FULL SCALE BRIDGE ABUTMENT PASSIVE EARTH PRESSURE TESTS AND CALIBRATED MODELS

Patrick Wilson[1] and Ahmed Elgamal[2]

[1]*Graduate Student Researcher, Dept. of Structural Engineering, University of California, San Diego, CA, USA*
[2]*Professor, Dept. of Structural Engineering, University of California, San Diego, CA, USA*
Email: prwilson@ucsd.edu, elgamal@ucsd.edu

ABSTRACT: Passive earth resistance at the abutments may limit earthquake induced bridge deck displacements. Including this abutment contribution in seismic bridge design may reduce the demand on the bridge piers and foundation. In consideration of such bridge abutment force-displacement resistance, static and shake table full scale abutment passive earth pressure tests are presented and discussed. Experimental and numerical results are used to calibrate hyperbolic force-displacement backbone curves to represent the abutments in bridge analyses and simulations. Parameters for implementation of these relationships are presented to account for various possible design assumptions. A cyclic loading abutment spring model is described in order to employ the calibrated hyperbolic backbone curves in dynamic numerical bridge simulations.

KEYWORDS: Abutment, bridge, passive earth pressure, dynamic earth pressure, cyclic loading

1. INTRODUCTION

A series of collaborative tests and simulations has been underway to consider the seismic response of reinforced concrete bridges, including soil-structure interaction. For that purpose, facilities of the George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES, http://nees.org) have been employed to perform shake table experiments. One-quarter scale bridge models are being tested at the University of Nevada at Reno (Saiidi 2004). In addition, investigation of the abutment contribution to the dynamic bridge response was conducted at the University of California in San Diego as discussed in this article.

In seismic design (AASHTO 2007, Caltrans 2004), an abutment system relies on the soil backfill to provide resistance to longitudinal bridge deck displacement. During strong shaking, if the deck impacts the abutment, a sacrificial portion (the backwall) can break off into the backfill. Resistance to further displacement of the deck and backwall is then provided by passive earth pressure within the densely compacted soil (Shamsabadi et al. 2007). This abutment resistance can decrease the demand placed on other seismic components such as the bridge columns and foundation (AASHTO 2007). As a result, an accurate representation of the abutment backfill resistance may lead to a more economic seismic bridge design.

To aid in this regard, experiments performed on an abutment backwall, full scale in height, are described in this paper. Experimental results and numerical simulations are employed to provide abutment spring models for a range of possible design considerations and assumptions. These models can be used to represent the abutment contribution to the longitudinal seismic bridge response in pushover type analyses or dynamic bridge simulations.

2. FULL SCALE TESTS

In this study, static pushover experiments were performed initially to record the abutment-backfill longitudinal force-displacement capacity. Next, dynamic shake-table excitation experiments were conducted to investigate the influence of ground shaking on the mobilized passive earth resistance, and to provide a basis for calibration of the dynamic computational models. The experiments were performed inside a large steel soil container on an outdoor shaking table (Figure 1) at the Englekirk Structural Engineering Center (ESEC) in San Diego, California.

