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Damage to Pile in Liquefied Ground and Applicability of Analysis

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Abstract

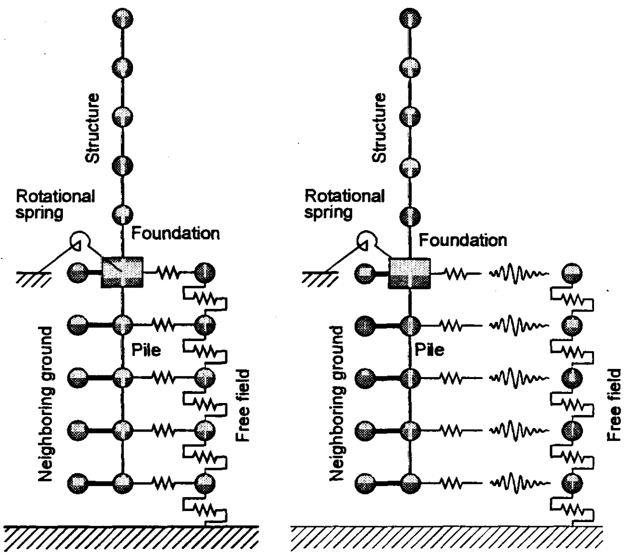
Analytical method to explain damage to piles associated with the ground deformation is investigated. Since three-dimensional FEM is far from practical use, a model based on Winkler type spring is investigated. A separate model to solve the ground and pile-structure system independently is introduced, in which ground response as well as earthquake motion at the referent point is applied in the pile-structure system. In order to make the separate model into practical use, a multiple-support excitation procedure are introduced and compared with the whole model, in which soil-pile-structure system is solved simultaneously. The agreement is perfect when both displacement and velocity of the ground are used as input on the pile-structure system. Then, piles damaged during the 1995 Kobe earthquake are analyzed by various method. The damage can be explained if the effective stress analysis is made in the analysis of the ground, but cannot by the total stress analysis. In addition, effect of several factors affecting the behavior is examined.

INTRODUCTION

Damage to piles has occurred in many past earthquakes. At first, damage caused by inertia force of a superstructure was found and investigated, in which case damage occurs at the pile top. After that, damage to piles associated with ground deformations was found; damage appeared near the boundary between soil layers with different stiffness, especially boundary between liquefied and non-liquefied layers. Damage to piles reduced load carrying capacity of the ground, and resulted in differential settlement of structures. In order to make a rational design of a pile in liquefiable layer, it is important to develop a relevant and practical analytical method.

One of the ideal analytical method may be to analyze a soil-pile-superstructure system simultaneously by, for example, a finite element method. This method, however, is not practical at present because of several reasons. Since two- dimensional analysis has difficulty in expressing three-dimensional behavior of soil-pile-structure system, three-dimensional analysis is preferable, but it requires huge amount of computer capacity and cost. Moreover, there are several difficulties in modeling the actual behavior. Frictional behavior between the pile surface and soil is an example of the difficulty.

Considering the situation at present, it may be a better method to model a soil-pile-superstructure system into the Penzien type model schematically shown in Figure 1(a). Interactive behavior between the pile and free



(a) Whole model (b) Separate model
Figure 1 **Soil-pile-structure model**

field is expressed by, so called, interaction spring or Winkler type spring. There exists, however, some problems to use this model.

A computer code developed for the particular field usually does not have a function required in the other field. Since researches for the structure and the ground have been developed independently, the same problem may occur. For example, a computer code developed for the structural behavior cannot have a function to solve the ground, especially soil liquefaction. Similarly, that developed for the ground behavior or liquefaction analysis does not have a function to deal with nonlinear behavior of structure. Therefore, analysis of the model shown in Figure 1(a) (called whole model hereafter) is not an easy task, especially when soil liquefaction is of interest.

One of the solutions on this problem is to deal with the problem as multiple-support problem. The soil-pile-structure system is separated into the soil (free field ground) and the pile-structure system. Response of the ground is computed first and its response is applied to the pile-structure system as multiple-support excitation problem. This method is called a separate method in this paper. Since soil and other systems are solved separately, important factors in each field can be considered relevantly. The only shortage is that the computer code for the analysis of the pile-structure system must be improved in order to consider multiple-support excitation, but it can be done easily when using the procedure introduced in this paper.

The applicability of the Penzien type method on the pile in the liquefied ground is examined through the analysis of the pile damaged during the 1995 Hyogoken-nambu (Kobe) earthquake.

MULTIPLE-SUPPORT EXCITATION THEORY FOR PILE ANALYSIS

Governing equation under multiple-support excitation

Difficulty of the multiple-support excitation analysis exists a treatment of inertia force. The inertia force can be evaluated by a product of the absolute acceleration and mass. Absolute acceleration is obtained by the sum of base acceleration and acceleration relative to base. Therefore, evaluation is difficult when there are more than one base motion. Clough and Penzien (1975) showed a method to solve it when different acceleration time histories are specified at the base by defining static displacements, which are displacements of free nodes under the condition that displacement at one support is specified keeping the displacements of other supports zero. Accordingly, this method requires many additional calculations from the ordinary procedure to solve equation of motion, beside the requirement that computer code must be revised largely.

In the analysis of the pile, however, definition of the inertia force is clear. One of the authors proposed an alternate method (Tanaka et al., 1983), in which acceleration time history at a referent point of the ground and displacement time histories relative to that point are required as input. This method also requires improvement of a computer code, but it is very small because governing equation is same with ordinary one except that there are some additional terms in the right hand side (external load term) only.

