NONLINEAR STIFFNESS MATRIX MODELING FOR COMPLEX PILE GROUP FOUNDATION OF THE ANCHORAGE PORT ACCESS BRIDGE

B. Y. Choy¹, C. C. Tsai², and P. Meymand³

ABSTRACT

This paper presents a rational procedure to model complex soil-pile foundation systems in support of a seismic evaluation of the Anchorage Port Access Bridge constructed in 1975. The 2600-ft (792m) long bridge that connects the Port of Anchorage and the city of Anchorage is founded on steel pile footings with a total of 21 piers, 2 abutments at the south end, and 3 abutments at the north end. In total, 64 pile group foundations are composed of various numbers of vertical and batter H piles resulting in pile length ranging from 137.8 to 180.2 ft (42.0 to 54.9 m). In this study, 3D soil-structure pushover analysis that considers the nonlinear soil behavior, the effects of seasonally frozen ground, and the pile group configuration was first performed to obtain load–displacement response of each pile group. Then, through an inverse analysis, a full 6x6 stiffness matrix representing 3 translational and 3 rotational directions was developed for a given pier and can be further condensed into 6 uncoupled nonlinear springs. The pile group stiffness matrix effectively models the complicated pile group foundation behavior and can be readily inserted at the base of each bridge column in the overall structural model.

¹ Staff Engineer, URS Corporation, Oakland, CA 94536
²Assistant Professor, Dept. of Civil Engineering, National Chung Hsing University, Taichung, Taiwan
³Department Manager, URS Corporation, Oakland, CA 94536

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This paper presents a rational procedure to model complex soil-pile foundation systems in support of a seismic evaluation of the Anchorage Port Access Bridge constructed in 1975. The 2600-ft (792 m) long bridge that connects the Port of Anchorage and the city of Anchorage is founded on steel pile footings with a total of 21 piers, 2 abutments at the south end, and 3 abutments at the north end. In total, 64 pile group foundations are composed of various numbers of vertical and batter H piles resulting in pile length ranging from 137.8 to 180.2 ft (42.0 to 54.9 m). In this study, 3D soil-structure pushover analysis that considers the nonlinear soil behavior, the effects of seasonally frozen ground, and the pile group configuration was first performed to obtain load–displacement response of each pile group. Then, through an inverse analysis, a full 6x6 stiffness matrix representing 3 translational and 3 rotational directions was developed for a given pier and can be further condensed into 6 uncoupled nonlinear springs. The pile group stiffness matrix effectively models the complicated pile group foundation behavior and can be readily inserted at the base of each bridge column in the overall structural model.

Introduction

A geotechnical study was performed in support of a seismic evaluation of the Anchorage Port Access Bridge sponsored by Alaska Department of Transportation and Public Facilities (AKDOT&PF). The bridge, constructed in 1975, connects the Port of Anchorage and Elmendorf Air Force Base to the north with the city of Anchorage to the south. The 2600-ft long structure crosses Ship Creek, connecting the Anchorage business district to the south and the Port of Anchorage to the north. The Ship Creek Valley is bordered to the north and south by approximately 90 foot (27 m) high bluffs originally steep to moderately steep but now largely modified and flattened by grading. Ship Creek is restricted to an approximately 200 foot (61 m) wide channel, stabilized by riprap revetments near the center of the valley. Elevations on site range between approximately 7 feet (2.1 m) above MSL (mean sea level) within the Ship Creek channel to approximately 90 feet (27 m) above MSL at a point on Ocean Dock Road where it crosses the northern bluff. A vicinity map of the bridge is presented in Figure 1. The bridge is founded on steel H-pile footings that support steel columns and integral steel caps [1]. The superstructure is composed of varying cross sections made up of steel plate girders and a cast-in-place concrete deck. In this study, we provide a rational procedure to develop a simple stiffness

¹ Staff Engineer, URS Corporation, Oakland, CA 94536
² Assistant Professor, Dept. of Civil Engineering, National Chung Hsing University, Taichung, Taiwan
³ Department Manager, URS Corporation, Oakland, CA 94536

matrix that can represent the complex soil-pile foundation system for each pile group. This stiffness matrix can then be readily inserted at the base of each bridge column in the overall structural model.

