軸力変動を考慮できるAFDモデルを用いた杭基礎・地盤・上部工一体系弾塑性動的解析

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ABSTRACT: 一般に、水平荷重を受ける杭の挙動解析を行う際、大変形時には杭基礎の塑性化と地盤の降伏が予想され、この非線形挙動を適切に評価しなければならない。しかしこれまで、杭の軸力変動が非線形特性に及ぼす影響の適切な評価が難問視されてきた。本研究では、群杭の終局挙動を表現しうる3次元弾塑性静的・動的有限要素の解析コード(DGPILE-3D)に、新たに軸力変動による影響を適切に表現できるAFD (Axial Force Dependent)モデルを用いた解析手法を開発した。また、開発された解析手法を用いて、12群杭を有する杭基礎・地盤・上部工一体系弾塑性動的解析を実施し、その解析手法の妥当性を検証する。

Key Words: *finite element method, dynamic analysis, group-pile foundation, axial-force dependency, elasto-plasticity, RC pile*

1 INTRODUCTION

It is known that during a strong earthquake, the dynamic behavior of a group-pile foundation is not only related to the inertial force come from the superstructures but also to the deformation of the surrounding ground. During the Hyogoken-Nanbu 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.

On the other hand, dealing with a full system, which consists of superstructures, a foundation and a ground, in a numerical dynamic analysis in time domain is usually thought to be effective and executable nowadays when the nonlinearity of the superstructure, the piles and the soils is considered. A few studies have been done in this field through both experiments and numerical analyses. Murono et al. (1997) conducted a dynamic model test on a structure-foundation-ground system to investigate the seismic behavior of group-pile foundation.

Zhang et al. (2000) simulated numerically a field test of a real-scale 9-pile foundation subjected to lateral cyclic loading up to ultimate state with a 3-D elastoplastic finite element analysis, considering the influence of different constitutive models adopted for soils. Kimura and Zhang (2000) conducted a series of static and dynamic 3-D elastoplastic finite element analyses on a simplified sway-rocking model (S-R model) and on a full system to investigate the dynamic behavior of group-pile foundation during earthquake. It is, however, well known that the bending strength and the load-sharing ratio of the piles in a pile group are totally different in different position, e.g. front, back and middle position, when subjected to lateral loadings. In aforementioned works, a very important fact, that is, the influence of axial force on the stiffness and the bending strength of RC piles that greatly affects the nonlinearity of piles, is neglected. The reason why the influence of axial force was not considered is that it is difficult to model the influence under cyclic loading condition within the framework of common beam theory.

In structural engineering, a few axial-force dependent nonlinear models for RC material have been proposed. Among them, the multi-spring model and the fiber model (Lai et al., 1984; Li and Kubo, 1999) are rather popular. The equilibrium equations of these models are established in a strong form, that is, differential equations of structural analysis. For this reason, by introducing a new weak form of the equilibrium equation for beams (Zhang et al., 2000), the interaction between the bending moment and the axial force can be properly evaluated under generalized loading conditions. In calculating the stiffness matrix of a RC beam, the concept of the discretization of RC material proposed by Li and Kubo (1999) is introduced to the new beam theory proposed in this paper in which the compatibility of deformation is satisfied.

As to the nonlinearity of soil, it is known that in order to simulate the mechanical behavior of soils under generalized stress condition, a sophisticated constitutive model is usually necessary, which often means that the determination of a few parameters based on laboratory tests and in situ measurements is needed. For this reason, a kinematic hardening elastoplastic constitutive model using the concept of subloading, known as tij subloading model (Chowdhury et al., 1999), is adopted for soils in the 3-D dynamic analysis conducted in this paper.

The purpose of the paper is to provide an accurate numerical method of evaluating the dynamic behavior of a group-pile foundation based on 3-D finite element analysis for a superstructure-foundation-ground system, in which the nonlinearity of a pile is described by the axial-force dependent model and soils are described with the tij subloading model.

2 NONLINEARITY OF RC MATERIAL AND SOIL

2.1 *New beam theory considering the axial-force dependency*

In this paper, the derivation of the new beam model considering the axial-force dependency and taking a weak form of the equilibrium equation for a beam, which satisfies the compatibility of the deformation, is given in detail. For abbreviation, the model is called as axial-force dependent model (AFD model). In the model, the plane-section assumption is still kept valid and the stress-strain relations of reinforcement and concrete are shown in Figure 1.