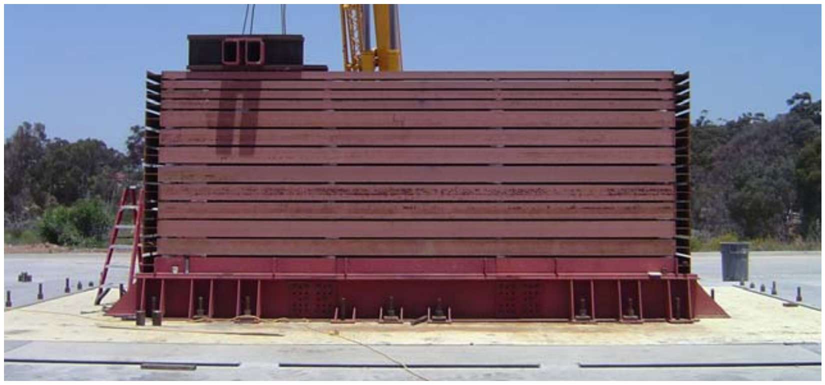

Figure 1: Soil container on outdoor shake table

2.1 Test Setup

Primary components of the experimental configuration (Wilson 2008) included a large laminar soil container (Figure 1), a model of a section along the width of an abutment (Figure 2a), a loading mechanism (Figure 2b) and a compacted sand backfill (Figure 2c). A restraining system (not shown) was later applied to the laminar box to create a rigid container configuration for the abutment tests. The inside dimensions of the soil container were about 2.9, 6.7 and 2.4 meters in width, length and height, respectively. The model abutment consisted of a separate seat box resting beneath a suspended sacrificial backwall (Figure 2a), which supported 1.7 meters of backfill. Hydraulic jacks reacted through load cells onto concrete-filled steel posts (Figure 2b) to push the wall into the backfill while measuring the applied load. Well-graded sand with about 7 percent fines (cohesion of about 14 kN/m^2 was observed in direct shear tests, Wilson 2008) was compacted (Figure 2c) for each test in compliance with Caltrans (1999) standard specifications for structural backfill. The unit weight of the dense sand backfill was approximately 20.6 kN per cubic meter. Dimensions of the backfill were about 2.9, 5.6 and 2.1 meters in width, length and height.

a) Backwall suspended above seat box
b) Loading mechanism
c) Backfilling and compaction inside container

Figure 2: Test setup photographs

As mentioned previously, the testing program consisted of both static push-over and dynamic excitation tests. First, the hydraulic jacks (Figure 2b) were used to push the backwall into the backfill in the two static pushover tests. In the testing configuration, the wall was permitted to move upwards. The force-displacement relationship was recorded up to and beyond the peak measured resistance. In additional experiments, dynamic excitations were imparted on the system by the shaking table while the hydraulic jacks (Figure 2b) were locked in a fixed position. Changes in measured force and pressure were recorded throughout the shake-table excitations (Wilson 2008).

2.2 Static Pushover Tests

Applied force and horizontal displacement were recorded as the abutment backwall was pushed into the backfill for two separate tests. Results from the two static pushover tests were combined to create an average force-displacement relationship for the abutment (Figure 3). The test average curve is shown shifted 2.54 cm (1 inch) to account for the expansion gap (Caltrans 2004) between the end of the bridge deck and the backwall.

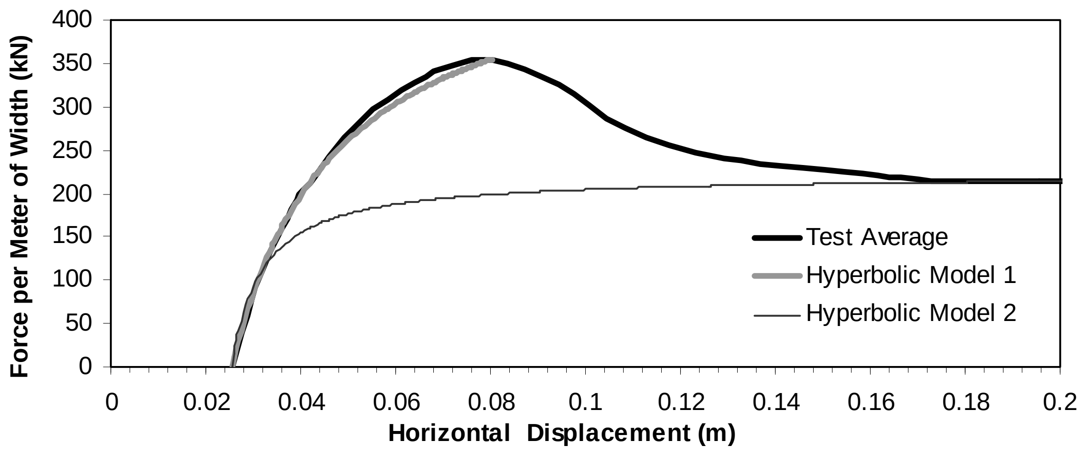

Figure 3: Average force-displacement curve from the two static pushover tests with added expansion gap

In these tests, the measured load increased until it reached a peak and then decreased to a steady-state level (Figure 3). The wall was observed to move slightly upward as it was displaced horizontally, resulting in low mobilization of friction between the wall and the backfill. This low mobilization of wall-soil friction resulted in measurement of a conservative force-displacement curve as discussed further in a later section.