Figure 1. Vicinity map of Anchorage Port Access Bridge.

Subsurface Condition

Review of the geologic mapping and all available geotechnical data indicates the subsurface soil conditions at the vicinity of the bridge consist of surficial fill underlain by primarily unconsolidated Holocene alluvial fan deposits [3, 4, and 5]. The borings encountered alluvial deposits that primary consisted of soft to stiff clay, which is known locally as Bootlegger Cove Clay. An idealized geologic cross section based on the previous and new investigation data is developed and shown in Figure 2. For the purposes of developing idealized soil profiles for foundation modeling, the subsurface conditions are grouped into three segments: North Bluff, Ship Creek Valley and South Bluff.
North Bluff

Three historical borings indicate a surficial dense sandy gravel layer ranging from 20 to 40 feet (6 to 12 m) thick. This layer is underlain by a dense sandy layer. The thickness of this sand layer is about 20 to 25 feet (6 to 7.6 m). The next layer encountered was stiff silty clay to the bottom of exploration at about 100 ft (30.5 m) bgs.

Ship Creek Channel Valley

The borings within the Ship Creek Valley indicated a surficial dense sandy gravelly layer with occasional silt pockets. This surficial layer is underlain by a medium stiff to hard silty clay layer, which is commonly known as the Bootlegger Cove Clay. The thickness of this clay layer is about 140 to 160 feet (42.6 to 48.7 m). The next encountered layer is an undetermined thickness of very dense and sandy gravel.

South Bluff

The borings within the South Bluff area encountered about 18 feet (6 m) of soft to stiff clay and silt underlain by 10 feet (3 m) of dense poorly graded sand. Below this sand layer, clay was encountered to the bottom of the borings. The upper 50 feet (15 m) of this clay layer are very soft to medium stiff with undrained shear strength less than 500 psf (24 kPa), which is consistent with the slip plane of the 4th Ave landslide that occurred in the 1964 Good Friday earthquake [6].

Design Profile

We developed idealized subsurface profiles based on available geotechnical data for the purpose of the foundation evaluations. In-situ and laboratory tests (including pocket penetrometer [PP], torvane, vane shear, moisture content, Atterberg limit, unit weight, unconfined compressive strength [UCT], unconsolidated–undrained triaxial compression test [UU], and grain size distribution) were performed on samples recovered from previous explorations and new explorations in this study. The undrained shear strength data along with strength design curves versus elevation for fine-grained soils are presented in Figure 3.
Existing Foundations

The Anchorage Port Access Bridge is founded on steel H-pile footings that support steel columns and integral steel caps [1]. There are a total of 21 piers, 2 abutments (Abutments 1A and 1C) at the south end, and 3 abutments at the north end (Abutments 20, 20WS and 23SW). Abutments 1A and 1C consist of 3 parallel walls (front wall, center wall, and back wall). Abutments 20 and 20WS consist of 2 walls (front wall and back wall). Abutment 23 consists of a main wall with 2 return walls. Each pier/abutment consists of 2 to 4 columns. In total, there are 64 pile group foundations. For the piers, most of the pile groups are composed of 4 vertical piles and 4 batter piles but may contain up to 13 piles in the larger footings. For Abutment 1A, 1C, 20, and 20SW, the pile groups are composed of 5 to 7 piles. For Abutment 23, the pile group is composed of 36 piles. Batter angles vary from groups to groups with a range between 1:3.5 and 1:6.5. The HP 12 x 53 piles were driven by Vulcan 020C hammer (60,000 ft-lb/stroke), Vulcan 140C hammer (36,000 ft-lb/stroke), Vulcan 200C hammer (50,200 ft-lb/stroke), or Kobe K-32 Diesel (60,100 ft-lb/stroke) to the deep dense sand/ gravel layer, which resulted in pile lengths ranging from 137.8 to 180.2 ft.

