Seismic Response of a Typical Highway Bridge in Liquefiable Soil

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ABSTRACT: For reliable estimation of bridge performance, especially when subjected to liquefaction and lateral spreading, appropriate modeling of soil-pile-structure interaction and understanding of the global bridge behavior are essential. In practice, soil and foundation systems are often approximated using very simple foundation springs or unrealistic lateral spreading mechanisms that may not represent all important aspects of the global system behavior. In this context, a detailed bridge model of a typical highway overpass bridge in liquefiable soils was developed with an emphasis on accurate characterization of the structural and soil behavior, including various soil-pile-structure (SPSI) modeling strategies for several components in the bridge system, such as piles, pile caps, and abutment structures. Using this model, the global behavior of the bridge system is more readily understood by capturing realistic force boundary conditions at the bridge pier bases and bridge deck ends. In addition, important bridge damage mechanisms are identified.

INTRODUCTION

The seismic response of a bridge depends on the force boundary conditions at the pier bases and bridge deck ends. The response can be complicated due to the interaction between the components of the bridge system - i.e., bridge structure, soil, foundation, and abutment structure, especially when bridge and pile structures are subjected to liquefaction and lateral spreading. However, soil and foundation systems are often approximated in practice using very simple foundation springs or unrealistic lateral spreading mechanisms that may not represent all important aspects of the global system behavior. Due to the interdependence and interaction between the bridge components, it is important to model the bridge as a complete system to better understand and quantify the global bridge response.

TARGET BRIDGE SYSTEM

This study considers a typical Caltrans highway bridge underlain by liquefiable soil susceptible to lateral spreading. The target bridge is shown in Figure 1. The bridge consists of a five-span reinforced concrete structure with a post-tensioned reinforced concrete box girder deck section. The three middle spans are 45.7 m (150 ft) long and the two end spans are 36.6 m (120 ft) long. The deck is 1.83 m (6 ft) deep and the four piers have a 6.71 m (22 ft) clear height. The pier columns are circular with a 1.2 m (4 ft)
Details on the bridge structure design are presented by Mackie and Stojadinovic (2007). The bridge columns are supported by 3x2 pile groups with center-to-center spacing of 1.83 m (6 ft). The individual piles are open-ended steel pipe piles with a diameter of 0.61 m (2 ft) and wall thickness of 0.0127 m (0.5 inch). The same pile type is used for the 6x1 abutment foundations with center-to-center spacing of 2.44 m (8 ft). The bridge piers and pile groups are labeled from the left abutment as Pier 1, Pier 2, Pier 3, Pier 4, and Pile 1, Pile 2, Pile 3, Pile 4, respectively. The pile groups in the left and right slope are labeled as Pile 0 and Pile 5. A seat-type abutment is considered in the analysis. The backwall is designed to shear off when subjected to large longitudinal bridge forces. The abutment piles are installed to the depth of the other bridge piles.

The soil below the left embankment consists of a medium stiff clay crust underlain by a thin, loose to medium dense sand, a layer of stiff clay, and a dense sand layer underlain by rock. The soil beneath the right embankment consists of the same clay crust underlain by a thicker layer of loose sand, followed by a dense sand layer underlain by rock. The lower clay layer below the left abutment becomes thinner toward the center of the bridge and does not exist below the right embankment. The embankments are 8.53 m (28 ft) in height and have 2:1 slopes. The groundwater table is located at the bottom of the surface clay layer. The properties of the loose and medium sand layers across the bridge are aimed to induce liquefaction under moderate ground shaking so that lateral spreading, especially on the right side, triggers broad bridge damage. The soil types and properties are shown in Table 1. Configuration of the bridge structure and abutment is shown in Figure 2.

![Figure 1 Target bridge system (dimensions in meter)](image)

### Table 1 Soil types and properties

<table>
<thead>
<tr>
<th>Soil layer number</th>
<th>Soil type</th>
<th>Unit weight (kN/m³)</th>
<th>Strength parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>①</td>
<td>Dense sand</td>
<td>21.2</td>
<td>φ = 45°</td>
</tr>
<tr>
<td>②</td>
<td>Medium stiff clay</td>
<td>17.3</td>
<td>c = 36 ~ 58 kPa</td>
</tr>
<tr>
<td>③</td>
<td>Loose sand</td>
<td>18.0 ~ 20.2</td>
<td>φ = 33 ~ 36°</td>
</tr>
<tr>
<td>④</td>
<td>Medium stiff clay</td>
<td>17.3</td>
<td>c = 40 ~ 58 kPa</td>
</tr>
<tr>
<td>⑤</td>
<td>Dense sand</td>
<td>21.2</td>
<td>φ = 40°</td>
</tr>
</tbody>
</table>

