13 Mar 1991, 1:30 pm - 3:30 pm

Dynamic Compliance of Pile Group Considering Nonlinear Behavior Around Piles

Koichi Kobayashi
Sato Kogyo Co., LTD., Kanagawa, Japan

Shintaro Yao
Kansai University, Osaka, Japan

Nozomu Yoshida
Sato Kogyo Co., LTD., Kanagawa, Japan

Proceedings: Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, March 11-15, 1991, St. Louis, Missouri, Paper No. 5.26

Dynamic Compliance of Pile Group Considering Nonlinear Behavior Around Piles

Koichi Kobayashi
Engineering Research Center, Sato Kogyo Co., LTD., Kanagawa, Japan

Shintaro Yao
Professor, Faculty of Engineering, Kansai University, Osaka, Japan

Nozomu Yoshida
Research Head, Engineering Research Center, Sato Kogyo Co., LTD., Kanagawa, Japan

SYNOPSIS Static and dynamic loading test of a foundation are carried out to obtain the behavior of a pile and a pile group. A conventional analysis based on the elastic wave theory are modified to obtain the complex rigidity of a pile group and are compared with the test results. It is shown that the analysis based on the conventional elastic wave theory are not enough to predict the dynamic behavior of piles since they do not take into account both the effect of slip and/or separation between the pile surface and the surrounding ground and the nonlinear behavior of the ground near the pile. An element to be put between the pile and surrounding ground is proposed so as to improve the analysis. The characteristics of the new element is represented as a compliance function which can be obtained from a static loading test of single pile.

INTRODUCTION

It is necessary to make clear the interactive effect between a pile foundation and ground as well as the vibration characteristics of a structure when dynamic behavior of the pile foundation-structure system is investigated. Many researches have been carried out to obtain the interactive behavior of pile-soil system. Novak(1976), Matlock(1980), Yao(1980), Scott (1982), and O'Neill(1986) carried out dynamic loading tests. Nogami(1976), Novak(1977), and Kobori(1977) obtained impedance functions at the pile top based on the elastic wave theory. Kuhlemeyer(1979) and Wolf(1980) obtained them through the finite element analyses and Sen (1985) got it by the use of boundary element method. However, there remains some uncertainty to use these theoretical result into the actual problems or design, which uncertainty comes from their assumption employed in the analyses. They used elastic or visco-elastic material and the assumption that the pile surface sticks to the surrounding soil. The analyses may not be true under the considerable loading because slip and/or separation between the pile surface and the surrounding soil and the nonlinear behavior of ground near the pile may take place. The validity of the analysis can be proved by comparing the analyses with test results or actual behavior, which comparison was hardly done.

This paper deals with the dynamic behavior of a pile-foundation system. Static loading tests of a pile and dynamic loading tests of a pile group were carried out to obtain the behavior of the pile-foundation system. The results are compared with the theoretical results based on the elastic wave theory. Based on these investigations, a new compliance function, which is to be used in addition to the compliance function derived from the conventional elastic wave theory, is proposed and is shown to be effective.

TEST

Test apparatus and test plan

The test was carried out at the alluvial deposit whose soil profile is shown in Fig. 1. In general, the site is composed from sandy clay soil with N-values ranging 1 to 10. P-wave and S-wave velocities are measured by an elastic wave exploration test and are also shown in the figure. Elastic deformation modulus E_d, elastic shear modulus G_d and Poisson's ration shown in the figure are calculated from elastic wave velocities. Figure 1 also shows the deformation coefficient E_s obtained from a borehole lateral load test using pressiometer.

Small size steel piles with 152.4mm diameter and 7m length were driven 6.75m into the ground by the drop hammer method. Dimensions of the piles are shown in Table 1. Configuration of piles is shown in Fig. 2.

Figure 3 shows static loading test apparatus of single pile subjected to horizontal or vertical load. The cyclic load is applied by a push-pull type hydraulic jack. Reactions of the hydraulic jack are carried out by a reaction frame which is supported by 4 piles in a pile group used in the dynamic loading test performed following the static loading test.

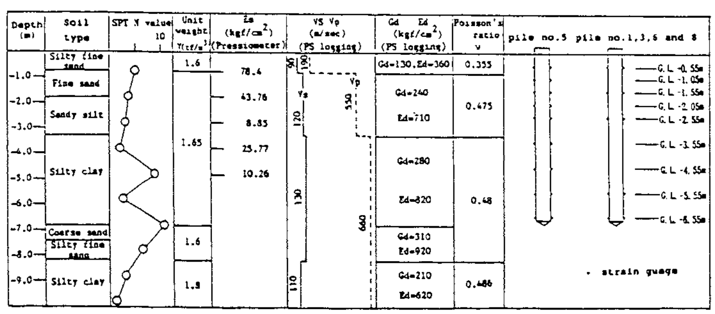

Fig. 1 Soil profiles

Figure 4 shows the dynamic loading test of the pile group schematically. A reinforced concrete foundation was made on the pile group after the static loading test. Total weight of the reinforced concrete foundation and vibration generator which is set on the concrete foundation is 4.02tf.

Places of measurements are also shown in Figs. 3 and 4. Beside them, totally 58 strain gages were put on 5 piles, whose positions are shown in Fig. 1.

The dynamic load was applied by the sweep up method; the frequendy is increased gradually keeping the eccentric mass moment constant. Eccentric mass moment is selected to either 10, 80, 120 or 1000 kg cm. Embedded concrete foundation was tested first to examine the effect of embedment. After that, soils surrounding the footing were taken off to test a foundation on the ground. Finally, soils beneath the concrete foundation was removed so that the concrete foundation was carried only by the plies, which results will be shown and be used in the following.