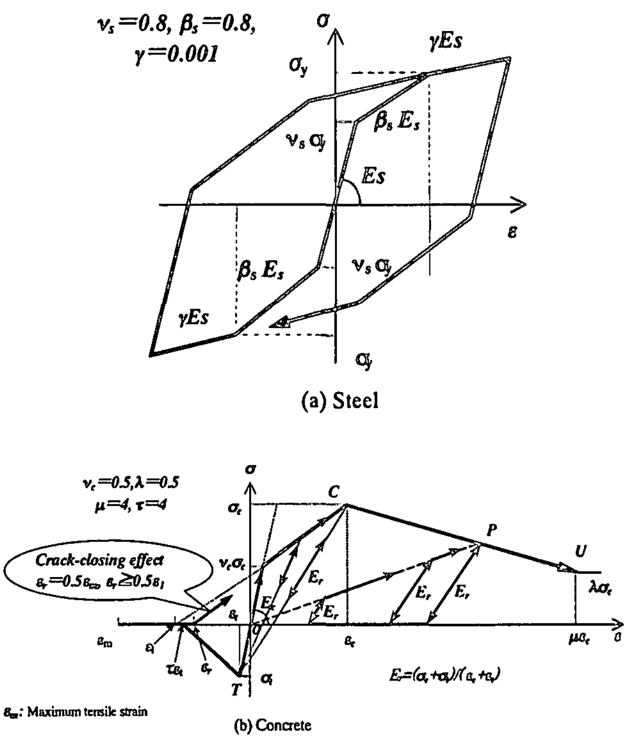


Figure 1 Nonlinear properties of reinforcement and concrete

ε_a, the strain at a arbitrary point *P(x,y)* at the sectional plane of a beam, can be divided into three parts, that is, the bending ε_1 strain due to M_x, the bending ε_2 strain due to M_y and axial strain ε_0 due to axial force, as shown in following equation,

$$\varepsilon_a = \varepsilon_1 + \varepsilon_2 + \varepsilon_0 = \left(x\{H_u''(z)\}^T + y\{H_v''(z)\}^T - \{H_w'(z)\}^T \right) \cdot [A]\{\delta\}$$

$$= \{F(z)\}^T \cdot [A]\{\delta\} \tag{1}$$

where

$\{\delta\} = \{u_i \quad v_i \quad w_i \quad \theta_{xi} \quad \theta_{yi} \quad u_j \quad v_j \quad w_j \quad \theta_{xj} \quad \theta_{yj}\}^T$ is a nodal displacement vector.

$$\{F(z)\}^T = \left(x\{H_u''(z)\}^T + y\{H_v''(z)\}^T - \{H_w'(z)\}^T \right) \tag{2}$$

$$
\begin{aligned}
\left\{H_u''(z)\right\}^T &= \{0\ 0\ 0\ 0\ 0\ 2\ 0\ 0\ 0\ 6z\} \\
\left\{H_v''(z)\right\}^T &= \{0\ 0\ 0\ 0\ 0\ 0\ 2\ 0\ 6z\ 0\} \\
\left\{H_w'(z)\right\}^T &= \{0\ 0\ 0\ 0\ 0\ 0\ 0\ 1\ 0\ 0\}
\end{aligned}
\tag{3}
$$

$$
[A]=\begin{pmatrix}
1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 \\
-3/l^2 & 0 & 0 & 0 & -2/l & 3/l^2 & 0 & 0 & 0 & -1/l \\
0 & -3/l^2 & 0 & -2/l & 0 & 0 & 3/l^2 & 0 & -1/l & 0 \\
0 & 0 & -1/l & 0 & 0 & 0 & 0 & 1/l & 0 & 0 \\
0 & 2/l^3 & 0 & 1/l^2 & 0 & 0 & -2/l^3 & 0 & 1/l^2 & 0 \\
2/l^3 & 0 & 0 & 0 & 1/l^2 & -2/l^3 & 0 & 0 & 0 & 1/l^2
\end{pmatrix}
\tag{4}
$$

The virtual energy stored in the beam element due to a virtual strain can be expressed as,

$$
\begin{aligned}
U &= \iiint_v \sigma_a \cdot d\varepsilon_a dv \\
&= \{d\delta\}^T \iiint E \cdot [A]^T \{F(z)\} \cdot \{F(z)\}^T [A] dv \cdot \{\delta\}
\end{aligned}
\tag{5}
$$

