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Unified analysis considering pile groups and superstructures

Analyse unifiée considérant les groupes de pieux et les superstructures

Jin Oh Won & Sangeeom Jeong
Department of Civil Engineering, Yonsei University, Seoul, 120-749, Korea

Cheol Ju Lee
Engineering & Construction Division, Samsung Heavy Industries, Co, Ltd, Seoul, Korea

ABSTRACT

A unified analysis procedure is developed for the efficient and accurate design and analysis of bridge pier structures. The nonlinear pile group analysis method based on structural stiffness theory, was adopted for considering the pile-soil-pile and pile-cap interactions, and soil nonlinearity. It extended and then combined with the analysis routine of superstructure. Among the components of bridge pier structures, a pile cap is modeled with four-node flat shell elements and a pier with three-dimensional beam element and the pile-soil-pile interaction is considered by p-multipliers. A nonlinear analysis method is proposed with an incremental and iteration technique in this study. The proposed method for a bridge pier subjected to both axial and lateral loads is verified by comparing with existing computer codes (Group 5.0, FBPier 3.0). Through the comparative studies, it is found that the present method gives similar results with the other analysis codes about lateral displacements. The proposed method is capable of predicting the behavior of a bridge pier installed in nonlinear soils on the various loading conditions.

RESUMÉ

Une procédure d’analyse unifiée est développée pour une conception précise et efficace ainsi que l’analyse de structures de pile de pont. La méthode non-linéaire d'analyse de groupe de pieux basée sur la théorie structurale de rigidité, a été adoptée pour considérer les interactions de pieu-sol-pieu et de pieu-chapeau, et la non-linéarité de sol. Il a été étendue puis combinée avec le processus d’analyse de la superstructure. Parmi les composants de structures de pile de pont, un pieu-chapeau est modelé avec des éléments plats de carcase de quatre noeuds, une pile avec des éléments tridimensionnels de poutre et l’interaction de pieu-sol-pieu est considérée par des p-multiplieurs. On propose une méthode non-linéaire d’analyse avec une technique d’accroissements et d’itérations dans cette étude. La méthode proposée pour une pile de pont soumise aux charges axiales et latérales est vérifiée en comparant avec des codes machine existants (groupe 5.0, FBPier 3.0). A travers les études comparatives, on constate que la méthode actuelle donne des résultats semblables aux autres codes d'analyse au sujet des déplacements latéraux. La méthode proposée est capable de prévoir le comportement d'une pile de pont installée dans des sols non-linéaires sur les diverses conditions de charge.

1 INTRODUCTION

Pile groups, commonly used to support bridge structures, are frequently subjected to lateral loadings. Two major sources, governing the behavior of pile groups are: (1) pile-soil interaction; (2) pile-cap interaction. The pile-soil interaction is nonlinear characteristic and then a load transfer curve method (Reese, 1977) is suitable for considering this effect rather than elasticity theory based methods (Doglas and Davis, 1964). The pile-cap interaction was mainly influenced by individual pile head stiffness, the geometric arrangement of piles and the condition of pile head fixity. The procedure for batter-pile foundation analysis using structural stiffness method (Reese, 1970) is a useful and valuable instrument of design, and provides a method of investigation the relative importance of various foundation parameters. In a closely-spaced pile group, the behavior of the individual piles within a group is best modeled based on the p-multiplier concept, suggested by Brown et al. (1988). Experimental data on p-multiplier, f_p (McVay et al. 1998; Rollins et al. 1998; Ruesta et al., 1997) are considerable.

This existing nonlinear analysis procedure is almost sufficient to the design and analysis of a pile group in view of geological engineering. By the way also in the design of bridge pier, the effects of structural and soil behavior as well as complex pile-soil-pile interaction should be included. However, bridge pier can not be modeled by using the existing nonlinear pile group analysis codes (Jeong et al., 2003) based on structural stiffness method as well as a commercial package Group 5.0 (Reese and Wang, 2000) for in which a pile cap is assumed as a rigid body.
2 METHOD OF ANALYSIS

2.1 Structural system

Figure 1 illustrates a general bridge pier and ground pile system and its analytical model in this study. The individual pile is incorporated to structure analysis as a form of a pile-head stiffness and its analytical model in this study. In this method, a pile member is described as a series of beam-column elements with discrete springs (t-z, q-z and p-y curves) to present the nonlinear soil support condition. For representing a pile cap and a pier, three-dimensional finite elements are used. As shown in Figure 1, a pile cap and a pier are modeled by 4 node flat shell element and a three-dimensional beam element, respectively.