Table 1 Dimension of Piles

diameter	152.4 mm
cross-section area	23.2 cm^2
Young's modulus	2.1×10^6 kgf/cm^2
apparent unit weight	0.15 kgf/cm
2nd moment of inertial	629.5 cm^4

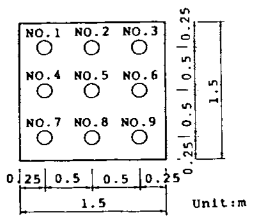

Fig. 2 Configration of piles (No.5 pile is used in the static loading test, No.1,3,7,9 piles are used to the reaction frame in the static loading test)

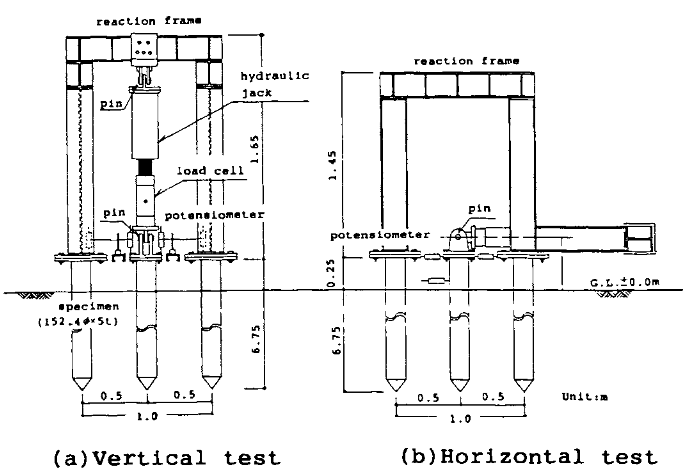

Fig. 3 Test apparatus for the static loading test

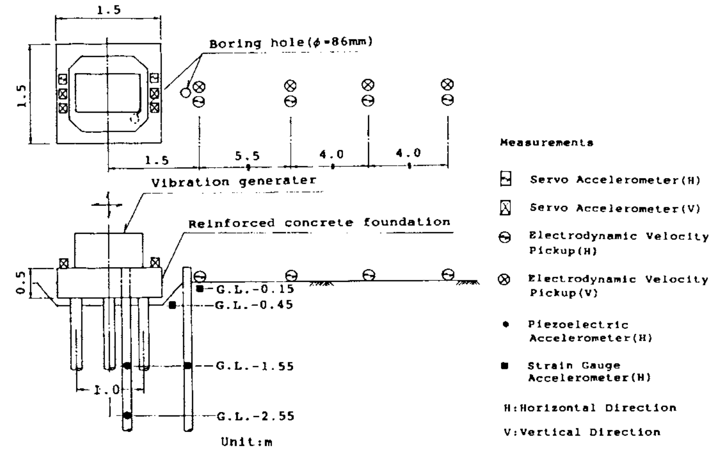

Fig. 4 Test apparatus for the dynamic loading test

Test result

Figure 5 shows horizontal load versus horizontal displacement relation at the pile top obtained from the horizontally loaded static test, where P_H denotes horizontally load and δ_H denotes horizontal displacement at the pile top. The hysteretic curve shows a spindle shape till the horizontal load exceeds 1.0tf, following which the shape moves towards a inverse-S shape. Figure 6 shows vertical load versus vertical displacement relation at the pile top obtained from the vertically loaded static test, where P_V denotes vertical load and δ_V denotes vertical displacement at the pile top. The hysteretic curves are spindle shape.

Figure 7 shows the resonance curve obtained from the horizontally loaded dynamic test, and Fig. 8 shows the ones obtained from the vertically loaded dynamic test. The resonance curves at the horizontally loaded tests have clear peak at about 15 Hz. Significant change of phase angle takes place around 15 Hz where amplitude becomes its maximum value. On the other hand, those in the vertically loaded test changes monotonically and no clear peak is observed.

Figures 9 and 10 show dynamic restoring force characteristics at the pile top obtained from horizontally loaded and vertically loaded test, respectively. Here, the load at the top of the pile group is computed by subtracting the inertia force of concrete foundation and vibration generator from the vibration force by the vibration generator. The restoring force characteristics shows spindle shape both in the horizontally loaded test and in the vertically loaded test.