### NUMERICAL MODELING OF SOIL-Foundation-STRUCTURE SYSTEM

Using the target soil profiles and foundation design, the soil-foundation system was modeled in OpenSees. The Pressure Dependent Multi-Yield (PDMY) elasto-plastic material model developed by Yang et al. (2003) was used to model sandy soils. To account for saturated conditions, the PDMY material was coupled with a Fluid Solid Porous Material (FSPM) model. This material imposes an incompressibility condition that allows the generation of pore pressures. For clays, the Pressure Independent Multi-
Yield (PIMY) material model was used. In the target soil profile, the clay below the left embankment had higher strength, due to consolidation, than the clay located in the center bridge area. The loose sand layer below the right abutment had increasing density (i.e., decreasing liquefaction potential) with depth. These variable soil conditions along the length of the bridge contributed to the generation of incoherent motions at each bridge pier. Fifty (50) sub-soil layers were used to capture these soil conditions. The soil parameters used in this study were based on recommendations (http://cyclic.ucsd.edu/opensees) for typical soil conditions. The embankment side soil was extended outward 73.2 m (240 ft) from the slope crest. The outer-most soil column elements were modified to generate a realistic free-field response by increasing their out-of-plane thickness and constraining the nodes at the same elevation to have the same horizontal movement.

Several types of interface springs were used to model soil-structure interaction, as shown in Figure 3. The parameters of these interface elements reflect the existence of different soil types, ground water conditions, pile group effects, and passive earth pressures in the pile caps and abutments. The 3x2 pile groups that support the piers were simplified using equivalent two-dimensional 1x2 pile group models that combine the three piles in each out-of-plane row to produce an equivalent single pile. In OpenSees, the equivalent pile was generated by patching three individual pile sections without changing the diameter or pile wall thickness. The pile group spring parameters were
factored (using *p-multipliers*) to consider pile group effects. The *p-y* springs in the liquefiable soils were modeled using the *pyLiq1* model (Boulanger et al. 1999 and 2004) available in OpenSees. The *pyLiq1* material was coupled with adjacent soil elements that provide porewater pressure information. The spring resistance forces were based on API (1993) criteria and were factored by the porewater pressure ratio to approximate the liquefaction effect on soil-pile structure interaction. Residual strengths after liquefaction were calculated based on correlations to Standard Penetration Test (SPT) values. To capture the response of pile caps and abutment backwalls, passive earth pressure springs were used. The envelope of the pile cap passive earth pressure for clay followed the approach (*φ*=0 sliding wedge method) suggested by Mokwa (1999). For the abutment wall resistance, the resultant force-displacement envelope was based on Caltrans’ Seismic Design Criteria (2004).

The height of the backwall (break-off wall) was 1.8 m (6ft) and its width was 13.7 m (45 ft). The interaction between the bridge deck and abutment was decomposed into two interaction components. A schematic of the bridge deck-abutment interaction and model is shown in Figure 4. The first component combined the bearing pad resistance and backwall resistance in a single spring model. This spring force was transferred to the stem wall and abutment pile foundation. The stem wall was connected to the soil without interface assuming its relative displacement was small. The second component included the expansion joint gap and backwall lateral soil resistance. In this case the force-displacement envelope was obtained using a series combination of a gap spring and a soil spring with parameters based on Caltrans’ Seismic Design Criteria (2004). For the selected abutments, the initial stiffness and ultimate resistance used were 164,300 kN/m/m (20 kips/in/ft) and 6,258 kN (1290 kips), respectively.

The OpenSees nonlinear bridge model was originally developed for a performance-based earthquake engineering investigation by Mackie and Stojadinovic (2007). The original model used simple foundation springs at the bottom of the piers and abutment springs at the bridge deck-end to model soil compliance. To couple the bridge model with the soil model developed for this investigation, the simple foundation springs used by Mackie and Stojadinovic (2007) were removed and the pier columns were connected to pile groups. Details of the modeling of bridge system is described in Shin (2007).