Although the recorded behavior was similar for the two tests, the peak measured load was about 15% lower in the second test than in the first. This difference may be attributed to a more dried out backfill condition during the first test resulting from a much longer time between model construction and testing. The residual steady-state resistance was essentially the same for both tests. The measured peak loads were close to currently accepted peak passive earth pressure predictions (Wilson 2008).

The force-displacement relationship of Figure 3 (average of 2 tests) provides a possible representation of the abutment backfill resisting behavior. As mentioned earlier this curve was obtained for the 1.7 meter high test backfill (typical height of 5.5 feet according to Shamsabadi et al. 2007). In these tests, the plane-strain backfill soil friction angle was estimated at about 48 degrees (i.e., triaxial friction angle of about 40 degrees, Terzaghi et al. 1996). The mobilized wall-soil friction angle was on the order of 3 degrees (Wilson 2008).

2.3 Dynamic Excitation Tests

In the shake table experiments, earthquake-like motions were imparted in 1-D with the direction of shaking normal to the backwall face. These tests were conducted with the backwall approximately in its original (zero displacement or at-rest) position and also with the wall displaced into the backfill (with a portion of passive earth resistance already mobilized). A sine wave motion as well as a modified acceleration time history from the 1994 Northridge earthquake (Century City Station record) were scaled to different peak ground accelerations (up to 1.2g) and used as the input excitations.

Below (Figure 4) are results from a shake table excitation test that was performed after the backwall was pushed to mobilize a portion of passive resistance in the backfill (about 193 kN per meter of width). Figure 4 shows the total force measured by the load cells (Figure 2b) throughout a sine wave input motion with peak accelerations slightly larger than 0.5g. This experimental result clearly illustrates how the inertial forces imposed on the backfill and backwall during shaking caused the level of mobilized resistance to increase and decrease. Dynamic test results such as these suggest that the longitudinal resisting capacity at the abutments, provided by passive earth pressure, may be changing throughout the earthquake due to these inertial forces.

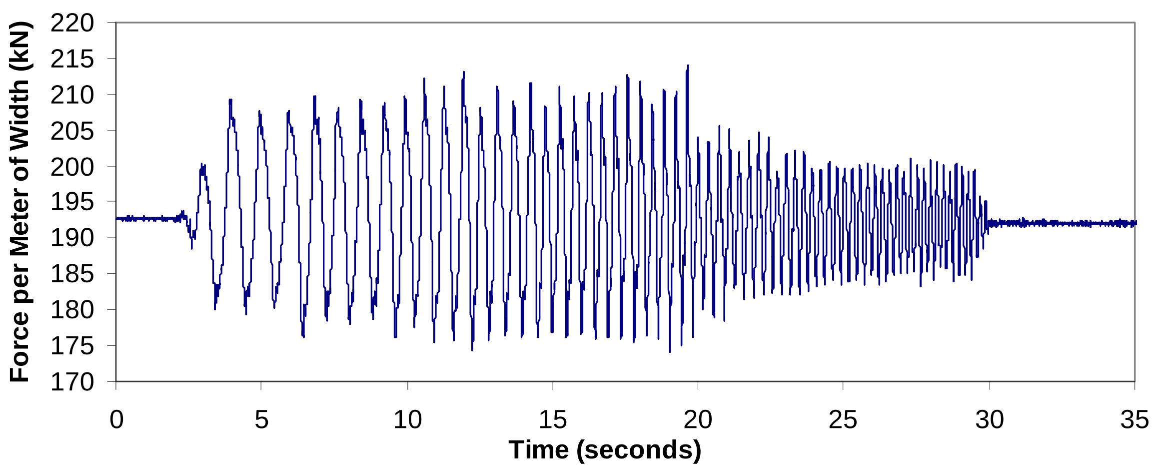

Figure 4: Total force measured by load cells throughout sine wave input motion

Figure 4 shows that the 0.5g shaking resulted in approximately plus or minus 20 kN per meter of wall width, or only about 6% of the peak passive resistance measured for the static backfill (Figure 3). Based on this consideration, and consistent with current design practice (Caltrans 2004, AASHTO 2007, Shamsabadi et al. 2007), the abutment models presented in the sections below are based only on the static backfill force-displacement capacity (Figure 3).

