SEISMIC ANALYSIS OF A REINFORCED CONCRETE EXCAVATION SHORING SYSTEM ADJACENT TO A PARTIALLY EMBEDDED BUILDING

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ABSTRACT

We performed a seismic response-history analysis of a shoring system designed to provide lateral support for a planned construction excavation that will expose one side of an existing building. The excavation is about 16 m deep and extends up to 25 m from the building. The site conditions consist of soft soil for the first 8 m and weathered rock underneath. The adjacent building is a five-story 19 m by 28 m rectangular reinforced concrete structure. Three stories are embedded 12.25 m underground, and two stories extend 7.25 m above ground. The excavation shoring system consists of a vertical piled retaining wall to hold back the soil at the far end of the excavation and a reinforced concrete space frame that laterally supports the retaining wall and channels the forces back to the exposed walls of the existing structure on the other end. The response-history analysis used the finite element method to model the adjacent structures and soil excavation. The force distributions in the retaining wall and shoring system were computed for ground motions corresponding to twice the Safe Shutdown Earthquake. We used an efficient hybrid approach that combined two-dimensional (2D) response history analysis in computer program SASSI2000 and 2D and three-dimensional (3D) quasi-static analyses in computer program SAP2000. This hybrid approach was used in order to provide estimates of the seismic demands for the shoring system design whose schedule would have been significantly delayed by a full 3D analysis. The analysis identified two distinct modes dominating the shoring system dynamic response: (1) inertia loading of the soft soil layers bearing onto the vertical retaining wall, and (2) bearing of the shoring system onto the excavation boundary due to the seismic response of the adjacent building. Critical member force demands were obtained by modal combination of equivalent static analyses which exhibited a good match to dynamic analysis results.

INTRODUCTION

A nuclear power plant (NPP) planned to implement an upgrade which included the construction of a new totally embedded building just north of an existing building. During construction, an open excavation with vertical sides was to occupy the footprint of the new structure. The NPP wanted to develop design forces for the excavation shoring system and investigate potential significant increases in demands on the existing building if a design-level earthquake occurred during construction. The planned excavation was to expose more than one-half of the north side of the existing building down to its foundation. The excavation shoring system was to engage the existing building walls and floor slabs for lateral support. The north-south dynamic response of the combined structural system was an unusual case of soil-structure interaction (SSI).

PROBLEM DESCRIPTION

Building, Excavation and Shoring System Configuration
The existing building is a five-story reinforced concrete shear wall structure 19 m by 28 m in the east-west and north-south directions, respectively. The building is founded on a 1.20 m thick base mat at Elevation (-)12.25 m relative to grade. The three lower stories are embedded in soil, and the top two stories extend 7.5 m above grade.

Figure 1a shows a conceptual plan of the excavation boundaries and shoring system. The excavation boundary on the south side is the existing building wall. The boundaries on the other edges are formed by overlapping vertical concrete piles of 1.3 m diameter, spaced at 1.1 m, and having a common secant of about 1.1 m. Every other pile will be steel-reinforced. Figure 1b shows a conceptual cross section of the excavation shoring system. The excavation depth will be 15.80 m, deeper than the existing building foundation. The soil under the existing building foundation will be stabilized by soil nails, with vertical siding anchored to them. The secant piles will be shored using a three-dimensional (3D) reinforced concrete frame composed of horizontal concrete beams along the perimeter and horizontal cross beams at three levels: top-of-beam Elevations (-)2.50 m, (-)7.10 m, and (-)12.60 m. The north-south horizontal beams at Elevations (-)2.50 m and (-)7.10 m will be laterally supported on the exposed north wall of the existing building, while the horizontal beams at Elevation (-)12.60 m will be laterally supported on the vertical siding erected and anchored by the soil nails next to the existing building foundation.