Equation of motion is expressed in terms of absolute displacement $\mathbf{u}^t$ as

$$\mathbf{M}\ddot{\mathbf{u}}^t + \mathbf{C}\dot{\mathbf{u}}^t + \mathbf{K}\mathbf{u}^t = \mathbf{0} \qquad (1)$$

where $\mathbf{M}$, $\mathbf{C}$ and $\mathbf{K}$ are mass, damping and stiffness matrices, respectively. Absolute displacement is divided into displacement at the reference point (rigid displacement) and displacements relative to the reference point.

$$\mathbf{u}^t = \begin{Bmatrix} \mathbf{u}_a^t \\ \mathbf{u}_b^t \end{Bmatrix} = \underbrace{\begin{Bmatrix} \mathbf{u}_a^R \\ \mathbf{u}_b^R \end{Bmatrix}}_{\text{Rigid}} + \underbrace{\begin{Bmatrix} \mathbf{u}_a \\ \mathbf{u}_b \end{Bmatrix}}_{\text{Relative}} \quad \begin{matrix} \text{Free nodes} \\ \text{Excited nodes} \end{matrix} \qquad (2)$$

Here subscripts a and b denote free and supported degrees of freedom, respectively. Since we deal with multiple-support excitation, relative displacements $\mathbf{u}_b$ are not zero, and this is the difference from the ordinary single-support excitation problem. Substitution of Eq. (2) into Eq. (1) yields

$$\begin{bmatrix} \mathbf{M}_a & 0 \\ 0 & \mathbf{M}_b \end{bmatrix} \begin{Bmatrix} \ddot{\mathbf{u}}_a + \ddot{\mathbf{u}}_a^R \\ \ddot{\mathbf{u}}_b + \ddot{\mathbf{u}}_b^R \end{Bmatrix} + \begin{bmatrix} \mathbf{C}_{aa} & \mathbf{C}_{ab} \\ \mathbf{C}_{ba} & \mathbf{C}_{bb} \end{bmatrix} \begin{Bmatrix} \dot{\mathbf{u}}_a + \dot{\mathbf{u}}_a^R \\ \dot{\mathbf{u}}_b + \dot{\mathbf{u}}_b^R \end{Bmatrix} + \begin{bmatrix} \mathbf{K}_{aa} & K_{ab} \\ \mathbf{K}_{ba} & K_{bb} \end{bmatrix} \begin{Bmatrix} \mathbf{u}_a + \mathbf{u}_a^R \\ \mathbf{u}_b + \mathbf{u}_b^R \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \end{Bmatrix} \qquad (3)$$

Equation of motion for free nodes is written by retrieving free nodes component from Eq. (2), resulting in

$$\mathbf{M}_a\ddot{\mathbf{u}}_a + \mathbf{C}_{aa}\dot{\mathbf{u}}_a + \mathbf{K}_{aa}\mathbf{u}_a = -\mathbf{M}_a\mathbf{I}_a\ddot{\mathbf{u}}_a^R - \mathbf{C}_{ab}\dot{\mathbf{u}}_b - \mathbf{K}_{ab}\mathbf{u}_b \quad (4)$$

Here, $\mathbf{I}_a$ is a three column matrix whose arguments are 1 for free degrees of freedom and 0 for other degrees of freedom in three-dimensional analysis. The terms that becomes zero under the rigid body displacement such as $\mathbf{K}_{aa}\mathbf{u}_a^R$ is eliminated same as in the ordinary procedure for single-support excitation problem. Equation (4) is the governing equation for the multiple-support excitation problem. There are two additional terms $\mathbf{C}_{ab}\dot{\mathbf{u}}_b$ and $\mathbf{K}_{ab}\mathbf{u}_b$ compared from the ordinary governing equations.

Difference between whole and separate models

Both the whole model and separate model gives the same equation of motion because they started from the same governing equation, Eq. (1). However, as discussed in the following, there is no guarantee that they give the same response. This comes from the uncertainty of the problem.

In the practical case to solve the equation of motion, the problem is not defined uniquely because input earthquake motion is given as scattered data. Therefore, there are infinite solutions depending on the assumption on the intermediate behavior between the times where input data is specified; there is no exact solution. In other words, one needs to assume the intermediate behavior when solving the equation of motion. One of the most acceptable assumptions is a piecewise linear interpolation of the input earthquake. However, this is not a practical assumption because deferential equation cannot be solved for multi-degrees-of-freedom systems. Therefore, many numerical integration schemes assume response behavior. Linear acceleration method, for example, assumes that the response acceleration changes linearly. Although this assumption is also easily acceptable, it is not a practical method because it has the stability problem. In the followings, therefore, Newmark's β method is employed as an example.

First, the whole model is investigated. The governing equation for the whole model is expressed in the incremental form as

$$\mathbf{M}d\ddot{\mathbf{u}} + \mathbf{C}d\dot{\mathbf{u}} + \mathbf{K}d\mathbf{u} = -\mathbf{m}^T d\ddot{\mathbf{u}}_b \quad (5)$$

where $\mathbf{m}^T = \mathbf{M}\mathbf{I}_b$. The Newmark's β method assumes response displacement and velocity at time t such that

$$\dot{\mathbf{u}}_t = \dot{\mathbf{u}}_{t-dt} + (1-\gamma)dt\ddot{\mathbf{u}}_{t-dt} + \gamma dt\ddot{\mathbf{u}}_t \quad (6a)$$

$$\mathbf{u}_t = \mathbf{u}_{t-dt} + dt\dot{\mathbf{u}}_{t-dt} + \left(\frac{1}{2}-\beta\right)(dt)^2\ddot{\mathbf{u}}_{t-dt} + \beta(dt)^2\ddot{\mathbf{u}}_t \quad (6b)$$

where dt is time increment, and β and γ are adjusting parameters and is set 0.25 and 0.5 in the subsequent numerical analysis.