The above inertial effect may however need to be investigated further, particularly for larger accelerations. Results from several conducted dynamic excitation experiments with input accelerations of up to 1.2g are currently being analyzed using nonlinear finite element (FE) models. These models can later be implemented to represent the abutments with soil backfills in full bridge simulations.

3. HYPERBOLIC ABUTMENT STIFFNESS BACKBONE CURVES

Recent work by Shamsabadi et al. (2007) proposed a hyperbolic relationship for representing the nonlinear abutment stiffness in monotonic pushover analyses as given by

$$F = \frac{F_{ult}(2Ky_{\max} - F_{ult})y}{F_{ult}y_{\max} + 2(Ky_{\max} - F_{ult})y} \tag{1}$$

where F is the resisting force, y is the horizontal displacement, F_{ult} is the peak resisting force, K is the secant stiffness at $F_{ult}/2$ and y_{max} is the maximum horizontal displacement. Duncan and Mokwa (2001) also defined the following equivalent relationship

$$F = \frac{y}{\dfrac{1}{K_{\max}} + R_f \dfrac{y}{F_{ult}}} \tag{2}$$

where K_{max} is the initial tangent stiffness and R_f is the failure ratio (refer to Duncan and Mokwa 2001 for a description of the failure ratio).

The above hyperbolic curves have been shown to work well for representing measured force-displacement results from large scale passive earth pressure experiments up to the peak measured resistance (Duncan and Mokwa 2001, Cole and Rollins 2006, Shamsabadi et al. 2007). In this section, calibrated parameters which can be used with the hyperbolic formulae of Equations 1 and 2 are presented (Table 1 and Table 2). These parameters were determined based on the static pushover experimental results as well as from calibrated FE simulations as described below.

The model parameters of Tables 1 and 2 were calibrated for the 1.7 meter tall wall, per meter of abutment width, for the five scenarios described below. To scale for width, K or K_{max} and F_{ult} can be multiplied by the abutment width in meters. These models (Figure 3 and Figure 5) assume that no resistance occurs until the expansion gap has closed completely. Each model should be limited to a displacement range of y_{max} beyond the expansion gap length. Resistance further than this level is apt to decrease for a dense sand backfill (large strain shear strength reduction, Terzaghi et al. 1996).

Table 1: Calibrated parameters for hyperbolic backbone curve models using equation 1

Model	K, secant stiffness (kN/m/m)	F_{ult}, peak resisting force (kN/m)	y_{max}, displacement at F_{ult} (m)
1	14500	355	0.055
2	18000	215	0.2
3	12500	485	0.07
4	13000	550	0.075
5	15000	330	0.2

Table 2: Calibrated parameters for hyperbolic backbone curve models using equation 2

Model	K_{max}, initial stiffness kN/m/m	F_{ult}, peak resisting force (kN/m)	Rf	y_{max}, displacement at F_{ult} (m)
1	21500	355	0.7	0.055
2	32000	215	0.95	0.2
3	22000	485	0.95	0.07
4	18500	550	0.7	0.075
5	30000	330	0.95	0.2

Models 1 and 2 are based on the experimental results (Figure 3). As mentioned previously, the test configuration allowed the backwall to move upwards as it was pushed into the backfill. These models may be used as conservative estimates for the abutment stiffness, when restriction of vertical wall movement is not guaranteed, or when the wall-soil friction between the abutment backwall and backfill is unknown.

1) The first model (Figure 3) represents the average force-displacement relationship measured in the two static pushover test results up to the peak measured resistance. By limiting the displacement range (Figure 3), this model

is able to benefit from the pre-softening backfill peak strength. This benefit of added capacity may be applicable, for instance, to stiff bridge structures which do not exceed 8 cm of longitudinal bridge deck displacement in the design earthquake of interest.