Site Configuration and Seismic Input

The soil layers at the site consisted of relatively soft soil to depths of up to 8.0 m, followed by significantly stiffer rock underneath. The water table was located at Elevation (-)4.0 m. The design basis earthquake ground motion was twice the Safe Shutdown Earthquake (2xSSE). The 2xSSE ground response spectra were defined at a geologic outcrop at the foundation of the existing building (Figure 2a). AMEC (2013) performed site response analysis to obtain best-estimate strain-compatible soil properties (Figure 2b) and in-column motions at the bottom of the excavation (i.e., at Elevation (-)15.80 m). AMEC (2013) developed a set of median-centred horizontal and vertical acceleration time histories for the seismic response analysis. The 5% damped spectra for these ground motions are shown in Figure 2a for two horizontal (H1 and H2) and one vertical (V) components.

ANALYSIS METHODOLOGY

The analysis methodology followed the overview outlined in Figure 3 to idealize the existing building and excavation shoring 3D SSI response using a two-dimensional (2D) model. The 2D model was calibrated using a 3D model of each of the existing building and the excavation shoring. The 2D response was then distinguished into individual modes whose responses were propagated back to 3D in the form of equivalent static loads to estimate the distribution of demands on the shoring system elements. The modal responses were then combined using the square-root-of-the-sum of squares (SRSS) method. This process is described in more detail in the following sub-sections.

3D Finite Element Models

A 3D FE model of the existing building was developed using computer program SAP2000 (CSI, 2010) and the original building drawings (Figure 4a). Concrete Young’s modulus and Poisson’s ratio were 27 GPa and 0.17, respectively, with stiffness reduction where cracking was expected. Fixed-base north-south and vertical vibration modes were extracted for calibration of the 2D FE model (Table 1).

A 3D FE model of the excavation shoring system and surrounding soil was developed using SAP2000 (CSI, 2010) (Figure 4b). In order to model far-field boundaries, the soil mesh extended between 40 m and 45 m to the north and south of the vertical retaining wall, 20 m to the east and west of the shoring
system sides, and 45 m below the surface. The soil element size ranged from 1.0 to 1.1 m. Rigid end offsets were used to model the joint regions, which had significant widths compared to the shoring structure size. The offset between the vertical piles and the concrete frame was modelled using rigid outriggers with no moment transfer capacity to the concrete frame. Young’s modulus for piles was reduced by one-half for cracking, and unreinforced pile stiffness was credited due to 3D displacement compatibility. In three independent load cases, unit forces were applied towards the north at the south ends of each pair of shoring beams, pushing the shoring system into the soil. Figure 5 shows the forces applied at the top tier and the resulting displacement profile. These results were used to calibrate the 2D FE model.

**2D Finite Element Model of Existing Building**

A 2D FE model of the existing building was developed in SAP 2000 (CSI, 2010) and calibrated to match the dynamic characteristics of the first few vibration modes in the 3D FE model. The 2D FE model mass represented a unit width (1 m) of the existing building, lumped at the floor elevations. The stiffness of the wall material was calibrated to a Young’s modulus of 3,000 MPa and Poisson’s ratio of 0.17. The flexural stiffnesses of the floor slabs and the exterior east-west walls were represented using frame elements of unit width and the actual depths of the elements they represented. The concrete Young’s modulus and Poisson’s ratio for these frame elements were 27.7 GPa and 0.17, respectively. Table 1 shows the close match of the first three horizontal modes (the second and third 3D modes combine into the second 2D mode). The first vertical mode frequency is comparable, but the mass participation ratio is not, which is of minor influence on the governing horizontal response. The mismatch in vertical mass participation ratio is attributed to the absence of floor slab vertical flexibilities in the 2D model.

**2D Finite Element Model of Excavation and Shoring System**

A 2D FE model of the excavation and shoring was developed using SAP2000 (CSI, 2010) and calibrated to match the north-south response of the 3D FE model. A schematic is shown in Figure 6a, and positioned relative to the 2D FE model of the existing building. The FE model used plane strain elements to represent a unit width of soil layers and frame elements to represent the shoring system. The material properties were the same as the 3D FE model. Beam element widths were assigned the total beam width divided by the center-to-center width of the excavation of 11.3 m. Beam intersections with cross beams, which occupied significant volumes, were stiffened by a factor of 100. Pile elements were assigned a diameter of 1.24 m instead of 1.30 m to scale their stiffness from 1.1 m spacing to unit width. Rigid springs were added where indicated to report the soil bearing pressures on the secant piles. Four calibrated flexible springs were added where indicated and calibrated to account for the minor-axis bending flexibility of the east-west cross beams, which cannot be explicitly modelled in the 2D model (Table 2). The calibration did not explicitly include the stiffness of the vertical siding anchored by the soil nails, which was judged to be minor compared to the cross shoring beam behind it (Figure 1b).

