structures, the eigen periods of the responding moments at all position change from 2.3 sec. to about 4.0 sec., as is the same phenomenon observed in the responding displacement.

Figure 13 shows the hysteresis of the moment-curvature relations of the piles obtained from the plastic analysis. A big hysteresis loop can be observed when the maximum bending moment exceeds the yielding moment of the pile. The biggest loops are found occurred at the places near the ground surface and the boundary between two different soil layers.

Figure 14 shows the hysteresis of the shear stress-strain relations at different soil layers. In the surface layer (e15, e14, e13), because of the small confining stress, the hysteresis loop shows a round locus, which means that the stress-strain relations are deeply affected by the change of the confining stresses. In the second layer, which is a very soft clayed layer (e12, e11, e10, e9, and e8), the stress-strain relations show a typical elasto-perfect plastic behavior. Particularly in e8, which is the boundary between two different soil layers, a large shear strain of 12% occurred, resulting in a dramatic increase in the horizontal displacement of the ground above the boundary.

CONCLUSIONS

1. During a major earthquake, the influence of the nonlinearity of the ground and the structures (piles and pier) may greatly affect the mechanical behavior of the pile foundation. The moment in the pile caused by the deformation of a ground, particularly at the boundary of two different soil layers, may be larger than the moment caused by the inertial force from the upper structure. Based on the results obtained in this paper, it is worth emphasizing that the influence of the deformation of a ground on the piles must be considered carefully.

2. Elastic analysis cannot describe the difference of the distribution of moment in different piles, even in a three-dimensional dynamic finite element analysis. Proper evaluation of the dynamic interaction between the pile foundation and the ground should be considered in plastic condition.
3. Numerical calculation conducted in this paper is stable even if the time interval used in the direct integration method is not so short. The constitutive model of the ground is also simple and only four parameters are needed. These parameters can be determined with the N value of SPT, which is quite familiar to engineers. On the other hand, because the mechanical behavior of soil is complicated, the four-parameter constitutive model adopted in this paper has its limitation. The analytical result is sensitive to the values of the parameters in the model. Thus great attention should be paid to the determination of the parameters, especially the values of c and ϕ.

REFERENCES

1. Adachi, T., Kimura, M. and Zhang, F. 1994. Analyses on ultimate behavior of lateral loading cast-in-place concrete piles by three-dimensional elasto-plastic FEM, Proc. of 8th Int. Conf. Computer Method and Advance in Geomechanics. Vol. 3. pp. 2279-2284. Morgantown: Balkema.
2. Fukutake, K., 1997. Seismic evaluation of a group-pile foundation surrounded by a frame wall using a 3-D nonlinear dynamic analysis, 24th Symposium of Japan Earthquake Engineering, pp. 673-676 (in Japanese).
3. Kimura, M., Yashima, A. and Shibata, T. 1991. Three-dimensional finite element analysis of laterally loaded piles, Proc. of 7th Int. Conf. Computer Method and Advance in Geomechanics, pp. 145-150. Cairns: Balkema.
4. Kimura, M. and Zhang, F. 1997. Seismic evaluation of pile foundation, Proc. 6th Int. Symp. Numerical Models in Geomechanics, pp. 545-548. Montreal: Balkema.
5. Kimura, M. and Zhang, F. 1999. Seismic evaluation of pile foundation by static and dynamic 3-D finite element analyses, Asian Regional Conference on Geotechnical Engineering, Seoul, to be published.
6. Taji, Y., Sato, M., Kagawa, T. and Minowa, C., 1997. Investigation on Soil-pile-Structure interaction in liquefying sand by a large scale shaking-table test and centrifuge test, 24th Symposium of Japan Earthquake Engineering, pp. 277-280 (in Japanese).
7. Taguchi, Y., Tateishi, A., Oka, F., and Yashima, A., 1997. Numerical simulation of soil-foundation interaction behavior during subsoil liquefaction, Proc. 6th Int. Symp. Numerical Models in Geomechanics, pp. 561-564, Montreal: Balkema.
8. Design Codes of Foundations and Earth-Retaining Structures of Japan Railway, 1997, ISBN 4-621-04315-3 C 3051, Maruzen Print Co. Ltd., printed in Japan (in Japanese).

Design Codes of Japan Highway Bridge, Vol. V, Seismic Designing, 1996, ISBN 4-88950-234-3, Maruzen Print Co. Ltd., printed in Japan (in Japanese).