On the other hand, virtual energy W brought about by the external force due to a virtual displacement, is $W=\{d\delta\}^T\{F\}$. Therefore, the virtual energy theory $(W=U)$ can be obtained as

$$
\{F\} = \iiint E \cdot [A]^T \{F(z)\} \cdot \{F(z)\}^T [A] dv \cdot \{\delta\} = [K] \cdot \{\delta\} \tag{6}
$$

where $[K]$is the stiffness matrix of the beam element and can be rewritten as

$$
[K] = \iiint E [A]^T [I][A] dv \tag{7}
$$

$$
\begin{aligned}
[I] = \{F(z)\} \cdot \{F(z)\}^T &= \left(x\left\{H_u''(z)\right\} + y\left\{H_v''(z)\right\} - \left\{H_w'(z)\right\}\right) \\
&\cdot \left(x\left\{H_u''(z)\right\}^T + y\left\{H_v''(z)\right\}^T - \left\{H_w'(z)\right\}^T\right).
\end{aligned}
\tag{8}
$$

In the classical beam theory, $[I]$ is defined by Equation 9 in which the influence on the M-Φ relation due to the axial force is not considered, namely,

$$
\begin{aligned}
[I] = [I_1] &= x^2\left\{H_u''(z)\right\} \cdot \left\{H_u''(z)\right\}^T + \\
&\quad y^2\left\{H_v''(z)\right\} \cdot \left\{H_v''(z)\right\}^T + \left\{H_w'(z)\right\} \cdot \left\{H_w'(z)\right\}^T
\end{aligned}
\tag{9}
$$

Based on Equations 8 and 9, $[I]$ can be rewritten as

$$
[I] = [I_1] + [I_2] \tag{10}
$$

where $[I_2]$is evaluated by the following equation:

$$
\begin{aligned}
[I_2] &= xy \cdot \left(\left\{H_u''(z)\right\} \cdot \left\{H_v''(z)\right\}^T + \left\{H_v''(z)\right\} \cdot \left\{H_u''(z)\right\}^T\right) \\
&- x\left(\left\{H_u''(z)\right\} \cdot \left\{H_w''(z)\right\}^T + \left\{H_w''(z)\right\} \cdot \left\{H_u''(z)\right\}^T\right) \\
&- y\left(\left\{H_v''(z)\right\} \cdot \left\{H_w''(z)\right\}^T + \left\{H_w''(z)\right\} \cdot \left\{H_v''(z)\right\}^T\right)
\end{aligned}
\tag{11}
$$

$[I_2]$ is the newly added item which takes into consideration the influence on the M-Φ relation due to the axial force. Based on the above equations, $[K]$ can be expressed as

$$
[K] = [A]^T \cdot [D] \cdot [A], \quad [D] = \begin{bmatrix} 0_{5\times5} & 0_{5\times5} \\ 0_{5\times5} & D_1 \end{bmatrix} \tag{12}
$$

where

$$
[D_1] = \begin{bmatrix}
4lEI_y & 4lEI_{xy} & -2lEx & 6l^2EI_{xy} & 6l^2EI_y \\
4lEI_{xy} & 4lEI_x & -2lEy & 6l^2EI_x & 6l^2EI_{xy} \\
-2lEx & -2lEy & lEA & -3lEy & -3l^2Ex \\
6l^2EI_{xy} & 6l^2EI_x & -3lEy & 12l^3EI_x & 12l^3EI_{xy} \\
6l^2EI_y & 6l^2EI_{xy} & -3l^2Ex & 12l^3EI_{xy} & 12l^3EI_y
\end{bmatrix}
\tag{13}
$$

$$
\begin{aligned}
&EA = \iint E \cdot dA, \quad EI_y = \iint E \cdot x^2 \cdot dA \\
&EI_x = \iint E \cdot y^2 \cdot dA, \quad EI_{xy} = \iint E \cdot x \cdot y \cdot dA \\
&Ex = \iint E \cdot x \cdot dA, \quad Ey = \iint E \cdot y \cdot dA
\end{aligned}
\tag{14}
$$

The integration in Equation 14 can be evaluated by discretizing area A into N small areas A_i and taking the sum as

$$
EI_y = \iint E \cdot x^2 \cdot dA = \sum_{i=1}^{N} E_i x_i^2 A_i . \tag{15}
$$

As to the discretization of the RC circular section, it is discretized to m-concentric rings with equal thickness of $r=r_j-r_{j-1}=R/m$, as shown in Figure 2. Each ring is divided again into $8j-4$ elements for the j-th ring $(j=1,2...m)$. It makes a total of $4m^2$ divided elements with equal areas of $a=\delta R^2/4m^2$. The elements of the j-th ring are located on the circumference with radius.