Foundation Stiffness

Stiffness Matrix

The single pile-soil system can be represented as a 6x6 coupled stiffness matrix. This stiffness matrix can be estimated to match the soil-pile behavior for all six degrees of freedom [7, 8, and 9]. The common form of the stiffness matrix in the local pile coordinate system is given by the following expression:
In which $k_x$, $k_y$, $k_z$, and $k_{\theta x}$, $k_{\theta y}$, $k_{\theta z}$, are stiffness coefficients corresponding to translational and rotational degrees of freedom associated with the local $x'$ (along the pile axis), $y'$ and $z'$ axes, respectively as shown in Figure 4.

$$[K] = \begin{bmatrix}
  k_x & 0 & 0 & 0 & 0 & 0 \\
  0 & k_y & 0 & 0 & 0 & k_{\theta z} \\
  0 & 0 & k_z & 0 & -k_{\theta y} & 0 \\
  0 & 0 & 0 & k_{\theta x} & 0 & 0 \\
  0 & 0 & -k_{\theta y} & 0 & k_{\theta y} & 0 \\
  0 & k_{\theta x} & 0 & 0 & 0 & k_{\theta z} \\
\end{bmatrix}$$

Figure 4. Coordinate systems for individual pile vs. global pile group. [9]

The $x'$-coordinate is taken as along the pile axis. The off-diagonal terms represent coupling between two degrees of freedom for two orthogonal axes perpendicular to the pile axis. For development of a 6x6 coupled stiffness matrix, linearization of p-y curves is needed or Terzaghi’s linear subgrade modulus can be directly used. Because the p-y curves are nonlinear, developing the foundation stiffness matrix involves linearization of the p-y curves by performing lateral pushover analysis of the single pile to a representative displacement expected during the earthquake.

The pile group can also be represented by a linear 6x6 stiffness matrix with an approach as described by Lam and Martin [7]. The global stiffness of the pile group is computed by the summation of individual pile-head stiffness, involving static equilibrium. The procedure assumes that the pile cap is infinitely rigid. For a vertical pile group, the form of the pile group stiffness matrix will be identical to the individual pile. The pile group stiffness for the translational displacement terms and the cross-coupling terms can be obtained by merely multiplying the corresponding stiffness components of the individual pile by the number of pile. However, the
rotational stiffness (two rocking terms and one torsion term) require consideration of an additional component. In addition to individual pile-head bending moments at each pile head, a unit rotation at the pile cap will introduce translational displacements and corresponding forces at each pile head; these pile-head forces will work together among the piles and will result in an additional moment reaction on the overall pile group. For a pile group with batter piles, assembling group stiffness is even more complicated than vertical piles, which involve transformation of local stiffness to global stiffness and considering an additional component for rotational stiffness.

**Procedure of Obtaining Stiffness Matrix**

To avoid pitfalls assembling individual pile stiffness to a group stiffness, the group stiffness was obtained by an alternative approach as described in the following:

1. Model entire pile group system with all vertical piles and battered piles including the pile cap within soil stratigraphy.

2. Perform push-over analyses with a single load $P_j$ corresponding to only one degree of freedom $j$. This load is applied at the center of footing in 10 step increments.

3. Obtain displacement $\delta x_{ij}$ at all 6 degrees of freedom due to the force applied at specific degree of freedom $j$.

4. Calculate flexibility matrix (6x6) corresponding to this specific degree of freedom

$$f_{ij} = \frac{\delta x_{ij}}{P_j} \quad (2)$$

5. Calculate full stiffness matrix (6x6) by inverting flexibility matrix but only select the stiffness elements corresponding to this specific degree of freedom, which is

$$[K_j] = \begin{bmatrix} k_{1j} & k_{2j} & k_{3j} & k_{4j} & k_{5j} & k_{6j} \end{bmatrix} \quad (3)$$

6. Repeat step 2 to 5 to obtain the stiffness element $k_{ij}$ corresponding to other degrees of freedom

7. Assemble full stiffness matrix (6x6) as following

$$[K] = \begin{bmatrix} k_{11} & k_{12} & k_{13} & k_{14} & k_{15} & k_{16} \\ k_{12} & k_{22} & k_{23} & k_{24} & k_{25} & k_{26} \\ k_{13} & k_{23} & k_{33} & k_{34} & k_{35} & k_{36} \\ k_{14} & k_{24} & k_{34} & k_{44} & k_{45} & k_{46} \\ k_{15} & k_{25} & k_{35} & k_{45} & k_{55} & k_{56} \\ k_{16} & k_{26} & k_{36} & k_{46} & k_{56} & k_{66} \end{bmatrix} \quad (4)$$