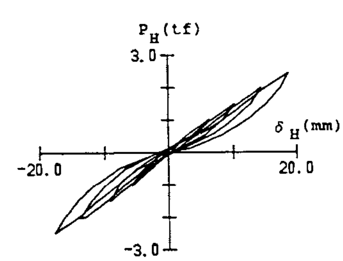

Fig. 5 P_H-δ_H relation for horizontal loading static test

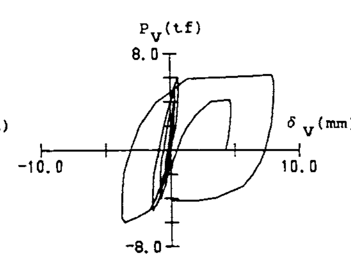

Fig. 6 P_V-δ_V relation for vertical loading static test

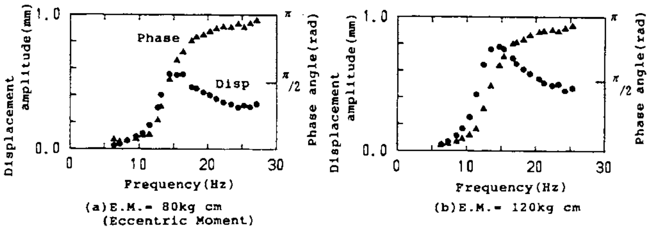

Fig. 7 Resonance curves for horizontally loaded dynamic test

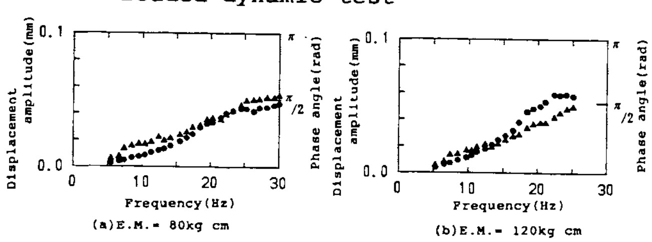

Fig. 8 Resonance curves for vertically loaded dynamic test

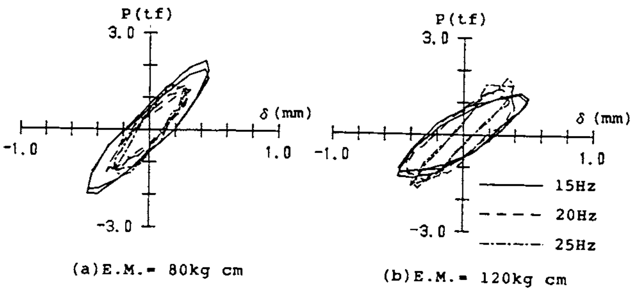

Fig. 9 Restoring force displacement relation for horizontally loaded test

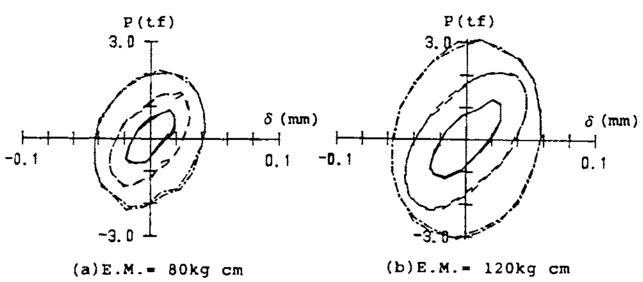

Fig. 10 Restoring force displacement relation for vertically loaded test

ANALYSIS

The analysis based on the elastic wave theory, from which complex rigidity at the pile top is obtained, is one of the convenient method to predict the behavior of pile-foundation system for the design purpose of a structure. These analyses have been carried out under the assumption that the ground behaves in an elastic or visco-elastic manner and that the soil completely sticks to the pile surface hence no relative movement between them takes place. However, it is obvious that these assumptions do not hold under the rather large earthquake motion. The slip and/or separation between the pile surface and surrounding ground will take place. The ground will behave in a nonlinear manner, in which case the local disturbance of the ground near the pile due to the existence of a pile or pile ground will affect the behavior of them. In this paper we propose a method which can take these effect into account. Complex rigidity of a pile group can be obtained from the complex rigidity of two piles on the bases of the principles of superposition.

Complex rigidity of a pile group under vertical load

In this section we first introduce a method to get the complex rigidity of two piles based on the elastic wave theory, then we extend it into general case.

Let's consider two equal piles shown in Fig. 11, which are driven into the rigid foundation and are subjected to vertical load at each top. Equation of motion of each pile is represented as follows:

$$E_i A_i \frac{d^2 w_i}{dz^2} + m_i \omega_i^2 w_i = P_{fi} \qquad (1)$$

where E denotes Young's modulus, A denotes cross-sectional area, m denotes mass density, ω denotes circular frequency and P_f denotes frictional reaction of the ground. The coordinates axis z and plie displacement w are positive in the downward direction. Subscript i is used to distinguish two piles, hence it takes the value 1 or 2. To analyze the interactive effect of plie group, Nogami et al (1983) assumed that plane strain condition holds in each sublayer which is shown as shaded area in Fig. 11 as an example. Following their method, the relation between the reaction of the ground, $\{P_f\}=\{P_{f1}\ P_{f2}\}^T$, and vertical displacement of the pile, $\{w\}=\{w_1\ w_2\}^T$, is expressed as

$$\{P_f\}=[k]\{w\} \qquad (2)$$

in which

$$[k]=\begin{bmatrix} k_{11} & k_{12} \\ k_{21} & k_{22} \end{bmatrix}=\begin{bmatrix} \beta & 0 \\ 0 & \beta \end{bmatrix}\begin{bmatrix} \alpha_{11} & \alpha_{12} \\ \alpha_{21} & \alpha_{22} \end{bmatrix}^{-1} \text{: stiffness matrix} \qquad (3)$$

where

$$\alpha_{ij}=K_0\left(\frac{a_0^* r_{ij}}{r_0}\right)$$

$$\beta = 2\pi\mu^* a_0 K_1(a_0^*)$$

K_0, K_1: 2nd kind modified Bessel function of order of 0 and 1

r_0 : radius of the pile

$$r_{ij}=\begin{cases} r_0 & \text{for } i=j \\ r_{12} & \text{for } i\neq j \end{cases}$$