**INPUT MOTIONS**

Four sets of input motions corresponding to return periods of 15, 72, 475, and 2475 years (97%, 50%, 10%, and 2% probability of exceedance in 50 years) at the site of an I-880 bridge study (Somerville 2002 [12]) were applied to the proposed model. These motions are near-fault motions that are distinguished by strong and long period pulses. To minimize dispersion in the input motion vs. response relationship, particularly for the case in which lateral spreading affects the bridge response, the 10 ground motions for each hazard level were scaled to a constant PGA/MSF value; where MSF is a magnitude scaling factor corresponding to the modal magnitude for each hazard level. Because the scaled motions were based on rock outcrop conditions, the motions were corrected to remove free surface effects prior to their uses as rigid base input motions in OpenSees. The ranges of PGA for each hazard level were 0.12 to 0.17g, 0.39 to 0.61g, 0.62 to 0.90g, and 1.1 to 1.4g, respectively. Additionally, a non-near-fault motion was considered.
SEISMIC RESPONSE OF BRIDGE SYSTEM

Seismic responses of bridge system components such as soil, bridge structure, pile, \( p-y \) springs, and abutment were obtained from OpenSees simulations applying four hazard levels (40 motions). This section presents details of the local seismic bridge response characteristics for a moderate intensity motion (Northridge 1994 at Century city Lacc North, CMG Station 24389, \( a_{\text{max}} = 0.25g \)) and a strong motion with a 475-year return period (Erzincan, Turkey 1992, \( a_{\text{max}} = 0.70g \)).

Soil Response & Liquefaction

The loose sand layer across the bridge site was liquefied over its full or partial thickness depending on the input motion intensities. Liquefaction caused lateral spreading to occur in both abutment slopes. In Figure 5 (a), permanent displacement patterns of the bridge system for the Erzincan motions are presented. This figure shows that larger lateral deformation occurred near the right abutment, with permanent soil movement ranging from about 0.2 m to 1.3 m depending on the hazard level. Figure 5 (b) illustrates the pore pressure ratio in the soil. For stronger shaking, the entire loose sand layer across the bridge liquefied. Figure 6 shows the time variation of pore water pressure ratio with depth below the right abutment for the Northridge motion illustrating different degrees of pore water pressure generation with time.
Figure 6 Spatial and temporal variation of pore pressure below right abutment in liquefiable soil – Northridge 1994, $a_{\text{max}} = 0.25\text{g}$

**Bridge Pier Response**

Actual input motions at the bottom of each bridge pier are presented in Figure 7 in terms of acceleration response spectra. Due to non-uniform soil conditions (i.e., different soil profile below the left and right abutments) and liquefaction induced lateral spreading, the bottom of each pier was subjected to different acceleration time histories and peak accelerations.

Figure 7 Acceleration response spectra at pile cap – Northridge motion 1994, $a_{\text{max}} = 0.25\text{g}$
**Pile and p-y Spring Response**

The horizontal slope movements significantly influenced the pile and pier bending moments. In particular, the bending moments in Pier 4 appeared to be closely correlated to horizontal slope movements. For small to moderate shaking events, the maximum bending moments at each bridge pier were greater than the residual bending moments after shaking, indicating that inertial forces controlled the maximum bending moments. For strong shaking events, however, the residual bending moment at each bridge pier or deck was greater than the transient maximum bending moment, indicating that the kinematic forces associated with lateral spreading controlled the maximum bending moment.

The pile force distribution and pile cap movement were significantly influenced by lateral spreading and the pattern of soil displacement with depth. This effect varied with ground motion intensity. Figure 8 shows maximum bending moments and their locations for motions corresponding to all four hazard levels in Piles 4 at the right abutment. In Pile 4, the maximum pile moment occurred, for small and moderate shaking, below the pile cap or at the interface between the surface clay and lower loose sand layers. However, for higher intensities, the maximum bending moment location moved down to the interface between the loose and dense sand layers (around 12 m depth).