In order to confirm the validity of AFD model proposed in this paper, a cantilever with a circular section, whose material parameters are listed in Table 1, subject to monotonic and cyclic loads, is calculated with a finite element beam analysis based on the proposed theory. The cantilever considered here is an RC material with a length of 8 m and a diameter of 1.2 m. In the calculations, the cantilever is simulated with eleven beam elements and the circular section is divided into 100 elements in the form as shown in Figure 2. Monotonic and cyclic axial and/or bending loads are applied at the top of the cantilever. It is found from the analysis that the axial-force dependent M-Φ relation is well simulated under monotonic and cyclic loading conditions, as shown in Figures 3 and 4.

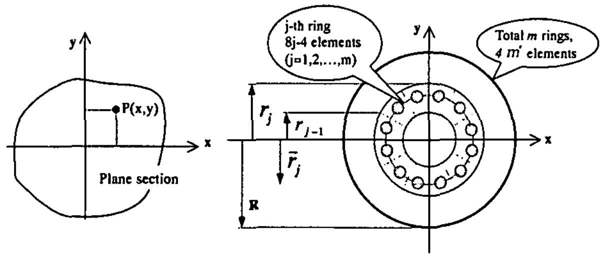


Figure 2 Discretization of a circular section of RC pile

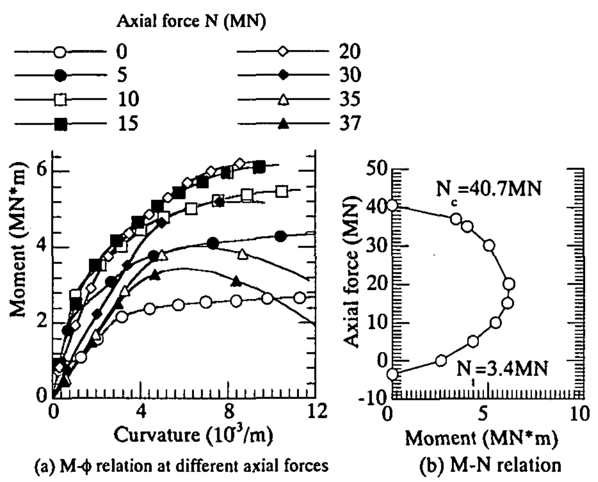


Figure 3 Nonlinear behaviors of Cantilever I under monotonic loading

Table 1 Parameters of RC cantilever

Compressive strength of concrete : σ_c=36.0 MPa
Tensile strength of concrete : σ_t=3.0 MPa
Young's modulus of concrete: E_c=2.5×10^4 MPa
Young's modulus of steel: E_s=2.1×10^4 MPa
Yielding strength of steel: σ_y=3.8×10^2 MPa
reinforcement: ϕ29×24
Overburden of reinforcement: 15 cm
Diameter: 120 cm

2.2 *Nonlinear properties of soils*

Nakai and Matsuoka (1986) proposed tij clay model based on the concept of *SMP* (spatially mobilized plane), in which the influence of the intermediate principal stress can be properly

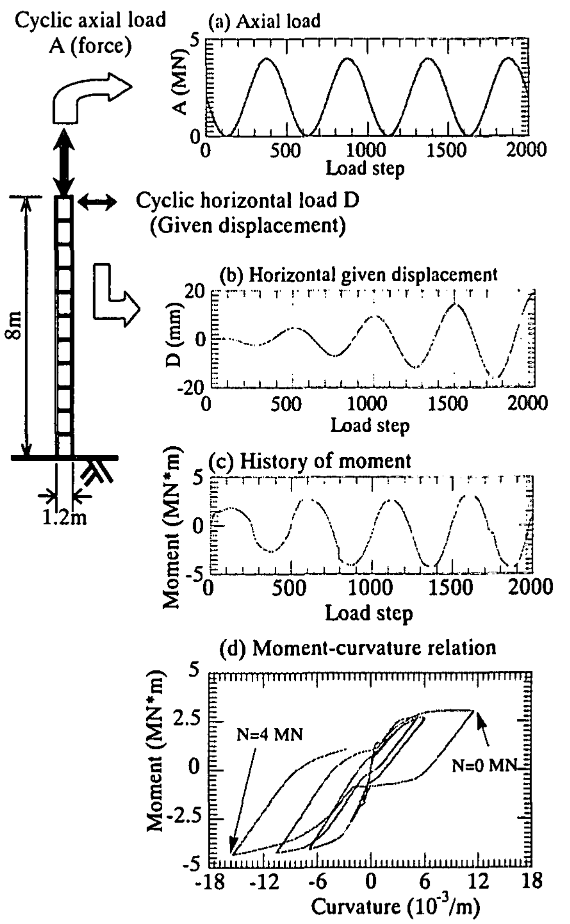
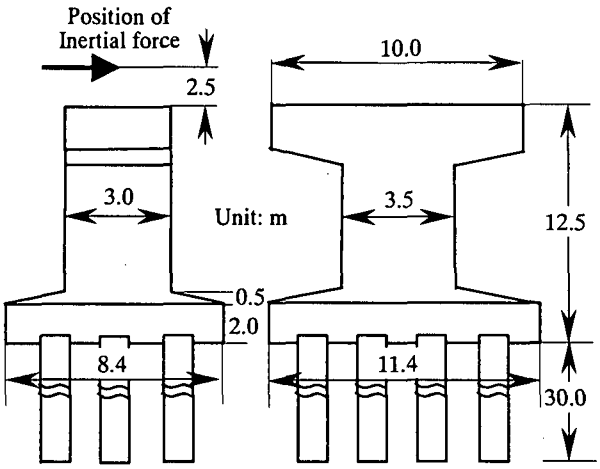