In this study a nonlinear analysis technique is proposed to adjust individual pile head stiffness appropriated at specified displacements.

2.2 Pile cap modeling

Trials have been done to consider pile-cap interaction as well as to consider the stiffness of a pile cap. Consequently a plate element was used as a pile cap in several analysis methods (Clancy et al., 1993; Zhang et al., 2000; Kittiyodom et al., 2002). These methods have limitations, however, that the horizontal behavior of a pile cap can not be considered because horizontal degrees of freedom (x and y direction) are excluded. This limitation can be solved by using a flat shell element (Choi et al., 1996). In this study 4-node flat shell element was developed by combining a Mindlin’s plate element and a membrane element with torsional degrees of freedom as shown in Figure 2. The element possesses six degrees of freedom per node that, in addition to improvement of the element behavior, permit an easy connection to other six-degrees-of-freedom per node elements such as beams or folded elements. The stiffness matrix of an n-node flat-shell element in local coordinate system is constructed as followings:

$$K_{shell} = \begin{bmatrix} K_{pdown} & 0 \\ 0 & K_{mshell} \end{bmatrix}$$

(1)

Figure 1. Modeling of a pile group

Figure 2. Flat shell element

2.3 Extension a pile head stiffness matrix to a beam stiffness matrix

Fig. 3 shows the general structure coordinate system (X, Y, Z) and the pile coordinate system (u, v, w). Eq. (2) and Eq. (3) represent equilibrium equations at individual pile heads as a matrix form.

$$\begin{bmatrix} c_i & 0 & 0 & 0 & 0 & 0 \\ 0 & c_i & 0 & 0 & c_i & 0 \\ 0 & 0 & c_i & 0 & -c_i & 0 \\ 0 & 0 & 0 & c_i & 0 & 0 \\ 0 & 0 & -c_i & 0 & c_i & 0 \\ 0 & 0 & 0 & 0 & c_i & 0 \end{bmatrix} \begin{bmatrix} \delta_i \\ F_j \end{bmatrix} = \begin{bmatrix} M_i \\ M_i \\ M_i \\ \alpha_i \\ \alpha_i \\ \alpha_i \end{bmatrix}$$

(2)

$$S_i \delta_i = F_j$$

(3)

where, $S_i$ is an individual pile head stiffness matrix, $\delta_i$ a displacement or rotation, and $F_j$ force or moment at the $i$th pile head. Each component ($c_i$-$c_6$) of a pile head stiffness matrix can be estimated by single pile analysis subjected to axial and lateral forces and moments with typical boundary conditions (Reese et al., 1970).

Generally in three dimensional finite element analyses, individual piles are modeled like beams. So Eq. (2) extended to an equilibrium equation of a beam element that has node 1 and node 2.

2.4 Nonlinear analysis technique

The load-displacement curves at each pile head are nonlinear characteristics and the pile head stiffness ($c_i$-$c_6$) is decreased as the pile head movement increases. For this reason a nonlinear analysis technique is needed to adjust pile head stiffness $c_i$-$c_6$ in Eq. (2) at a final displacements or rotations.

In this study, a mixed load increment and iteration method is suggested. Fig. 4 shows a calculating process of stiffness at $i$th load increment. External forces are divided by $N$, and ($k_{ij}$) means the stiffness at $i$$th$ load increment and $j$th iteration.

In each load increment, tangential slope is adopted in $j=1$ and secant modulus in $j>1$ for the stiffness of pile head, which are expressed as Eq. (4) and Eq. (5), respectively.

$$k_{ij} = \frac{df((u_j, u_{j-1})}{du} (j = 1)$$

(4)

$$k_{ij} = \frac{f((u_{j+1}, u_{j-1})) - f((u_{j-1}, u_{j-1}))}{(u_j)_{j-1}} (j > 1)$$

(5)

$$u_{i+1} = u_{i-1} + \Delta u_j$$

(6)

where, ($u_{i+1}$) is an accumulated final displacement at previous load increment, ($u_i$) an accumulated displacement at $i$$th$ load increment and $j$th iteration.

At each load increment, displacements ($\Delta u_i$) are calculated through the structural analysis and then accumulated displac-
ments \( (u_i) \) are estimated using Eq. (6). If the convergence crite-
ria, \( \Delta u_{i-1} \div \Delta u_i \div \varepsilon \) is satisfied, the accumulated final displacements
\( (u_i) \) are calculated and goes to next load increment. This process
iterate until the load increment number reaches \( N \). In the structure
analyses, the tangential slope \( (df(u)/du) \) and load \( (f(u)) \) are
estimated using cubic spline method.