Figure 6b shows a schematic plan view of the deflected shape of the pile and shoring beam system. The calibration applied unit forces independently at the south end of each shoring tier in the 2D and 3D FE models. The objective of calibration was to achieve similar displacements at the points of load application in both 2D and 3D models, and a displacement value at the end of each flexible spring that averages the displacement profile along the cross beam and matches the average soil pressure transferred from the vertical piles.

**2D SSI Model**

The 2D structural models of the existing building and the excavation shoring were combined into one model and exported to computer program SASSI2000 (Ostadan, 2007). In addition to the existing and the
shoring system, the SASSI2000 model incorporated an excavated soil mesh with a horizontal element size on the order of 1.0 m. For the purpose of designing the shoring system, this 2D idealization excludes the contribution of passive earth pressure against the north face of the existing building not open to the excavation and the possibility of non-uniform horizontal displacement in the building along the length open to the excavation.

The excavation impedance was calculated using the direct method in SASSI2000 (Ostadan, 2007). Stiff zero-length translational spring elements were defined between the interaction nodes at the boundary of the excavation soil mesh and the structural model to record seismic bearing pressures. These springs were modified to have low shear stiffness where they intersected the secant piles, so only normal pressure was transferred to the piles. The side soil around the partially embedded existing building can only impart seismic earth pressure bearing against the south wall in one direction but cannot impart tension in the opposite direction. Since SASSI2000 (Ostadan, 2007) can only model linear-elastic response, the springs connecting the soil mesh to the south wall were assigned effectively zero stiffness to calculate an upper-bound estimate of the forces in the shoring beams. No sliding was modelled between the existing building foundations and the underlying soil. This represents a case where adequate north-direction base shear can be transferred to the underlying rock through the following load paths: friction, vertical depressions in the existing building base mat acting as shear keys, and passive soil resistance against the embedded areas of the existing building north wall not exposed by the excavation.

FINITE ELEMENT SIMULATIONS

2D Dynamic SSI Analysis

The horizontal and vertical in-column acceleration time histories developed by AMEC (2013) were applied in SASSI2000 (Ostadan, 2007) at the bottom of the excavation. The dynamic response was dominated by two modes: the inertia load of the soil column north of the secant piles bearing southward on the piles and pushing them towards the existing building, and the existing building pushing the shoring system into the soil to the north. These two response modes had distinct vibration frequencies. The frequency of the soil column is approximately 3.5 Hz. Figure 7 shows the axial force response history in the north-south shoring beams for the H1 input motion. The forces in the lower two tiers are mostly in phase, while the force in the upper tier oscillates at a different frequency. The upper tier beam is located at Elevation (-)2.5 m, so its response exhibits participation from both dynamic modes. The lower two tiers are at Elevations (-)7.60 m and lower, so their response is primarily due to the existing building mode. The maximum upper tier beam force magnitude in Figure 7 occurs just after 12.5 sec, with the forces in the other two tiers aligned in phase.

The demands on the shoring system were calculated using the SRSS of the higher response from the H1 and H2 motions together with the response to the vertical motion. The maximum axial force demands in the shoring beams are listed in Table 2. The maximum demands on the secant piles per unit width are shown in Figure 8. These pile demands are for a unit width of excavation and represent an “average” pile demand from the 2D dynamic analysis.