Figure 4 Nonlinear behavior of Cantilever I under cyclic loading

evaluated. The model has been verified through many true triaxial tests on normally consolidated clay in generalized stress paths. Chowdhury et al. (1999) extended the tij clay model to a kinematic model using the concept of subloading so that it not only can describe a monotonic loading but also a cyclic loading. The model can describe the stress-strain-dilatancy relation of soft clay well in monotonic and cyclic loadings. The model consists of seven parameters that can be determined with conventional triaxial compression tests. Detailed discussion about how to determine these parameters can be referred to corresponding reference.

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

Based on the constitutive models for soils and RC material discussed in sections 2, 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 5. 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 sandy reclaimed soil, followed

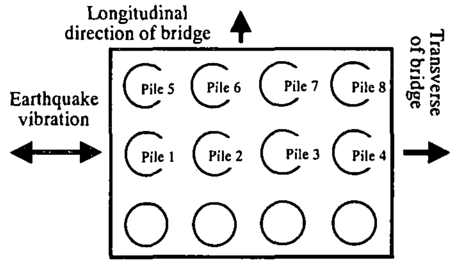


(a) Longitudinal direction of bridge (b) Side direction of bridge

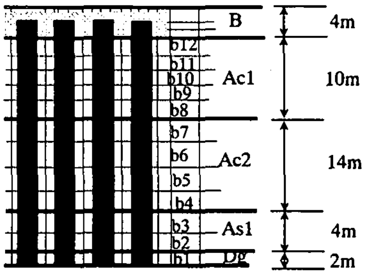
Figure 5 Geometry of the elevated highway bridge

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, diluvial gravel. The bottom layer of the ground is supposed to be an elastic material in the numerical analysis. Figure 6 shows the setup of the group-pile foundation and the geologic profile of the ground. In the figure, b_1 ~b_{12} represent the numbers of the beam elements whose results are discussed in detail.

A dynamic analysis of the elevated highway bridge using a full system is conducted to simulate the mechanical behavior in a major earthquake. A direct integration method of Newmark-β is adopted. The finite element mesh for the ground is shown in Figure 7. Figure 8 shows the input earthquake wave used in the analysis, which is an earthquake wave recorded in Port-island of Kobe during the Hyogoken-Nanbu earthquake in Japan. It has a maximum acceleration of 687 gal in a horizontal direction. A Rayleigh type of damping is adopted and the



(a) Plane view of piles and footing



(b) Geologic profile of ground and the position of the elements under consideration

Figure 6 Setup of group-pile foundation and ground

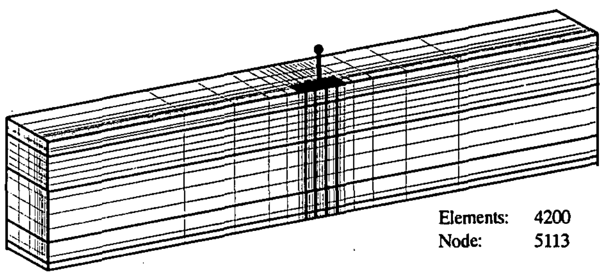


Figure 7 Finite element mesh

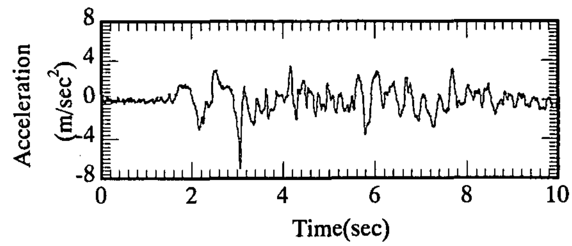


Figure 8 Input earthquake wave

damping factors of the structures and the ground are assumed as 2%, 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 damp is assumed to be constant irrespective of the changes in the stiffness matrix.