r_{12}: distance between the centers of two piles

$\mu^* = \mu(1+iD)$

D : damping ratio

μ : shear modulus of ground

$$a_0^* = \frac{a_0}{\sqrt{1+iD}}\, i$$

$a_0 = r_0 / Vs$: dimensionless frequency

Vs : shear wave velocity of ground

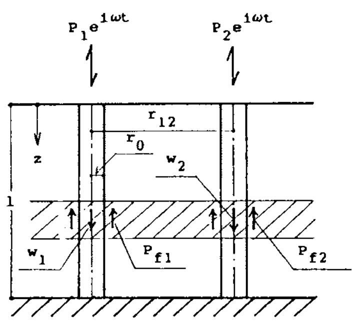

Fig. 11 Schematic figure of analysed model under vertical load

The governing differential equations of a pile group with two piles are obtained by substituting Eq. 2 into Eq. 1. In the case that both piles are subjected to equal loads, i.e., $P_{V1}=P_{V2}=P_V$, $\omega_1=\omega_2=\omega$, the governing equation can be reduced to one equation:

$$EA\frac{d^2w}{dz^2} + m\omega^2 w = kw \qquad (4)$$

where $k=k_{11}+k_{12}$. From the solution of Eq. 4 under the boundary condition:

$$w = 0 \qquad \text{at } z=l$$
$$P_V = EA\frac{dw}{dz} \qquad \text{at } z=0 \qquad (5)$$

the relation between the vertical load at the pile top, P_V and the vertical displacement at the pile top, δ_V is obtained as

$$P_V = EA\alpha\frac{1 - e^{2\alpha l}}{1 + e^{2\alpha l}}\delta_V = k_G\delta_V \qquad (6)$$

where

$$\alpha = \sqrt{\frac{k - m\omega^2}{EA}}$$

and

$$k_G = EA\alpha\frac{1 - e^{2\alpha l}}{1 + e^{2\alpha l}} \qquad (7)$$

is complex rigidity at the plie top in which interactive effect of two piles is taken into account.

Looking at Eq. 3, it clear that the interactive effect between two piles comes from the nondiagonal term of the stiffness matrix [k]. Therefore complex rigidty of single pile (a pile without other piles), k_s, is computed by putting $\alpha12=\alpha21=0$ or $k_{12}=k_{21}=0$:

$$k_s = EA\alpha_s\frac{1 - e^{2\alpha_s l}}{1 + e^{2\alpha_s l}} \qquad (8)$$

where

$$\alpha_s = \sqrt{\frac{k_{11} - m\omega^2}{EA}} = \sqrt{\frac{\beta/\alpha_{11} - m\omega^2}{EA}}$$

The displacement of a pile is the sum of the displacement due to the load at its pile top and that due to the load at the other pile top. For example, displacement of pile 1 is expressed as

$$\delta_{V1} = \frac{1}{k_s}P_1 + (\frac{1}{k_G} - \frac{1}{k_s})P_2 \qquad (9)$$

It is noted that Eq. 9 holds exactly only when P_1 equals to P_2. However, here we assume that Eq. 9 holds approximately when P_1 dose not equal to P_2. Then the relations between the load at each pile and displacements at the pile top are exprssed as

$$\begin{Bmatrix}\delta_{V1}\\ \delta_{V2}\end{Bmatrix} = \begin{bmatrix} f_{11} & f_{12}\\ f_{21} & f_{22}\end{bmatrix}\begin{Bmatrix}P_1\\ P_2\end{Bmatrix} \qquad (10)$$

where $f_{11}=1/k_s$ and $f_{12}=f_{21}=1/k_G-1/k_s$.

Yao (1977) showed that when computing the additional displacement due to the load at an other pile the existence of the third pile itself hardly affect the additional displacement. Therefore in more general case when a pile group is composed of more than two piles, the displacement of a pile in a pile group is computed as the sum of the displacement of single pile and the additional displacements due to the load at the other piles each of which additional displacement is computed under the assumption that there are only two piles : a referent pile and a loaded pile. Then the relations between the displacements and the load at the pile tops are written as

$$\{\delta_V\} = [f]\{P_V\} \qquad (11)$$

Here [f] is a flexibility matrix, whose component f_{ij} is computed as follows:

$$f_{ij} = \begin{cases} 1/k_s & \text{for } i=j \\ 1/k_{Gij} - 1/k_s & \text{for } i\neq j \end{cases} \qquad (12)$$

where k_{Gij} is the complex rigidity when only both i-th and j-th piles exist. The flexibility matrix of a pile group is obtained from only the complex rigidities of two pile with different pile distance in this method, which is the characteristics of the method. Figure 12 shows complex rigidity of the pile used in the test, in which calculation piles and soild are modeled as shown in Tables 1 and 2.

If the foundation is sufficently rigid, displacements of piles in a pile group is identical to each other. The load acting on the foundation is the sum of the load at each pile. Therefore the complex rigidity of a pile group-foundation system k_{GE} is obtained as

$$K_{GE} = \sum_{i=1}^{n}\sum_{j=1}^{n} k_{ij} \qquad (13)$$

where k_{ij} is the component of the inverse matrix of [f], and n is the number of piles in a pile group. So as to distinguish the complex rigidity proposed later, complex rigidity derived from the proceeding procedure is called elastic complex rigidity hereafter.