Their increment can be evaluated as

$$d\dot{\mathbf{u}} = dt\ddot{\mathbf{u}}_{t-dt} + \gamma dt d\ddot{\mathbf{u}}$$
$$d\mathbf{u} = dt\dot{\mathbf{u}}_{t-dt} + \frac{1}{2}(dt)^2\ddot{\mathbf{u}}_{t-dt} + \beta(dt)^2 d\ddot{\mathbf{u}} \quad (7)$$

This Equation is solved with respect to velocity and acceleration increments as

$$d\ddot{\mathbf{u}} = \frac{1}{\beta(dt)^2}d\mathbf{u} - \frac{1}{\beta dt}\dot{\mathbf{u}}_{t-dt} - \frac{1}{2\beta}\ddot{\mathbf{u}}_{t-dt} \quad (8a)$$

$$d\dot{\mathbf{u}} = \frac{\gamma}{\beta dt}d\mathbf{u} + dt\left(1-\frac{\gamma}{2\beta}\right)\ddot{\mathbf{u}}_{t-dt} - \frac{\gamma}{\beta}\dot{\mathbf{u}}_{t-dt} \quad (8b)$$

Substitution of Eq. (8) into Eq. (1) yields

$$[\frac{\mathbf{M}}{\beta dt^2} + \frac{\gamma\mathbf{C}}{\beta dt} + \mathbf{K}]d\mathbf{u} = \mathbf{M}\left(\frac{1}{\beta dt}\dot{\mathbf{u}}_{t-dt} + \frac{1}{2\beta}\ddot{\mathbf{u}}_{t-dt}\right) + \mathbf{C}\left(\frac{\gamma}{\beta}\dot{\mathbf{u}}_{t-dt} - \left(1-\frac{\gamma}{2\beta}\right)dt\ddot{\mathbf{u}}_{t-dt}\right) - \mathbf{m}d\ddot{\mathbf{u}}_g \quad (9)$$

Displacement increment $d\mathbf{u}$ can be obtained by solving this equation

Equation (9) is a simultaneous equation with respect to displacement increment in the whole model analysis, and it includes degrees of freedom for both the pile-structure and the ground. On the other hand, since we consider the separate model schematically shown in Figure 1(b) under the multiple-support excitation. Then, degrees of freedom are separated into those of the structure and the ground, and same subscripts a and b are used to distinguish them. In addition, subscript to represent the time t and t-dt is needless to write because it is obvious. Finally, we obtain the simultaneous equation with respect to displacement increment of the pile-structure system as

$$\left(\frac{\mathbf{M}_a}{\beta dt^2} + \frac{\gamma}{\beta dt}\mathbf{C}_{aa} + \mathbf{K}_{aa}\right)d\mathbf{u}_a = -\frac{\gamma}{\beta dt}\mathbf{C}_{ab}d\mathbf{u}_b - \mathbf{K}_{ab}d\mathbf{u}_b + \mathbf{M}_a\left(\frac{1}{\beta dt}\dot{\mathbf{u}}_a + \frac{1}{2\beta}\ddot{\mathbf{u}}_a\right) + \mathbf{C}_{aa}\left(\frac{\gamma}{\beta}\dot{\mathbf{u}}_a - \left(1-\frac{\gamma}{2\beta}\right)dt\ddot{\mathbf{u}}_a\right) + \mathbf{C}_{ab}\left(\frac{\gamma}{\beta}\dot{\mathbf{u}}_b - \left(1-\frac{\gamma}{2\beta}\right)dt\ddot{\mathbf{u}}_b\right) - m\ddot{\mathbf{u}}_g \quad (10)$$

In the same manner, application of the Newmark's β method for the multiple-support excitation problem, Eq. (9), results in the simultaneous equation as

$$\left(\frac{\mathbf{M}_a}{\beta dt^2}+\frac{\gamma}{\beta dt}\mathbf{C}_{aa}+\mathbf{K}_{aa}\right)d\mathbf{u}_a$$
$$=-[\mathbf{C}_{ab}]\{d\dot{\mathbf{u}}_b\}-[\mathbf{K}_{ab}]\{d\mathbf{u}_b\}+\mathbf{M}_a\left(\frac{1}{\beta dt}\dot{\mathbf{u}}_a+\frac{1}{2\beta}\ddot{\mathbf{u}}_a\right)$$
$$+\mathbf{C}_{aa}\left(\frac{\gamma}{\beta}\dot{\mathbf{u}}_a-\left(1-\frac{\gamma}{2\beta}\right)dt\ddot{\mathbf{u}}_a\right)-m\ddot{\mathbf{u}}_g \qquad (11)$$

Left sides of Eqs (10) and (11) are identical, and a few terms in the right sides are different, which are summarized in the following

$$\text{Whole: } \mathbf{C}_{ab}\left(\frac{\gamma}{\beta}\dot{\mathbf{u}}_b-\left(1-\frac{\gamma}{2\beta}\right)dt\ddot{\mathbf{u}}_b-\frac{\gamma}{\beta dt}d\mathbf{u}_b\right) \qquad (12)$$

Separate: $-\mathbf{C}_{ab}d\dot{\mathbf{u}}_b$

From the procedure to derive the final simultaneous equation, it is clear that this difference comes from the different assumption on the response value. Quantities in the ground (excited nodes) are unknown and are predicted by the interpolation based on Newmark's β method in the whole model analysis, whereas they are known quantity in the separate model (multiple-support excitation) analysis.

Since simultaneous equations are different from each other between the whole analysis and separate analysis, it looks that response values are also different. However, if the same numerical integral algorism is employed for both the whole and separate analyses, response become identical because the same equation with whole model analysis is used to evaluate the ground response although they are calculated separately. In other words, $d\dot{\mathbf{u}}_b$ in Eq. (12) is a result of the upper line of the same equation, which can be easily understood when looking at Eq. (8b). This is confirmed through the numerical analyses in the following sections. This indicated that the same numerical integral scheme is recommended to be used in the separate model analysis.

EXAMPLE OF DAMAGE TO PILE

A case study is made in order to investigate the applicability of the model such as in Figure 1, and to confirm the formulation of multiple-support excitation. The pile damaged during the 1995 Hyogoken-nambu (Kobe) earthquake (Committee on Building Foundation Technology against Liquefaction and Lateral Spreading, 2000) is chosen as an example.