In calculating 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 1.155 and 1.067 sec., respectively. The eigenvalue analysis is conducted with a hybrid of Jacobian and subspace methods. In the dynamic analysis, the time interval of the integration is 0.01 sec.

Table 2 Parameters of pile

Compressive strength of concrete : σ_c=2.4×10^4 kPa
Tensile strength of concrete : σ_t=3.5×10^3 kPa
Young's modulus of concrete: E_c=3.0×10^7 kPa
Yielding strength of steel: σ_y=3.0×10^5 kPa
Arrangement of the reinforcement:
Part A (0.00-2.40): main: ϕ29×28, hoop: ϕ16 ctc 150, OB:15 cm
Part B (2.40-10.0): main: ϕ29×28, hoop: ϕ16 ctc 300, OB: 15 cm
Part C (10.0-30.0): main: ϕ29×14, hoop: ϕ16 ctc 300, OB: 15 cm

OB: Overburden of reinforcement; ctc: center-to-center distance

The parameters of the piles described by ADF model are listed in Table 2. Because the parameters involved in tij subloading model should be evaluated by laboratory tests which are not available in present cases, these values are estimated with the N value of SPT in such a way that the initial stiffness for tij subloading model and D-P model is the same in small strain level and that the shear stress ratio q/σ'_m at critical state for both models is the same, if noticing the fact that four parameters involved in D-P model can be easily determined with the N value of SPT (Zhang et al., 2000). The corresponding values of the parameters of tij subloading model for clays are listed in Table 3(a). The sandy soils of the ground are described by D-P model and their values are listed in Table 3(b).

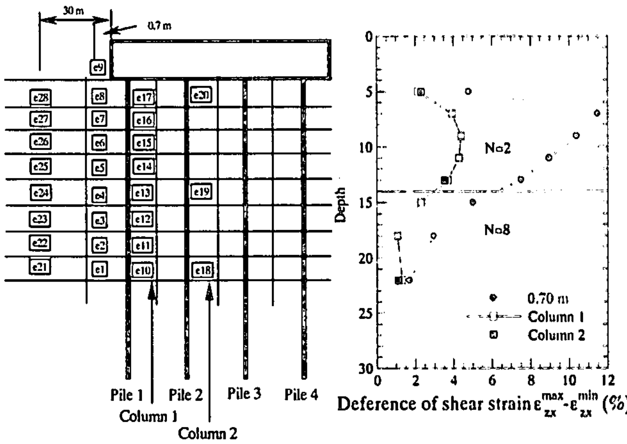


Figure 9 Distribution of shear strain in ground at different positions

Figure 9 shows the distribution of the maximum difference of shear strain ($\varepsilon_{zx}^{\max} - \varepsilon_{zx}^{\min}$) occurred at different positions. It can be seen that the shear strains of the soils between the piles (soils of Column 1 and Column 2) are almost the same as those at far field (30 m away from the footing). While the shear strain of the soils in front of the piles are much larger than the others, showing that large shear strains occur at the interface between the soil and outside piles of a pile group. The soils within the piles, however, behave in the same way as the soils at far field.