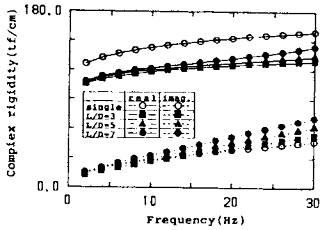

Fig. 12 Complex rigidity of a pile in a pile group with two piles (Vertical)

Table 2 Constants of Soil

shear wave velocity	120 m/sec
unit weigth	1.6 tf/m^3
Poisson's ratio	0.45
hysteresis damping ratios	0.05

<u>Complex rigidity of a pile group under horizontal loading</u>

The same procedure described in the previous section is used to compute complex rigidity of a pile group under horizontal loading. Figure 13 shows a model of a pile group with two equal piles. When loads at each pile top are identical, the equation of motion of two piles are also identical and are written as

$$-EI\frac{d^4u}{dz^4} + m\omega^2u = P_f^* \tag{14}$$

where E denotes Young's modulus, I denotes moment of inertia of a pile section, u denotes horizontal displacement and P_f^* denotes the horizontal reaction of the ground.

Following the method proposed by Nogami et al (1983), displacements of the two cylinders subjected to equal horizontal load P_f^* is computed by

$$u = \{f(r_0) + f(r_{12})\}P_f^* \tag{15}$$

where f(r) is the displacement of the ground distant r from the pile center when unit load is applied at a cylinder, and is expressed as follows depending on the direction of loading:

$$f(r) = \frac{1}{\mu}\frac{S_1}{S_0} \quad \text{for } \theta=0 \text{ degree}$$
$$f(r) = \frac{1}{\mu}\frac{S_2}{S_0} \quad \text{for } \theta=90 \text{ degree} \tag{16}$$

where θ is the angle of the direction of horizontal load measured from the axis connecting the pile center as shown in Fig. 13, and

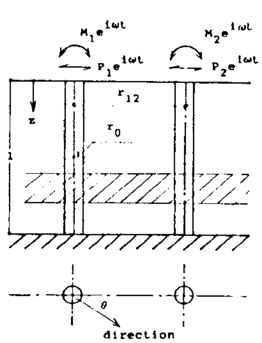

Fig. 13 Schematic figure of analysed model under horizontally load

$$S_0=\pi a_0{}^2\frac{r_{12}}{r_0}\{4K_1(q)K_1(s)+sK_1(q)K_0(s)+qK_1(s)K_0(q)\}$$

$$S_1=K_1(s\frac{r_{12}}{r_0})\{2K_1(q)+qK_0(q)\}-\{K_1(q\frac{r_{12}}{r_0})+q\frac{r_{12}}{r_0}K0(q\frac{r_{12}}{r_0})\}\{2K_1(s)+sK_0(s)\}$$

$$S_2=\{K_1(s\frac{r_{12}}{r_0})+s\frac{r_{12}}{r_0}K_0(\frac{r_{12}}{r_0})\}\{2K_1(q)+qK_0(q)\}-K1(q\frac{r_{12}}{r_0})\{2K_1(s)+sK_0(s)\}$$

$$a_0 = \frac{r^2\omega}{V_s}$$
$$q= \frac{a_0i}{\eta\sqrt{1+iD_s}} \tag{17}$$
$$s= \frac{a_0i}{\sqrt{1+iD_s}}$$
$$\eta= \frac{V_1}{V_s}$$

Substituting Eq. 15 into Eq. 14, equation of motion is rewritten as

$$-EI\frac{d^4u}{dz^4} + (m\omega^2 - \frac{1}{f(r_0) + f(r_{12})})u = 0 \tag{18}$$

The solution of Eq. 18 can be expressed using four unknown coefficient C_1,C_2,C_3 and C_4 as

$$u=e^{\beta z}(C_1\sin\beta z+C_2\cos\beta z)+e^{-\beta z}(C_3\cos\beta z+C_4\sin\beta z) \tag{19}$$

where

$$\beta = \sqrt[4]{\frac{k^* - m\omega^2}{4EI}}$$

The values of the unknowns are determined so as to satisfy the boundary conditions at the pile tip and pile top. However, it is known that the boundary condition at the pile tip does not affect the behavior around the pile top when Bl $>\pi$. The value of Bl lies in the range from 6.7 to 12.5 in this test, hence this condition is satisfied. Therefore we put C_1=C_2=0 instead of using the boundary condition at the pile tip. The other unknowns are determined using the boundary condition at the pile top:

$$P_H = -EI\frac{d^3u}{dz^3}$$
$$M = -EI\frac{d^2u}{dz^2} \quad \text{at } z=0 \tag{20}$$

From Eqs. 19 and 20, the relation between the load at the pile top and the displacement and deflection angle at the pile top, δ_H and ϕ, are computed as follows:

$$\begin{Bmatrix}P_H\\M_H\end{Bmatrix} = \begin{bmatrix}4EI\beta^3 & -2EI\beta^2\\-2EI\beta^2 & 4EI\beta\end{bmatrix}\begin{Bmatrix}\delta_H\\\phi_H\end{Bmatrix} \tag{21}$$

In the case that θ is not 0 nor 90 degrees, the horizontal load is divided into two components : components in the direction θ=0 and θ =90. Then the displacement at each load is computed independently. Total displacement is computing by summing these two displacements.

Figure 14 shows the complex rigidities of the pile-soil system used in the test. Following the procedure described in the previous section one can construct the relation between the displacements and load at the pile top. If the foundation is sufficiently rigid, rotation at the pile top is completely constrained and horizontal movements of the piles in a pile group at the pile top are identical to each other. Complex rigidity of the pile group-foundation system can be computed as the coefficient of the relation between the horizontal load of the foundation and horizontal displacement at the pile top.