The building is located in the Fukaehama reclaimed land and 350 meters far from the shoreline.

The structure is a three story steel building supported by Type A-PC piles with 28 m long and 40 cm exterior diameter. The ground is filled with decomposed granite usually called "Masado" in Japanese. It was developed between 1964 and 1970.

Among the damaged piles, three piles were investigated by means of borehole camera (JGS, 2004). Observed damage pattern and soil profiles are shown in Figure 2. Cracks were observed in the fill under the water table.

SIMPLIFIED ANALYSIS

Damage of this pile was analyzed based on

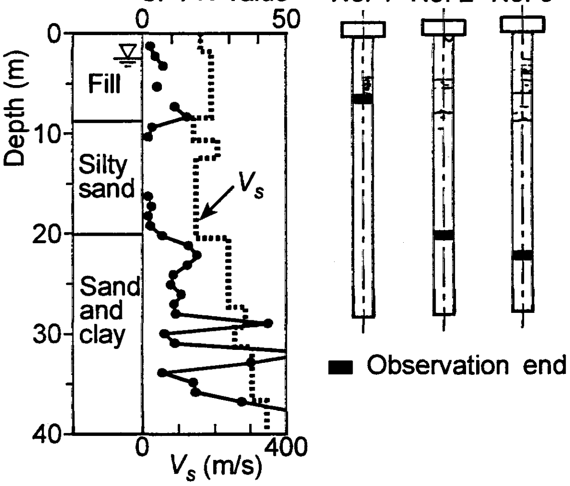


Figure 2: **Soil profiles and damage to piles**

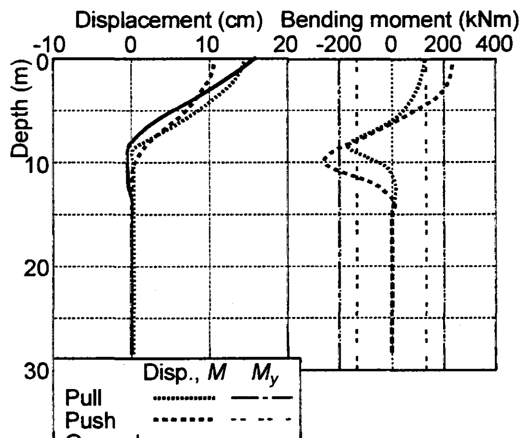


Figure 3: **Result of simplified analysis**

the AIJ method (AIJ, 2001) by a committee of Japanese Geotechnical Society (JGS, 2004). Liquefied layer is first evaluated from the SPT-*N* value and liquefaction strength. Then displacement of the ground is evaluated under the assumption that shear strain in the liquefied layer is 2% (AIJ, 2001), and that in other layer is zero. Bending moment of the pile is evaluated by the seismic deformation method. Spring constant and ultimate strength of the interactive spring are evaluated from the coefficient of subgrade reaction and ultimate subgrade reaction based on AIJ (2001). Difference of the pile behavior under push and pull type axial force is taken into account in the analysis. Bending moment diagram under the action of ground displacement only is shown in Figure 3. Maximum bending moment exceeds yield moment at about GL-9 m, which agrees with observed behavior. Bending moment at the pile top also exceeds yield moment, and cracks in the No. 2 pile seem to correspond it.

This simple analysis seems to succeed in expressing the damage to pile. However, there is no proof that strain of the liquefied layer is 2%. Therefore, we cannot guarantee that this method is always successful.

MODELING FOR ANALYSIS

Both finite element method and seismic deformation method are frequently used in the analysis of soil-pile system in the static analysis. The latter method is extended into multiple-support excitation problem, and is used in this paper because this method is more practical than FEM. It has an advantage that different code can be used for the analysis of the ground and structure as discussed in the introduction. Separate model based on multiple-support excitation theory has a big advantage in these points.

Analyzed model and the ground properties are shown in Figure 4. Dynamic deformation characteristics are shown in Figure 5, which is obtained by laboratory test of the in-situ samples.

Spring constant of the interactive spring is computed from the coefficient of subgrade reaction defined in JRA (2002). Young's modulus is evaluated from SPT-*N* value as

$$E_0 = 2800N \ \text{kN/m}^2 \quad (13)$$

Then coefficient of subgrade reaction is defined

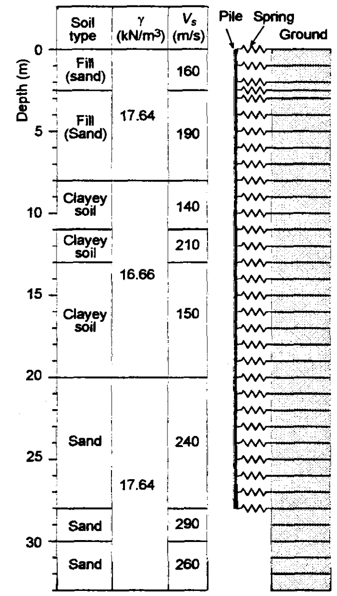


Figure 4: **Structural model and soil parameter**

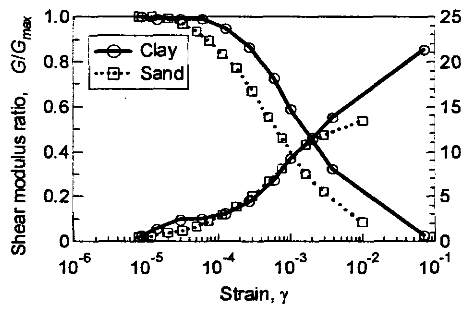


Figure 5: **Dynamic deformation characteristics**

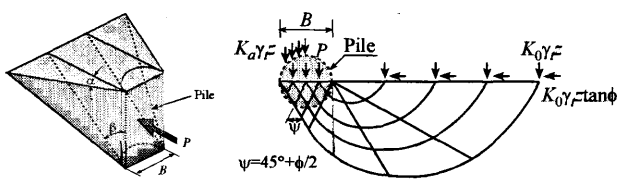


Figure 6 **Mechanisms at ultimate state**

to be

$$k_H = k_{h0}(B_H / 0.3)^{-3/4} \quad (14)$$

where B_H is pile width and

$$k_{H0} = \alpha E_0 / 0.3 \quad (15)$$

Here α is an adjusting parameter and is 2.0 for the analysis of earthquake behavior.