2D Quasi-Static Analysis

Two quasi-static load cases were imposed on the 2D FE model in SAP2000 (CSI, 2010) to represent the two primary dynamic response modes. The soil inertia mode was represented by applying the maximum horizontal free-field displacements, obtained from the 2D dynamic SSI analysis, on the soil profile 20 m north of the retaining wall face (Figure 9a). The 20 m distance was based on comparing the SSI displacements at increasing distances north of the retaining wall face to the free-field displacements in AMEC (2013). To represent the contribution of the existing building shear and rocking flexibility to this
mode, the south end of the top tier shoring beam was attached to a horizontal spring calibrated to 15,000 kN/m so that the maximum horizontal displacement at the north end of the shoring beam matched the SSI results. The resulting axial force in the top tier beam was 280 kN.

The existing building mode was represented by applying horizontal forces on the south ends of the shoring beams (Figure 9b). The applied forces were based on the maximum shoring beam demands in Table 2, scaled to represent a unit-width 2D model. Since only the top tier beam response had significant contribution from the soil inertia mode, and since Figure 7 showed that its maximum axial force occurred when the two modes were in phase, the horizontal force applied to the top tier beam was adjusted to subtract the effect of the soil inertia mode.

Figure 10a shows the resulting bending moment distributions along the secant piles from both modes and their SRSS combination. The bending moments at the critical pile section near Elevation (-)8.0 m have opposite signs if the shoring beam forces from both modes are additive. Therefore, the in-phase case of the dynamic modes would reduce the pile demand, and the phasing of the modes at maximum pile response is not exactly known. Using the SRSS method in combining the bending moments resulted in an acceptable match with the 2D dynamic SSI demands at most locations along the pile profile, and a particularly good match (SRSS was 1% higher than SSI) at the critical section near Elevation (-)8.0 m (Figure 8).

Figure 10b shows the shear force distributions along the secant piles from both modes and their SRSS combination. The SRSS shear force combination resulted in a generally acceptable match to the 2D dynamic SSI demands along the secant pile. At the location of maximum shear, the SRSS combination was only 2% lower.

3D Quasi-Static Analysis

After successfully reproducing the results of interest from the 2D dynamic SSI analysis in the 2D quasi-static analysis, the load cases employed in the latter were applied on the 3D FE model of the excavation and shoring system in SAP2000 (CSI, 2010) (Figure 5a). The responses from the two quasi-static load cases were combined using the SRSS method. The full spatial distribution of the secant pile demands could therefore be generated (Figure 11). Since the secant piles were reinforced only every other pile, the maximum secant pile design demands were computed as the maximum values tributary to each reinforced pile from the neighbouring unreinforced piles. The maximum pile demands were typically at or near the corner piles, as expected, since these piles are more directly attached to the north-south shoring beams and therefore push more into the soil than the piles near the centre of the cross shoring beam. The maximum bending moment and shear force values at the critical pile sections were 720 kN-m and 790 kN, respectively. In addition, the maximum minor-axis bending moments in the east-west shoring cross beams that provide lateral support to the piles could be extracted, as reported in Table 2.

Analysis Approximation

Computer program SASSI2000 (Ostadan, 2007) employs only linear analysis and cannot model compression-only soil behaviour. However, a tension force in the shoring beam due to the soil inertia mode is not realistic since the soil cannot develop tension to pull on the shoring system. Therefore, the pile bending moments for the soil inertia mode in Figure 10a are only valid for soil pushing against the retaining wall and generating compressive axial forces in the north-south shoring beams; the maximum in the opposite direction is zero. The maximum resultant bending moment in the secant pile should therefore not be more than the value from the existing building mode alone. Some minor conservatism therefore existed in the 2D dynamic SSI analysis and was in turn propagated to the SRSS combination of responses from the quasi-static analyses.
CONCLUSION