3 COMPARISON WITH OTHER COMPUTER CODES

For verification, the present method was compared with other
computer codes: Elastic displacement method, Group 5.0 and
FBPier 3.0. A schematic diagram of a 2×2 pile group is shown in
Figure 5. It consists of a pier, a pile cap and four identical vertical
piles which are spaced at 3m (=6D, D: pile diameter).
The pile spacing is large enough to neglect p-multipliers. The four
piles are 10m in embedded length, 0.5m in diameter and 147,264kN-m^2
flexural rigidity (EI). The thickness of a pile cap is 0.75m and pile head conditions are fixed. The pier is 10m in
length, 1m in diameter and 1,963,600kN-m^2 axial
rigidity (EI). The details of structural properties are shown in Table 1.

3.1 Linear soil

To verify structure analysis routine, the present method was
compared with the results of three different programs for a pile
group installed in linear soil. The same axial soil spring con-
stants, 2,000kN/m² are used along the pile depth. At pile toe end
bearing spring constant is 10,000kN/m² and its tension part is
neglected. The constants of horizontal soil springs are increased
from 0 to 100,000kN/m² along the pile depth. The pile group is
subjected to only lateral load, 1000kN at pier top, point A.

An Elastic displacement method can not consider the soil
reaction modulus of individual piles, so that individual pile head
stiffness \( (c_i) \) used in the present method. For Group
program only axial pile head stiffness \( (c_i) \) is used as an estimated value by present method.

The same lateral forces were distributed to each pile head in
all the analysis methods. The predicted axial forces and mo-
ments at the pile head by each method had some difference.
FBPier and present method considering the stiffness of a pile
cap predicted larger axial forces and smaller moments at the pile
head rather than Group and elastic method.

Figure 5. Schematic diagram of a piled pier

Table 1. Material properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus (EI)</td>
<td>40,000 MPa (5,805,600 psi)</td>
</tr>
<tr>
<td>Shear modulus (Ei)</td>
<td>31,435</td>
</tr>
<tr>
<td>Poisson’s ratio (ν)</td>
<td>0.18</td>
</tr>
<tr>
<td>Thickness (t)</td>
<td>0.75 m (2.46 ft)</td>
</tr>
<tr>
<td>Area (A)</td>
<td>0.19635 m^2 (2.11 ft^2)</td>
</tr>
<tr>
<td>Elastic modulus (EI)</td>
<td>40,000 MPa (5,805,600 psi)</td>
</tr>
<tr>
<td>Shear modulus (Ei)</td>
<td>12,842,920 kN/m^2</td>
</tr>
<tr>
<td>Poisson’s ratio (ν)</td>
<td>0.18</td>
</tr>
<tr>
<td>Thickness (t)</td>
<td>0.75 m (2.46 ft)</td>
</tr>
<tr>
<td>Area (A)</td>
<td>0.19635 m^2 (2.11 ft^2)</td>
</tr>
</tbody>
</table>

Table 2. Displacements and forces of pile groups

<table>
<thead>
<tr>
<th>Displacement/ Force</th>
<th>Elastic method</th>
<th>Group 5.0</th>
<th>FBPier 3.0</th>
<th>Present method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral displ. at pier A (mm)</td>
<td>-</td>
<td>553.4</td>
<td>510.9</td>
<td></td>
</tr>
<tr>
<td>Axial displ. at pier A (mm)</td>
<td>-</td>
<td>-9.2</td>
<td>-8.5</td>
<td></td>
</tr>
<tr>
<td>Lat. displ. at 1,3 pile head (mm)</td>
<td>57.4</td>
<td>68.0</td>
<td>52.9</td>
<td></td>
</tr>
<tr>
<td>Axial displ. at 1,3 pile head (mm)</td>
<td>-46.6</td>
<td>-51.2</td>
<td>-51.4</td>
<td></td>
</tr>
<tr>
<td>Rotation angle of 1,3 pile head (°)</td>
<td>-0.031</td>
<td>-0.028</td>
<td>-0.030</td>
<td></td>
</tr>
<tr>
<td>Lat. force at 1,3 pile head (kN)</td>
<td>250</td>
<td>250</td>
<td>250</td>
<td></td>
</tr>
<tr>
<td>Axial force at 1,3 pile head (kN)</td>
<td>-915</td>
<td>-1000</td>
<td>-1080</td>
<td></td>
</tr>
<tr>
<td>Moment at 1,3 pile head (kN-m)</td>
<td>-1128</td>
<td>-968</td>
<td>-830</td>
<td></td>
</tr>
</tbody>
</table>

3.2 Nonlinear soil

At first axially and laterally loaded single pile behavior was
predicted by present method considering nonlinear load transfer
curves and then compared with the results of FBPier and LPile.
It was found that the predictions were almost the same each other.