The resulting pile group stiffness matrix effectively “condenses” the complete pile group foundation behavior for a given pier. The bridge structural engineer can then insert the pile group stiffness matrix at the base of each column in their overall structural model to model the individual pile group stiffness.
Pile Group Foundation Modeling

Soil-pile interaction under vertical and lateral loadings is modeled using nonlinear Winkler foundation models. The analysis procedure described above is performed using the computer program GROUP 8.0 [10]. GROUP 8.0 can analyze the pile group response to applied lateral and axial loads with a series of nonlinear springs that are internally generated by the program as a function of user-specified soil properties (e.g. soil type, unit weight, shear strength parameter). In addition, the interaction effect among the piles in each pile group is also internally evaluated in GROUP 8.0. Pile properties used in the analyses include length, diameter, moment of inertia, area, and modulus of elasticity. As shown in Figure 5, the pile group details are modeled in a 3D fashion according to the as-built information such as pile length, batter angle, cutoff elevation, and the pile cap. Upon review of the as-built pile to cap connection details, the structural engineer indicated that the pile head conditions should be modeled as fixed head. Corresponding idealized profiles were used according to the abutment/pier locations. Because the pile-soil system is nonlinear, the calculated stiffness will be dependent on the magnitude of deflection.

A parametric analysis to evaluate the contribution of soil passive resistance acting against the pile caps is also performed. Because lateral resistance of batter piles is mobilized at smaller strains than pile cap passive earth pressures, the contributions of pile cap passive resistance to overall group stiffness was negligible. In addition, the magnitudes of the cross-coupled stiffness of each pile group calculated are relatively small compared to the diagonal stiffness terms. Therefore, the cross coupling stiffness would have negligible contribution on the behaviors of the foundations under vertical and lateral loadings. As a result, only the diagonal terms of the stiffness matrix for each pile group are presented. Figure 6 and 7 shows the stiffness matrices for a selected pier (Pier 11 composed of 9 piles) and a selected abutment (Abutment 23SW composed of 36 piles), respectively. The foundation stiffness matrix should be selected based on a representative displacement expected during the earthquake, and the calculated pile group stiffness corresponds to the stiffness at pile head (bottom of pile cap).

In addition, the potential effects of seasonally frozen ground on pile group stiffness are also evaluated. The depth of seasonally frozen ground at the site can be estimated to be on the order of 10 feet (3 m) or less [11]. The shear strength of frozen soil in the upper 10 feet can be on the order of up to three times the unfrozen shear strength [11]. Using these frozen ground conditions, the k11, k55, and k66 frozen ground pile group stiffness values show no appreciable variation from the unfrozen ground stiffness values while the k22, k33, and k44 frozen ground pile group stiffness values are appreciably higher than the unfrozen ground stiffness values, and are a function of strain.
Figure 5. GROUP 8.0 3D model and coordinate system.

Figure 6. Pier 11 stiffness matrix.
The transfer functions from unfrozen ground condition to frozen ground conditions for $k_{22}$, $k_{33}$, and $k_{44}$ are evaluated and are shown in Figure 8. These transfer functions can be factored against the unfrozen ground stiffness values to obtain frozen ground stiffness values for any given pier. These values represent a theoretical upper-bound pile group stiffness that may not be fully attained at the time of a seismic event.

Conclusions

A rational procedure to model complex soil-pile foundation systems is illustrated in this paper. The computer program GROUP 8.0 is utilized to perform a 3D nonlinear soil-structure pushover analysis to obtain the load-displacement response for each pile group. By internally inverting the flexibility matrices generated by GROUP 8.0, a 6x6 stiffness that represents the 3 translational and 3 rotational components of the pile group can be developed. Given the small magnitudes of the cross-coupled stiffness of each pile group relative to the diagonal stiffness terms, the stiffness matrix can further be condensed into 6 uncoupled nonlinear springs. In addition, the effects of seasonally frozen ground are also evaluated by considering the increase of the soil shear strength. This procedure is able to provide a pile group stiffness matrix that can
effectively model the complicated pile group foundation behavior and be readily inserted at the base of each bridge column in the overall structural model.

References