A hybrid 3D/2D finite element analysis approach was successfully applied to estimate force demands on the excavation shoring system resulting from seismic SSI in the north-south direction. This hybrid analysis approach was able to estimate input forces to support the design of the excavation shoring system at a fraction of the time and computational cost of a full 3D SSI analysis. 2D FE models of the excavation, shoring, and existing building were created and calibrated to replicate the effective planar response characteristics of 3D FE models. Response-history SSI simulations were performed on the assembled 2D FE model in computer program SASSI2000 (Ostadan, 2007). The axial force response histories in the shoring beams were used to identify their maximum demands and to decompose the dynamic response into two primary modes. The maximum response of the 2D system to each mode was reproduced using quasi-static analysis in computer program SAP2000 (CSI, 2010). The SRSS combinations of critical member demands from both modes exhibited a good match with the maxima from the dynamic SSI analysis. The quasi-static load cases were then applied on a 3D model of the system in computer program SAP2000 (CSI, 2010) to capture the spatial distribution of the responses estimated by 2D analyses. The SRSS combination of the 3D quasi-static analysis results was used to calculate the governing pile demands near the corner of the excavation and the minor-axis bending moment demand on the cross shoring beams that provide lateral support to the vertical piles.

REFERENCES


ILLUSTRATIONS

Table 1 – Significant Modes of Existing Building FE Models

<table>
<thead>
<tr>
<th>Direction</th>
<th>Mode</th>
<th>Frequency (Hz)</th>
<th>Mass Participation Ratio (%)</th>
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<tr>
<td></td>
<td></td>
<td>3D</td>
<td>2D</td>
</tr>
<tr>
<td>North-South</td>
<td>1</td>
<td>10.6</td>
<td>10.6</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>26.1</td>
<td>27.1</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>27.1</td>
<td>6.1</td>
</tr>
<tr>
<td>Vertical</td>
<td>1</td>
<td>20.1</td>
<td>21.2</td>
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</table>

Table 2 – Calibration Spring Stiffness and Design-Level Demands in Shoring Beams (per beam)

<table>
<thead>
<tr>
<th>Beam Top Elevation (m)</th>
<th>North-South Shoring Beams</th>
<th>Cross-Beam behind Piles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Axial Spring Stiffness (kN/m)</td>
<td>Axial Force (kN)</td>
</tr>
<tr>
<td>(-)2.50, Top Tier</td>
<td>250</td>
<td>2,010</td>
</tr>
<tr>
<td>(-)7.60, Middle Tier</td>
<td>450</td>
<td>1,160</td>
</tr>
<tr>
<td>(-)12.60, Bottom Tier</td>
<td>4,000 (each of 2 springs)</td>
<td>1,350</td>
</tr>
</tbody>
</table>
Figure 1 – Conceptual Configuration of Planned Excavation

(a) Plan       (b) Cross Section

Figure 2 – 2xSSE Design Ground Motion Spectra and Soil Profiles

(a) In-Column Spectra at Elevation (-)15.80 m  (b) Strain-Compatible Soil Shear Moduli

Figure 3 – Overview / Flowchart of Analysis Methodology

Assemble SAP Fixed-Base Building 3D Model
Calibrate
Develop SAP Fixed-Base Building 2D Model
Calibrate
Develop SAP Excavation Shoring + Soil 3D Model
Report Shoring Element Demands
Apply Static Load to 3D Model
Assemble SASSI 2D Model
Dynamic Analysis
Process and Review Results
Develop Equivalent Static Load

(a) Existing Building  (b) Excavated Soil and Shoring System

Figure 4 – 3D SAP2000 FE Models (from Northeast)

(a) Applied Forces  (b) Deformed Shape

Figure 5 – Calibration Loading Case for Shoring System 3D FE Model

(a) 2D FE Model Schematic Configuration  (b) Plan of True and Idealized Response

Figure 6 – 2D Model Idealization of 3D Shoring System Response
Figure 7 – Shoring Beam Axial Force Time History

Figure 8 – Maximum Pile Demands from 2D Dynamic SSI Analysis (kN, m)

(a) Free-Field Displacements at Soil Boundary  (b) Axial Forces at Shoring System Boundary

Figure 9 – 2D Quasi-Static Load Cases (kN, m)
Figure 10 – Maximum Pile Demands from 2D Quasi-Static Analysis (kN, m)

Figure 11 – Spatial Distribution of SRSS Demands from 3D Quasi-Static Analysis