After single pile analyses, a series of group pile analyses was
performed on the same pile group configuration as shown in
Figure 5. Individual piles were assumed to be as PHC (Preten-
sioned spun High strength Concrete) driven piles installed in
sandy soils. The soil around individual piles was modeled with
nonlinear load transfer curves. The types of soil is limited to
three sandy soils which are loose, medium and dense sand ac-
dording to density, of which SPT values were 7, 20 and 40, re-
spectively. The axial load transfer curves (t-z, q-z curves) were
estimated using equation of McVay et al. (1999) suitable for
driven piles, and the lateral load transfer curves (p-y curves)
were used as a API model (O’Neill 1984). The properties used
for estimating axial and lateral load transfer curves are shown in
Table 3.

External loads were applied on the pier top. Axial loads
were adopted as 1000kN, 3000kN, and 5000kN because linear
behavior had been seen until 1000kN and at 5000kN yielding occurred and 3000kN was a mean value of them. Lateral loads were applied from 100kN to 600kN by adding 100kN on a condition of axial load applied.

The predicted lateral and axial displacements at the pier top with increasing lateral loads are shown in Figure 6. In this paper only the cases of the dense sand on fixed head condition are described due to the paper limitation. As shown in Figure 6, present method gave similar results with FBPier about both the settlements and lateral displacements at the pier top.

### Table 3. Properties used for estimating load transfer curve

<table>
<thead>
<tr>
<th>Contents</th>
<th>Loose sand</th>
<th>Medium sand</th>
<th>Dense sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>N value</td>
<td>7</td>
<td>20</td>
<td>40</td>
</tr>
<tr>
<td>Ultimate skin friction, Gt (kPa)</td>
<td>14</td>
<td>40</td>
<td>80</td>
</tr>
<tr>
<td>Initial shear modulus, Gi (kPa)</td>
<td>6000</td>
<td>8000</td>
<td>10000</td>
</tr>
<tr>
<td>Poisson’s ratio, ν</td>
<td>0.35</td>
<td>0.32</td>
<td>0.30</td>
</tr>
<tr>
<td>Ultimate bearing capacity, Qf (kN)</td>
<td>549.8</td>
<td>1570.8</td>
<td>3141.6</td>
</tr>
<tr>
<td>Internal friction angle (°)</td>
<td>26.6</td>
<td>31.8</td>
<td>38.0</td>
</tr>
<tr>
<td>Unit weights (kN/m3)</td>
<td>14.8</td>
<td>16.6</td>
<td>20.0</td>
</tr>
</tbody>
</table>

### Figure 6. Comparison of prediction results of pile groups subjected to both axial and lateral loads (Fixed head condition)

As shown in Figure 6, the lateral displacements at the pier top in cases of axial load 0, 1000, and 3000kN increased almost linearly with increasing lateral loads, but in case of axial load 5000kN it increased more rapidly. The settlements at the pier top also affected both axial and lateral loads. In cases of axial loads 1000, 3000kN, the effects of lateral load on the settlement were negligible, but in case of axial load 5000kN the settlement yielded over lateral load 200kN and then it increased rapidly. It is because the axial stiffness (c3) of individual pile was decreased by large axial force and then sudden settlement and rotation of a pile cap occurred.

### 4 CONCLUSIONS

In this study the theory of batter-pile foundation analysis suggested by Reese (1970) is extended for combining with the superstructure analysis routine developed by three-dimensional finite element technique. A unified analysis procedure of a bridge pier structures is developed by considering the pile-soil-pile and pile-cap interactions, and soil nonlinearity. A pile cap is modeled by 4-node flat shell element and a pier is modeled using 3-dimensional beam element. A mixed load increment and iteration method is suggested for considering nonlinear pile-cap interaction.

A series of analyses have been conducted for a pile foundation subjected to lateral loads at the pier top. In linear soils, elastic displacement method, Group and present method gives similar results of lateral displacement at the pile head, and FBPier gives rather larger values. In nonlinear soils, it is found that present method is capable of predicting the behavior of a bridge pier in nonlinear soils on the various loading conditions through the comparison study with FBPier.

### REFERENCES