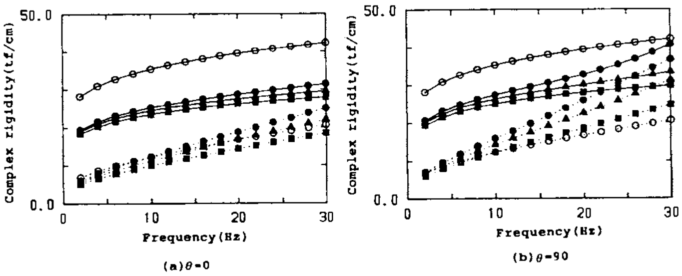

Fig. 14 Complex rigidity of a pile in a pile group with two piles(Horizontal)

Boundary elements

The method proposed by Nogami et al is used to compute the relation between the displacements of pile and the reaction of ground. Like the other method based on the elastic wave theory, pile surface is assumed to stick the soils surrounding the piles in this analysis. In other words, pile-soil system is modeled as shown in Fig. 15(a). However, the slip and/or separation may take place between the pile surface and surrounding soil, and the ground near the pile may behave in a nonlinear manner under the earthquake motion, which will affect the behavior at the pile top.

So as to consider these effects, the total displacement at the pile top, δ_T, is assumed to be divided into two components in the analysis presented here;

$$\delta_T = \delta_E + \delta_B \tag{22}$$

The idea is schmatically shown in Fig 15(b). The behavior of the ground far from the pile are assumed to be elastic and the displacement at the pile top due to elastic behavior is denoted as $_E$. The other effect, like separation between pile surface and soils and nonlinear behavior of soils near the pile, is taken into account as the characteristics of a volumeless element surrounding the pile, named pile-boundary element hereafter, and the displacement due to these effect is denoted as $_B$.

Using the notation shown in Fig. 15, Eq. 22 leads

$$\frac{1}{k_T} = \frac{1}{k_{GE}} + \frac{1}{k_{GB}} \tag{23}$$

where k_T is complex rigidity of single pile or a pile-founation system. As many researches have indicated, the elastic complex rigidity k_{GE} is frequency dependent quantity. However, since

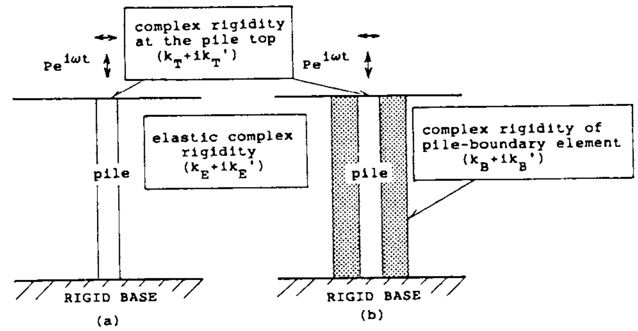

Fig. 15 Analytical model

the behavior of pile boundary element corresponds to the nonlinear behavior, the characterisitics of the pile boundary element, k_{GB} is frequency independent, hence can be obtained from the static loading test.

Equation 23 can be applied for single pile as well as a pile group, hence first we use it to obtain the complex rigidity of the pile-boundary element of single pile. Figure 16 shows complex rigidities of single pile top obtained from the horizontally and vertically loaded static tests, respectively. The term k_{GE} in Eq. 23 is k_s in Eq. 8 in the case of single pile, hence can be computed using known quantities shown in Tables 1 and 2. Substituting these complex rigidities into Eq. 23, the value of the complex rigidity of a pile-boundary element of single pile is obtained: 6.00+0.64i in tf/cm unit for horizontal loading and 1.70+335.6i for vertical loading.

Next we use Eq. 23 to obtain the complex rigidity of pile-foundation system. Here it is noted the methods of the superposition are different between the elastic complex rigidity and complex rigidity of pile-boundary element. The value of k_{GE} in Eq. 23 is computed by the method described in the previous section which takes into account the effect of pile group. On the other hand, pile boundary element is a fictitious element which represents local behavior around the pile such as slip and/or separation between the pile surface and soils. Therefore the existence of the other pile is supposed not to affect the characteristics of the complex rigidity of the referent pile boundary element. In other words, complex rigidity of the pile-boundary element of the pile group-foundation system, k_{GB} in Eq. 23, is obtained by summing the complex rigidity of the pile-boundary element of each single pile.

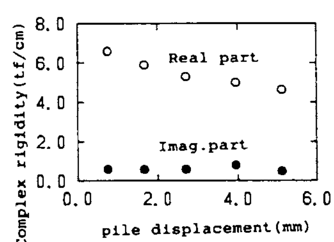

Fig. 16 Complex rigidity at the pile top (Horizontal)

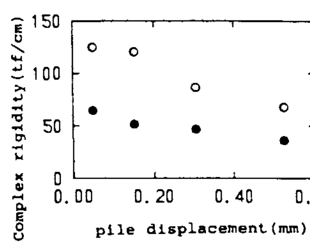

Fig. 17 Complex rigidity at the pile top (Vertical)

DISCUSSION

So as to show the effectiveness of the proposed method, the response of the pile foundation-ground system are calculated by using an 1-degee of freedom system with four complex rigidities calculated by various method. Elastic complex rigidity considering the effect of pile group is used in analysis-1. The complex rigidity of pile-boundary element is used in addition to the elastic rigidity used in the analysis-1 in analysis-2 which is the method we propose. The elastic complex rigidity which is 9 times the elastic complex rigidity of single pile is used in analysis-3. The complex rigidity of pile-boundary element is used in addition to the elastic rigidity of single pile used in the analysis-3 in analysis-4.