Ultimate subgrade reaction is computed based on Kishida and Nakai (1979). They considered two mechanisms shown in Figure 6 for near surface and deep depths. We use the latter mechanism for evaluating the ultimate subgrade reaction partly because there is no significant difference for both mechanisms. Then, it is evaluated from

$$p_y = 3K_p \gamma_t' z \qquad \text{Sand} \qquad (16)$$

$$p_y = 9c_u \qquad \text{Clay} \qquad (17)$$

where $K_p = \tan^2(45+\phi/2)$. Internal friction angle is set 30 degrees for sand and cohesion of the clay is set 10 kPa.

Hyperbolic model is used for the nonlinear characteristics.

A tri-linear model is used for the moment-curvature relationship of the pile. Here, yield moment M_y is 129.08 kNm and ultimate moment M_u is 183.43 kNm. Elastic behavior is assumed under the axial and shear forces.

The earthquake motion observed at the Higashi-Kobe Bridge is used as outcrop motion at the engineering seismic base layer, which is shown in Figure 7. Although we made earthquake response analysis up to 40 seconds, but as the response after 20 seconds was not predominant; response until 20 seconds is shown in the time history.

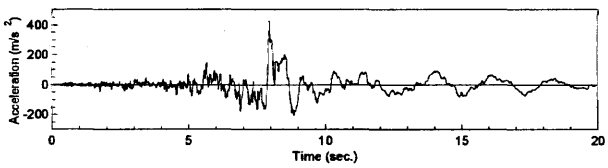


Figure 7: **Input earthquake motion**

COMPUTER CODES AND METHOD OF ANALYSIS

Several total stress and effective stress computer codes are used in the analysis of the ground in order to evaluate various factors affecting the dynamic response of the pile.

MDM model

A computer code using MDM model (Kumazaki et al., 1998) is a total stress, one-dimensional, earthquake response analysis code. The MDM model has characteristics that dynamic deformation a characteristic can be modeled with high accuracy as shown in Figure 8.

STADAS

STADAS (Yoshida, 1993) is a general purpose computer code for soil and soil-structure interaction behavior and both total and effective stress analyses are possible. Among the various constitutive models that the code has, a model which can simulate dynamic deformation characteristics perfectly (Yoshida et al., 1990) is used in the total stress analysis. Since the simulation of the dynamic deformation characteristics is perfect, the comparison such as Figure 8 is needless to show.

SD model

A multi-dimensional earthquake response analysis code which uses SD model (Cubrinovski, and Ishihara, 1998), a elastic-plastic constitutive model, is used in the effective stress analysis. Figure 9 shows result of simulation of the liquefaction-strength, and

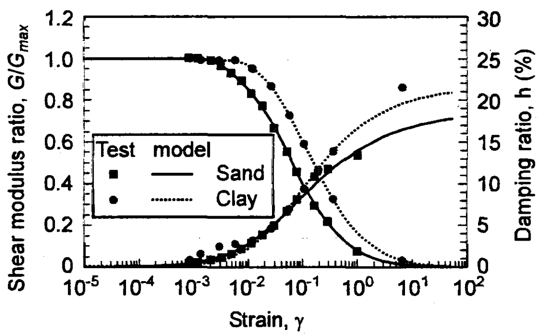


Figure 8: Simulation by MDM model

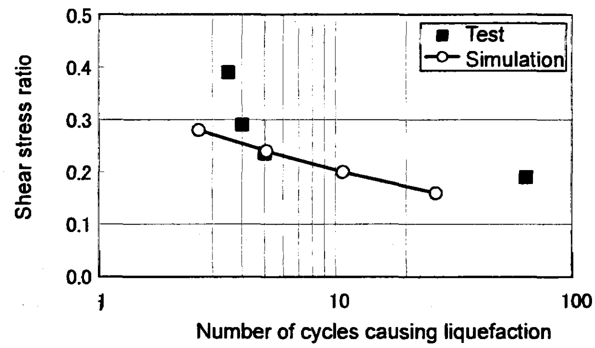


Figure 9: Simulation of liquefaction strength

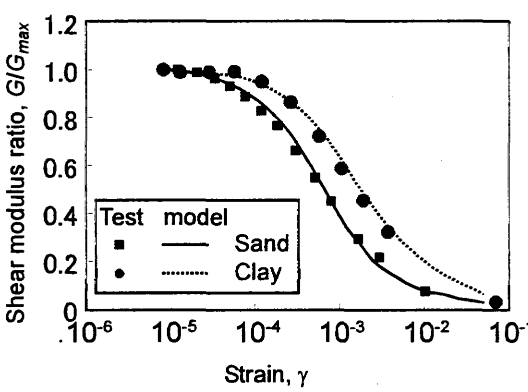


Figure 10: **Simulation by SD model**

Figure 10 is a comparison of shear modulus.

Summary of method of analysis

In summary, total stress analysis is carried out by STADAS and the code with MDM model, and effective stress analysis is made by the code with SD model. They are referred as MDM, STADAS and SD, respectively, in the following. Since it is clear and obvious that only the fill ground has susceptibility of liquefaction, only this layer is treated as two phases material in the effective stress analysis.

Response of the pile under the multiple-support excitation is computed by general purpose computer code STADAS (Yoshida, 1993). Here, displacement and velocity computed by each computer codes are applied as multiple-support excitation in the pile-structure model.