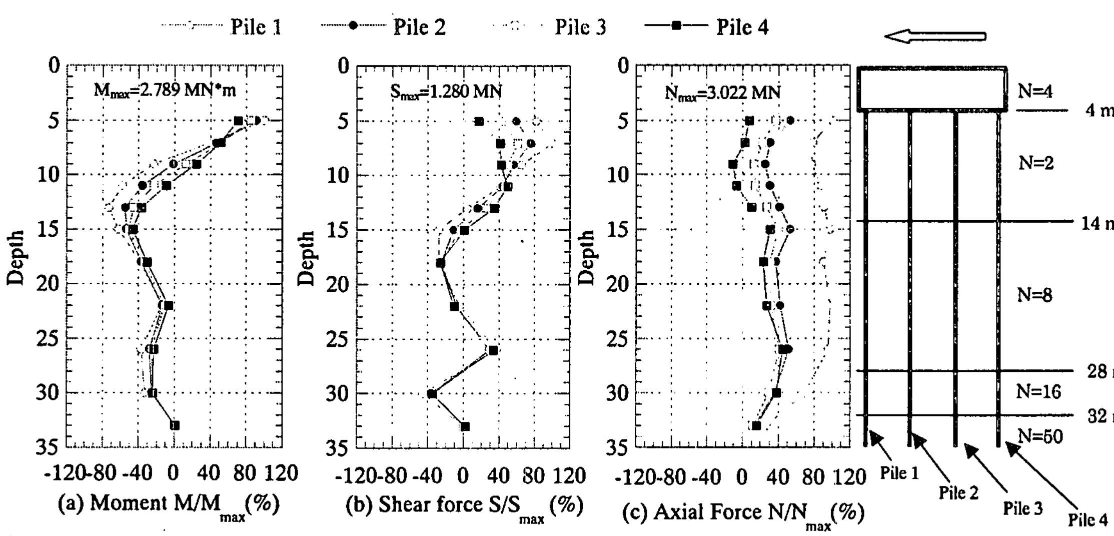


Figure 10 Distribution of sectional forces of group piles when t = 3.10 second

Table 3 Material parameters of the ground

(a) Clays described by tij subloading model

Soil	N	γ (kN/m^3)	λ	κ	ϕ' (°)	a	X_r	k_r	e_0	ν
Ac1	2	17.0	0.0553	0.0081	34.6	1.0	0.3	5.0	0.88	0.42
Ac2	8	18.0	0.0088	0.0017	34.6	1.0	0.3	5.0	0.88	0.40

(b) Sandy soils described by D-P model

Soil	N	γ (kN/m^3)	E (MPa)	ν	K_0	c (KPa)	ϕ (°)
B	4	18.0	10	0.30	1.00	0.0	23
As1	16	19.0	40	0.30	0.50	0.0	35
Dg	50	20.0	125	0.30	0.50	elastic	elastic

Figure 10 shows the distribution of the sectional forces in the piles at the time of 3.1 second when the ground deforms in the left direction. In the figure, the sectional forces M, N and S are normalized with the maximum values of M_{max}, N_{max} and S_{max} respectively. In this case, Pile 1 is a front pile and Pile 4 is a back pile. Due to the different axial force happened in Pile 1 to Pile 4, the distributions of the bending moments and the shear forces in different piles are quite different and show a very clear tendency, that is, the larger the axial force is, the larger the bending moment and shear force will be. In the figure, the distributions of the sectional forces are unified by dividing the corresponding maximum forces as shown in figure. It is known from the calculation that the difference of the maximum bending moment in four piles, which occurs at the heads of piles, is about 30 %. While the difference of the maximum shear force is about 150 %.

Figure 11 shows the load sharing ratios of the piles at the depths of 5 m, 7 m and 15 m. In the figure, the sectional forces N and S are normalized with the maximum values of N_{max} and S_{max} respectively. It is clear that the shear force in the piles changes with the axial force in seismic loading in all depths.

From Figures 10~11, it can be concluded that the phenomenon that the bending strength and the load-sharing ratio of the piles in a pile group subjected to lateral loading can be properly simulated with the analysis.

4 CONCLUSIONS

Based on the AFD model that can properly describe the axial-force dependency of RC materials, a series of three-dimensional elastoplastic finite element dynamic analyses are conducted on a superstructure-foundation-ground system. The conclusion can be given as follow:

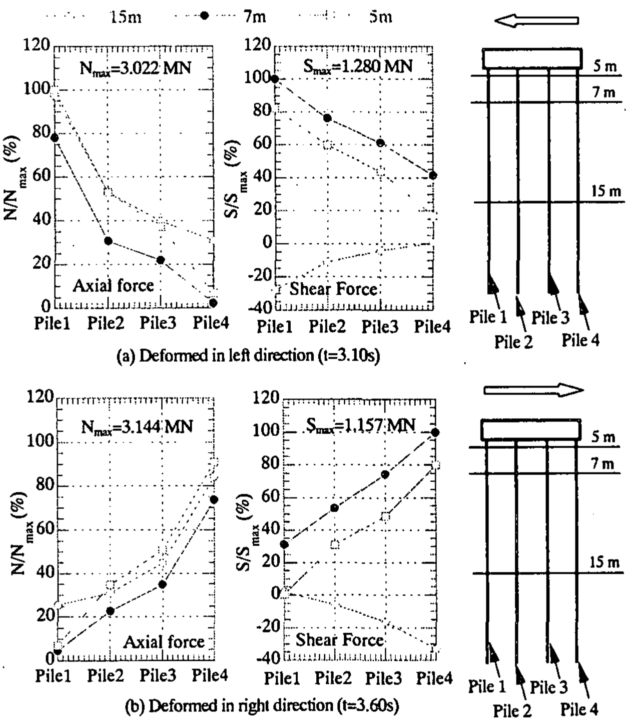


Figure 11 Load sharing ratio of group piles

The AFD model for the *M-Φ* relation of RC material is proposed based on a new weak form of the equilibrium equation for RC material. It can be applied to finite element analyses and satisfies the compatibility of deformation.