Figures 17 and 18 compare response spectra and Figs. 19 and 20 compare hysteresis loops. In the case of horizontal loading, the agreement is very bad in only an elastic complex rigidity is used. The use of pile-boundary element surprisingly improves both the response spectra and hysteretic curves compared with the analysis which uses only elastic complex rigidity, and the comparison between the test result becomes very good. On the other hand, the comparison between the test and analysis-1 is not bad in the case of vertical loading; the use of pile-boundary element improves them but not much. Comparison between the analysis-1 and analysis-3 or between the analysis-2 and analysis-4 indicate that the effect of pile group have more effect than the pile-boundary element. Although the same load was applied at the plie top both in the horizontally loaded test and vertically loaded test, resulting displacement seems much different because of the difference of rigidities. The maximum displacement at the pile top is about 0.6mm for horizontally loaded test and that is about 0.05mm for vertically loaded test as can be seen in Figs. 19 and 20. The maximum shear strains of the ground, therefore, are estimated the order of 10^{-3} in the horizontally loaded test and 10^{-4} in the vertically loaded test. In other words, the effect of nonlinear behavior of ground is much larger in the case of horizontally loaded test than that in the case of vertically loaded test. This is a main reason why pile boundary element have the predominant effect in the case of horizontally loaded test and the effect of pile group have the predominant effect in the case of vertically loaded test.

CONCLUDING REMARKS

The behavior of the pile foundation-ground system was gasped from the static and dynamic loading tests with size steel piles. A method to predict the dynamic behavior of the pile-ground system is proposed from the comparison between the test results and analysis based on the elastic wave theory. They are summarized as follows :

1) The comparison between the test results and analysis based on the conventional elastic wave theory indicates that nonlinear behavior near the pile affect the restoring force characteristics of pile top very much.
2) Both the effect of pile group and the effect of pile boundary element sould be taken into account to predict the behavior of pile foundation system.
3) THe effect of pile group is predominant when the load is small; the hysteresis loop becomes fat. The complex rigidity of pile group taking into account the effect of pile group can be obtained by the use of the principle of superposition from the one of two piles.
4) The effect of pile boundary element is predominant when the load is large. The characteristics of the pile-boundary element can be obtained from static loading test of single pile.