Before conducting the multiple-support excitation calculation, accuracy of the method is examined by comparing with the whole analysis by STADAS. In the whole analysis, element stiffness matrices for the interactive springs are asymmetric so that the structural behavior is affected by the ground motion, but the ground motion is not affected from the structure. Both results agree with each other by more than 6 digit. Considering the round error and cut-off error included in the output of the ground response for the use of multiple support excitation, this agreement indicates both analysis agrees perfectly as the theory in the preceding predicted.

GROUND RESPONSE

MDM model

Although MDM model is a total stress method, but liquefaction analysis is said to be possible (Kumazaki and Ueda, 1999), although naturally excess porewater pressure is not output. Stiffness proportional damping is used velocity proportional damping; coefficient against stiffness is 0.0008. Time histories at the ground surface are shown in Figure 11 and maximum response distributions are shown in Figure 12.

STADAS

Time histories are shown in Figure 13, and maximum response distribution is shown in Figure 14. As damping is known to affect displacement significantly (Yoshida, 2003), velocity proportional damping is chosen as parameters. Coefficient against stiffness proportional damping is chosen so that damping ratio for the first mode is 0.5, 1.0, and 3.0 %. As shown in Figure 13, effect of damping on acceleration is small, but that on displacement is very large. Since the predominant period of the ground is 0.65 second, MDM model used 0.17% and SD model used 1% damping. Therefore, the result under 1% damping is used in the subsequent analysis by STADAS.

SD model

Same as previous two analyses, stiffness proportional damping is used, and, as described above, damping ratio against the first mode is about 1%. Time histories at the ground surface are shown in Figure 15, and maximum

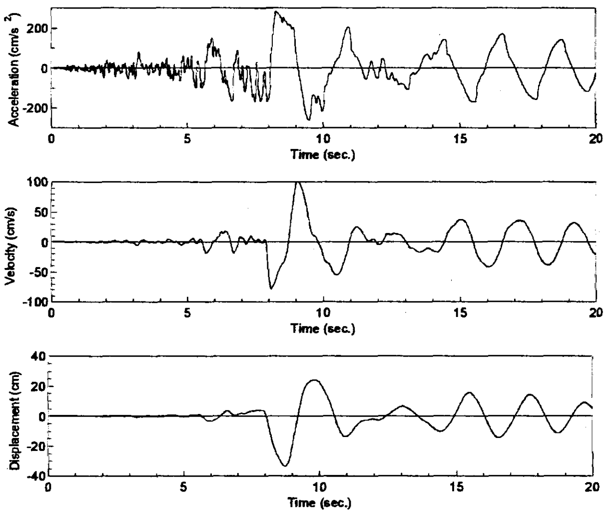


Figure 11 **Time histories at ground surface**

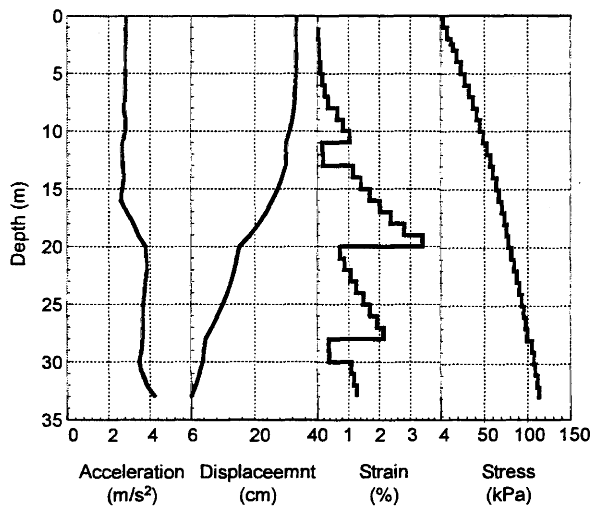


Figure 12: **Maximum response by MDM**

response distribution is shown in Figure 16. Since this analysis is an effective stress analysis, excess porewater pressure is shown in Figure 16, which indicates that liquefaction occurred in the fill.

BEHAVIOR OF PILE

Response of the ground obtained in the preceding section is applied to the pile through the interaction spring as multiple-support excitation problem.

Maximum displacement distributions obtained by three codes are summarized and associated bending moment diagram of the pile

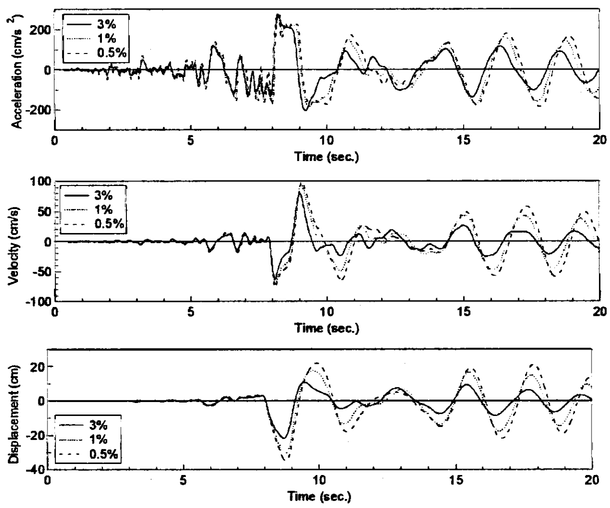


Figure 13 **Time histories at ground surface**

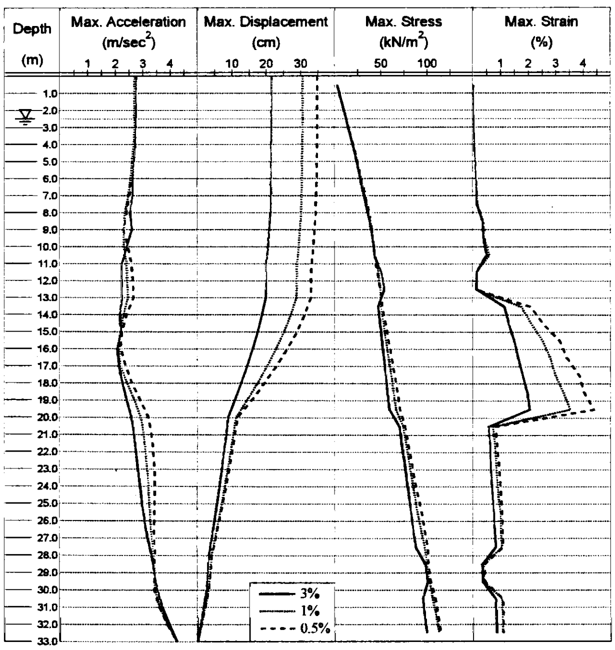


Figure 14: **Maximum response by STADAS**

is shown in Figure 17. Here, maximum displacement occurs almost the same time. The ground displacement by two total stress analyses are very similar to each other, which may come from the fact that stress-strain model is similar to each other. The difference of absolute value comes from the difference of damping ratio; MDM uses 0.17% and STADAS uses 1%.