![Figure 8 Maximum pile bending moments and their locations in Pile 4 near right abutment slope – four hazards (40 motions)](image)

In Figure 9, the p-y spring response in Pile 4 at 5.5 m is plotted together with the corresponding pore pressure ratio ($r_u$) time history of an adjacent soil element. The API-based soil resistance time history mobilized by the same displacement is also presented in the figure. After 5 seconds, when the soil was liquefied, the lateral soil resistance reached an ultimate residual lateral resistance much smaller than what would have been obtained using conventional API-based values. The figure shows the clear effect of liquefaction on lateral soil resistance and the benefit of using p-y springs that can take into consideration the effect of pore water pressure generation on the ultimate strength.
Abutment Response

Figure 10 illustrates bridge deck and abutment interaction in terms of spring force-displacement response for a strong shaking motion. Figure 10 (a) shows the lateral resistance in the bearing pad and backwall. The initial 10 cm of displacement reflects only bearing pad resistance. After gap closure, large lateral resistance was mobilized in the break-off wall. The wall resistance contribution ends after backwall failure (shears off with respect to the stem wall). The bearing pad interaction force was delivered to the stem wall of the abutment. Figure 10 (b) shows that passive earth pressure resistance in the backfill soil was mobilized with the broken portion of the wall after closing the gap. Figure 10 (c) shows the total lateral resistance transmitted from the abutment system to the bridge deck.

Figure 9 $p$-$y$ time histories during earthquake excitation - Erzincan, Turkey 1992, $a_{max} = 0.70g$

Figure 10 Response of abutment: (a) bearing pad spring response, (b) breakwall and soil spring response, and (c) total response - Erzincan, Turkey 1992, $a_{max} = 0.70g$
GLOBAL BRIDGE RESPONSE & ABUTMENT-BRIDGE INTERACTION

The expansion joints between the bridge deck and backwall closed when lateral spreading occurred. The entire bridge deck tended to move toward the left because the amount of lateral spreading in the right abutment was considerably larger than that in the left abutment. At the same time, lateral spreading also pushed the pile cap of Pier 4 to the left resulting in relatively small drift in that column. On the other hand, Pier 1 was subjected to very large column drifts because the bridge deck moved the upper end of the column to the left and lateral spreading moved the pile cap at the bottom of the column to the right. This global bridge behavior, which would likely not be anticipated by typical analyses that model only individual parts of the bridge, greatly affected the local response.

The bridge pier response was investigated using the displacements of the bridge deck and pile cap. Figure 11 shows drift ratios obtained by the column’s displacement difference between the top and bottom at each piers. The bottoms of the four piers (pile caps) moved towards the center of the bridge and the amount of each pier base displacement varied due to different levels of lateral spreading. As shown in Figure 11, Pier 1 base residual lateral displacement was about 20 cm after shaking and Pier 4 base residual lateral displacement was about 60 cm. In the meantime, the top of all piers moved together to the left as the larger lateral spreading beneath the right abutment pushed the entire bridge deck to the left. This bridge deck movement increased Pier 1 drift and decreased Pier 4 drift. Due to this global response, similar drift ratios were observed in Pier 1 and Pier 4 for this particular motion even though the base residual lateral displacement of Pier 4 was greater than that of Pier 1. This demonstrates that lateral spreading in an abutment slope can affect the column drift in the other side of bridge.

![Figure 11 Displacement of Pier 1 and Pier 4 at top and bottom (column height = 6.7m) – Erzincan motion, Turkey 1992, a_max = 0.70g](image)

CONCLUSIONS

A complete geotechnical bridge system model including pile foundations embedded in realistic soil conditions and abutment structures was developed and coupled to a bridge structure model (Mackie and Stojadinovic 2007). Appropriate modeling of soil-pile-structure interaction and soil-abutment-bridge interaction under lateral spreading soil conditions allowed the bridge structure to capture realistic force boundary conditions at
the pier base and bridge deck ends. Using this coupled model, the global behavior of the bridge system is better illustrated and important bridge performance variables can be identified. This study shows that the global response of a soil-foundation-bridge system was quite complex, particularly when soft and/or liquefiable soils are present. The use of OpenSees simulations provided improved understanding of the global response, and allowed identification of damage mechanisms that would not be captured by simplified analyses commonly used in contemporary practice.

One of the important components to the global bridge response was the abutment-to-abutment interaction caused by different level of lateral spreading and its effect on the bridge pier drift near slopes. Since the amount of lateral spreading at the right abutment in the simulation was considerably greater than that at the left abutment, the entire bridge deck moved toward to the left. At the same time, lateral spreading pushed the pile cap of the pier near the right slope to the left, so the column drift actually became smaller. On the other hand, the pier near the left slope was subjected to large column drift because the bridge deck moved to the left and the pile cap at the bottom of the column moved to the right due to lateral spreading of the left abutment. This type of response can only be obtained using advanced numerical models.

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REFERENCES