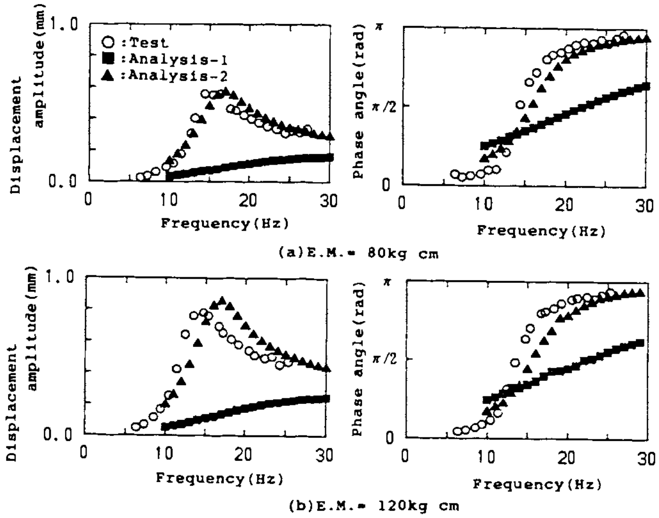

Fig. 18 Comparision of resonance curve for horizontally loaded dynamic test

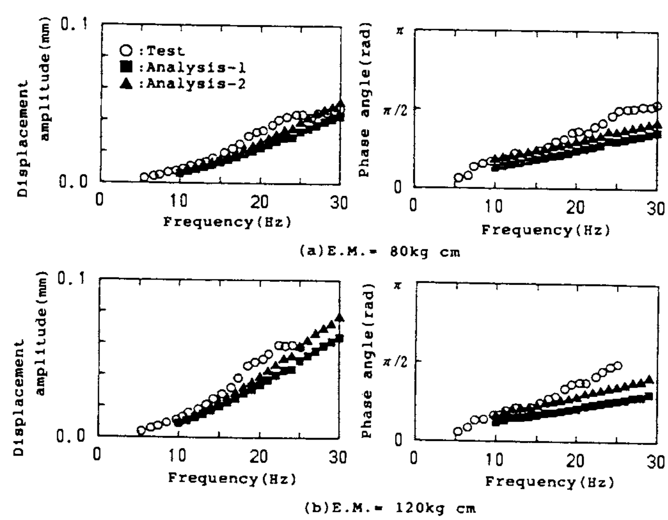

Fig. 19 Comparision of resonance curve for vertically loaded dynamic test

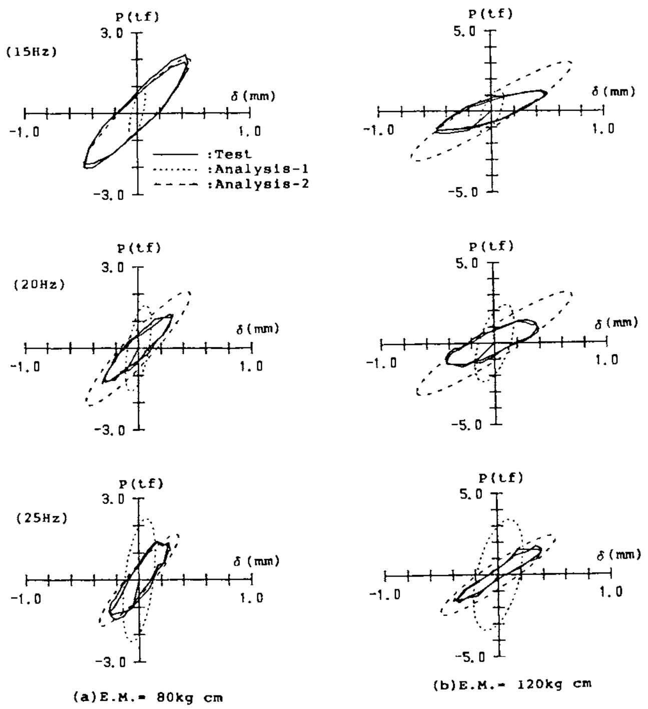

Fig. 20 Comparision of complex rigities under horizontal loading

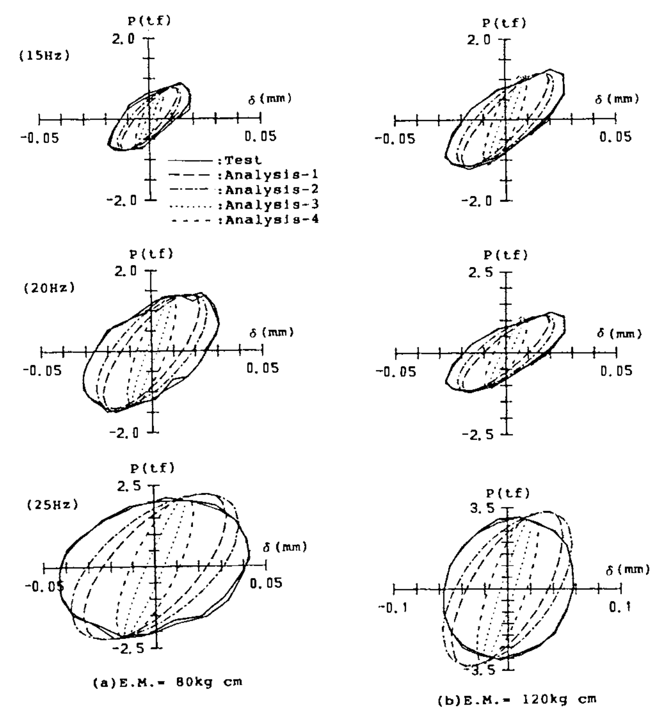

Fig. 21 Comparision of complex rigities under vertical loading

REFERENCES

Kobori,T., Minai,R. and Baba,K. (1977), "Dynamic Behavior of a Laterally Loaded Pile", Proc. 9th ICSMFE, Specially session 10, 6, Tokyo

Matlock,H., Ingram,W.B., Kelley,A.E. and Bogard, D.(1980), "Field Tests of the Lateral Load Behavior of pile Groups in Soft Clay", Proc. 12th Offshore Technology Conference

Novak,M. and Grigg,R.F. (1976), "Dynamic Experiments with Small Pile Foundations", Canadian Geotechnical Journals, Vol. 13, pp.372-385

Nogami,T. and Novak,M. (1977), "Soil-Pile Interaction in Vertical Vibration", Earthquake Eng. Struct. Dyn., Vol.4, pp.277-293

Novak,M. and Nogami,T. (1977), "Soil-Pile Interaction in Horizontal Vibration", Earthquake Eng. Struct. Dyn., Vol.5 pp.263-281

Nogami,T. (1983), "Dynamic Group Effect in Axial Response of Grouped Piles", J. Gt. Div., ASCE, Vol. 105, No.GT2, pp.289-304

Nogami,T. (1985), "Flexural Reapones on Grouped Piles under Dynamic Loading", Earthquake Eng. Struct. Dyn., Vol.13,pp.53-65

O'Neill,M. and Blaney,G.W. (1986), "Measured Lateral Response of Mass on Single Pile in Clay", J. Gt. Div., ASCE, Vol.112, No.GT4, pp.827-840

Scott,R.F., Tsai,C.-F., Steussy,D. and Ting,J.M. (1982), "Full-Scale Dynamic Lateral Pile Tests", Proc. 14th Offshore Technology Conference, Huston.

Sen,R., Davis,T.G. and Banajee, P.K. (1985), "Dynamic Analysis of Piles and Pile Groups Embedded in Homogeneous Soils", Earthquake Eng. Struct, Dyn.,Vol13,pp.53-65

Wolf,J.P. (1980), "Dynamic Stiffness of Group of Battered Piles", J. Gt. Div., ASCE, Vol.106, No.GT2, pp.198-203

Yao,S. (1977), "The Load-Settlement Behavior of Pile foundation", A Thesis for a Doctorate in Nagoya University (In Japanese)

Yao,S. (1980), "Dynamic and Static Test of Model Piles of Pile Groups", Proc. 7th WCEE, Vol.3, pp.457-466, Turkey

Yao,S., Yoshida,N. and Kobayashi,K. (1985), "An Experimental Study on Hysteresis Characteristics of a Pile subjected to a Vertical Cyclic Load", Abstract, Annual meeting of A.I.J., Vol.B, pp.1049-1050, Nagoya (In Japanese)

Yao,S. and Kobayashi,K. (1986), "An Experimental Study on Dynamic Behavior of Pile Foundation (Part 2, Horizontal Excitation)", 21st Japan National Conference on SMFE, Vol.1, pp.759-762 (in Japanese)