Large bending moment appeared at around GL-20 m, where stiffness changes significantly. A kink shape of the displacement distribution here caused large bending moment. Unfortunately, these bending moment distributions seem not to agree with damage in the pile. No damage is expected at GL-8 m and significant damage is expected at GL-20 m by the analysis. The disagreement at GL-8m seem to come from the total stress analysis in which stiffness reduction associated with excess porewater generation is not considered. However, disagreement at GL-20 m is

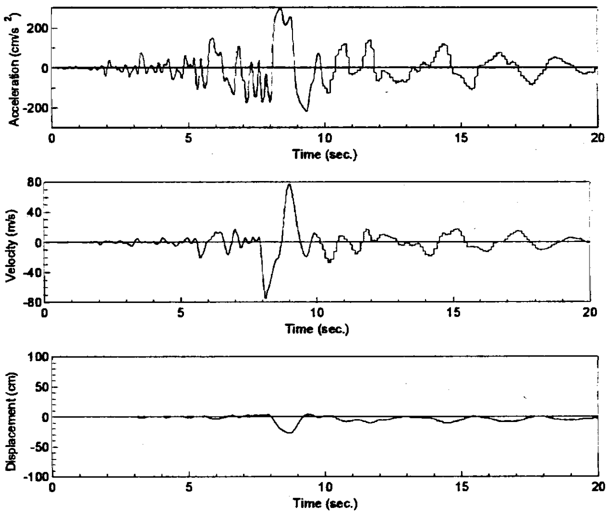


Figure 15: **Time histories at the ground surface**

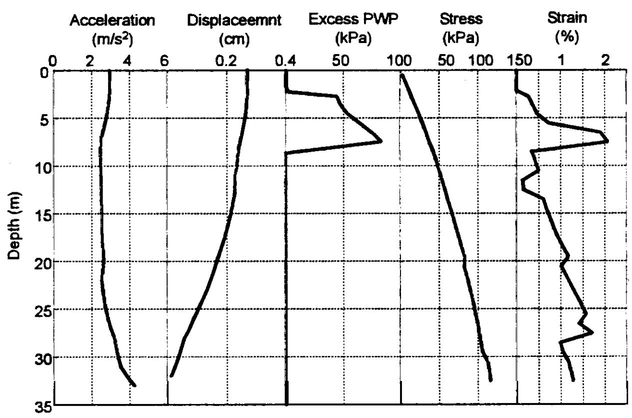


Figure 16: **Maximum response by SD model**

investigated with care. Investigation by borehole camera was stopped at about GL-20 m. Usually, investigation by borehole camera is made as deep as possible. Therefore, there is possibility that pile is significantly damaged around this depth so that borehole camera cannot be installed below here. Anyway, the total stress analysis cannot explain the pile damage associated by soil liquefaction.

Result of the effective stress analysis by SD model shows smaller bending moment at GL-20 m and larger bending moment in the liquefied layer. Bending moment between GL-2.5 m and 8 m, which correspond to liquefied layer, exceeds ultimate moment. An example of moment-curvature relationships of the pile is shown in Figure 18, which suggests significant damage to pile. Force-strain relationship of the interactive spring at the same location is shown in Figure 19; nonlinear behavior of spring is not significant.

The simplified analysis based on the design specification shown in the previous section showed 16 cm displacement at the ground surface, whereas those by earthquake response analysis are the order of 30 cm. This difference significantly affects the result of analysis. This indicates importance to evaluate the displacement of the ground by earthquake response analysis. Comparison of the result by total and effective stress analyses also indicates importance of prediction of ground displacement.

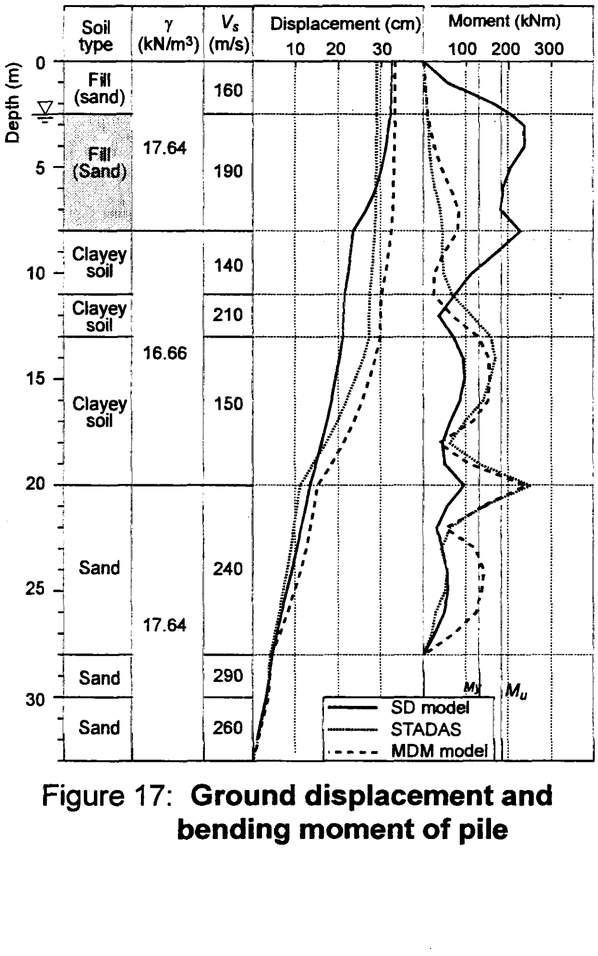


Figure 17: **Ground displacement and bending moment of pile**

EFFECT OF VARIOUS FACTORS

Analysis based on the interactive spring between the pile and free field ground has been used in practice as, for example, Penzien type analysis and its improvement. Considering it, effect of various factors on the pile response is investigated.