The AFD model can well simulate the axial-force dependency of RC materials under monotonic and cyclic loadings. By introducing the AFD model for RC material, the difference among the distribution in bending moments of front, back and middle piles in a group-pile foundation subjected to seismic loading can be well simulated. Without using the model, it is impossible to simulate well the difference aforementioned.

Large moment not only occurs at the head of piles, usually caused by the inertial force from the upper structure, but also within the ground. Therefore, it is worth emphasizing that the influence of the deformation of a ground on the piles must be considered carefully.

The validity of the numerical analyses proposed in this paper, by which the dynamic behaviors of a group-pile foundation is predicted, should be verified with model tests or centrifuge tests in future research.

REFERENCES

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.

Horikoshi, K., Ohtsu, H., Kimura, M. and Oka, F. 1996. Investigation of piles damages by the 1995 Hyogoken-Nanbu Earthquake, Tsuchi-to-Kiso, Vol. 44, No.11, pp. 27-29 (in Japanese).

Kimura, M. and Zhang, F. 2000. Seismic evaluation of pile foundations with three different methods based on three-dimensional elasto-plastic finite element analysis, Soils and Foundations, Vol. 40, No.5, pp. 113-132.

Lai, S. S., Will, G. T. and Otani, S. 1984. Model for inelastic biaxial bending of concrete member, Journal of Structural Engineering, ASCE, Vol. 110, No. 11, pp. 2568-2584.

Li, K. N. and Kubo, T. 1999. Analysis of circular RC member in MS/fiber model, Summaries of Technical Papers of Annual Meeting, Hiroshima, St. II, AIJ.

Murono, T., Nishimura, A. and Nagazuma, M. 1997. Model test on a group-pile foundation considering the vibration of ground in seismic evaluation, Proc. of the 24th Symposium of Japan Earthquake Engineering, pp. 625-628 (in Japanese).

Nakai, T. and Matsuoka, H. 1986. A generalized elasto-plastic constitutive model for clay in three-dimensional stresses, Soils and Foundations, Vol. 26 No. 3, pp. 81-98.

Zhang, F., Kimura, M., Nakai, T. and Hoshikawa, T. 2000. Mechanical behavior of pile foundations subjected to cyclic lateral loading up to the ultimate state, Soils and Foundations, Vol. 40, No.5, pp. 1-18.

Zhang, F., Yashima, A. Kimura, M. and Uzuoka, R. (2000). 3-D FEM Analysis of Laterally Cyclic Loaded Group-Pile Foundation Based on an Axial-Force Dependent Hysteretic Model For RC, Proc. of Int. Conf. on Geotechnical and Geological Engineering (GeoEng2000), Melbourne, Balkema, CD-ROM.

3-D DYNAMIC FINITE ELEMENT ANALYSIS ON GROUP-PILE FOUNDATION BASED ON AN AXIAL-FORCE DEPENDENT MODEL FOR RC

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It is commonly known that the dynamic behavior of a group-pile foundation is not only related to its own mechanical properties, but also to the upper structure supported by the foundation, and to the surrounding ground. The most important thing in the dynamic analysis of a full system that consists of superstructures, a foundation and a ground, however, is that the individual nonlinear behavior of soils and piles should be properly evaluated. In this paper, a new beam theory is proposed for reinforced concrete material (RC material). The theory is based on a weak form in which the axial-force dependency in the nonlinear moment-curvature relation is considered. Its validity is verified by experimental result RC cantilever beam. Then, a highway bridge with a 12-pile foundation is analyzed using a three-dimensional elastoplastic finite element analysis (DGPILE-3D). The piles are cast-in-place reinforced concrete pile and have a diameter of 1.2 m. Meanwhile, the ground soils are simulated with tij subloading model in which the concepts of the kinematic hardening and subloading are adopted. The purpose of the paper is to provide an accurate numerical way of evaluating the dynamic behavior of a pile foundation in earthquake.