Importance of accurate prediction of the ground response is already discussed in the preceding. Importance of damping in predicting the displacement is also discussed before. In addition to these factors, three cases are investigated. Results are summarized in Figure 20. Calculation is carried out by STADAS, and "Target" in the figure is the result of STADAS: agreement with this is discussed in the followings.

The term "Displacement" is a result where

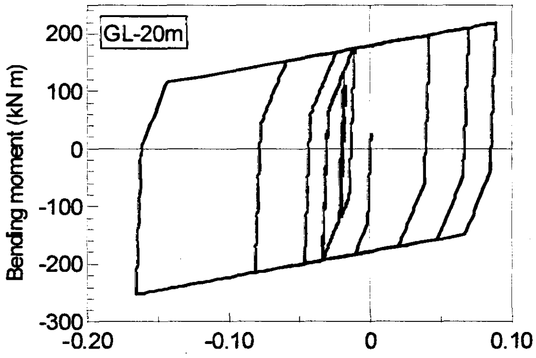


Figure 18: **Moment-curvature relationship at GL-20m.**

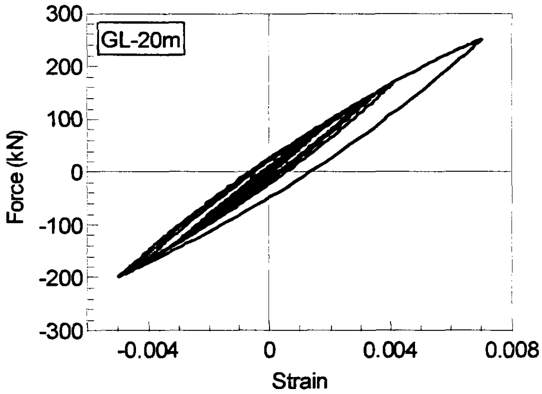


Figure 19: **Behavior of interactive spring**

only displacement time histories are applied and terms related to the relative velocity is not considered. The result agrees with "Target" at deep depths, but significant difference appears at depth shallower than GL-10 m. This indicates that both displacement and velocity input is necessary for multiple-support excitation problem.

The term "No inertia" is a result where inertia force of the pile is neglected. The result hardly changes from the "Target" behavior. Therefore, as frequently mentioned, underground lineral structure such as pile behaves associated with ground deformation, and it does not response by inertia force.

As indicated in the effective stress analysis, fill under the water table liquefies. When liquefaction occurs or excess porewater develops, stiffness of spring constant may reduce. Considering the effect, spring constant is reduced to 1/10 in the third analysis, which case is shown as "Liquefaction". Again, no significant change from "Target" is observed. Therefore, effect of soil liquefaction must be considered from the beginning, i.e., analysis of the ground.

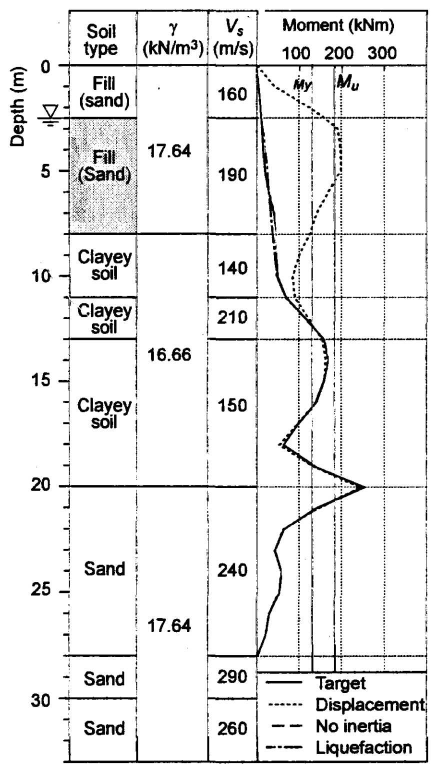


Figure 20: **Effect of various factors**

CONCLUDING REMARKS

Method to analyze a pile subjected to ground deformation when liquefaction occurs is investigated. The Penzien type model in which pile and free field ground is connected by interactive spring is shown to be applicable. The separate method, in which pile behavior is solved as multiple-support excitation problem is a convenient tool to use this model, because characteristic behavior in each field such as liquefaction and nonlinear behavior of structural members can be taken into account. Through various investigations, the following lessons can be obtained.

1. In order to obtain accurate result, each component must be evaluated relevantly. In the practical analysis, however, different design specification sometimes shows different equation for, for example, coefficient of subgrade reaction, ultimate subgrade reaction, etc. Moreover, method to consider excess porewater pressure is also different depending on design specification. Research papers have indicated that their method succeeded to explain the actual behavior. However, looking at the case study in this paper, one should be careful. There is few jobs which investigated sensitivity of parameters by comparing various factors. This kind of works is encouraged.
2. Evaluation of ground displacement relevantly is one of the most important. As shown in the example, factors that were not interested such as damping term are shown to affect. Investigation on damping and stress-strain model considering the confining stress dependency of soil is encouraged, too.
3. Not only displacement but also velocity input is necessary for multiple-support excitation problem. Velocity input is not necessary only when there is no damping in the interactive spring.
4. Effect of Inertia working on the pile is small and can be neglected. It looks as static analysis is valid, but, as shown in the preceding article, it is not true; velocity input is also important.

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