2) The second model (Figure 3) considers the possibility of abutment displacements that reach beyond the level of the peak resistance range measured in the experiments. This model compromises some accuracy to conservatively rely on only the residual steady-state resistance measured in the experiments. Evidently it is also possible to numerically describe a spring model behavior that tracks the recorded pre-peak and post-peak softening behavior for added accuracy.

Models 3, 4 and 5 (Figure 5) are based on FE simulation (Plaxis 2D plane-strain model) results. This FE model was carefully developed and calibrated first based on the experimental results (Wilson 2008). During the calibration phase, the FE model backwall (rigid plate elements with the actual wall mass) was permitted to move vertically, as in the physical tests. To consider the possible scenario of a wall restricted from moving upwards (e.g., integral abutment case or due to large friction between the bridge deck and backwall), the FE model was then adapted by restricting the vertical backwall movement and adding an assigned interface friction between the model wall and backfill. Hyperbolic model parameters (Table 1 and Table 2) were then calibrated to match the FE simulation results.

3) Model 3 assumes that the wall-soil interface friction is 30% of the soil friction angle. Similar to the first model, by limiting the displacement range (Figure 5), the third model is able to benefit from the peak backfill strength.

4) The fourth scenario assumes greater wall-soil friction (50% of the soil friction angle) than the third with all other factors being the same. This added friction results in greater backfill resistance compared to model 3 (Figure 5). Larger implemented values of wall-soil friction were not observed to result in a further substantial increase in resistance in the FE simulations.

5) Similar to the second, the fifth model considers the possibility of displacements that reach beyond the level of the peak resistance range measured in the numerical simulations. This model was calibrated by employing the backfill residual plane-strain friction angle of 40 degrees (i.e., triaxial friction angle of about 34 degrees, Terzaghi et al. 1996) in the FE model. This model assumes wall-soil friction equal to 30% of the soil friction angle. Larger wall-soil friction did not result in a substantial increase in resistance in the FE simulation.

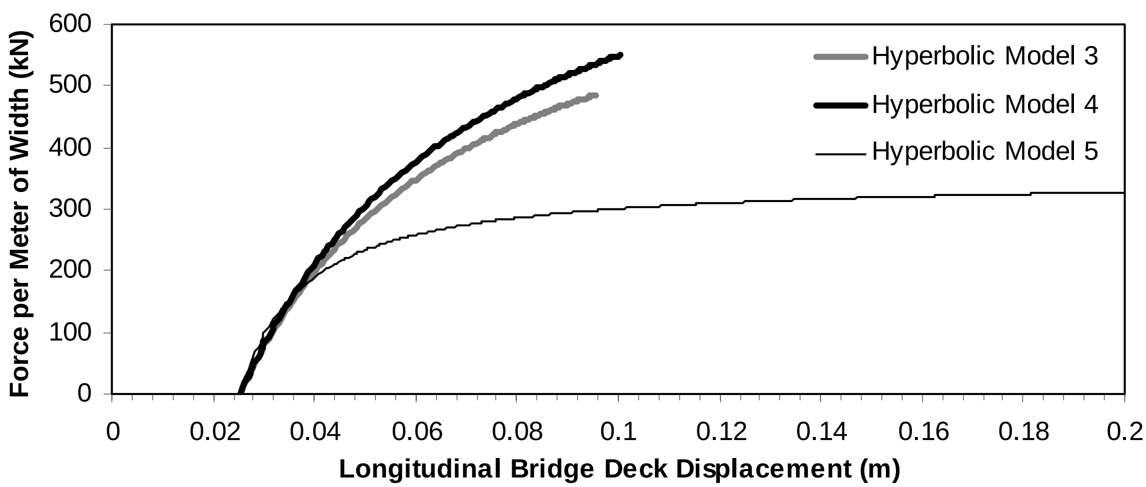

Figure 5: Hyperbolic models 3, 4 and 5

4. ABUTMENT CYCLIC LOADING SPRING MODELS

To apply the hyperbolic abutment stiffness backbone curves to dynamic bridge simulations, a cyclic loading model is presented in this section. In Figures 6 and 7 below, the model is described for two representative bridge deck longitudinal displacement cycles. Within the overall collaborative framework of the project (Saiidi 2004), this cyclic model has recently been implemented and is available for use in the FE platform OpenSees (Mazzoni et al. 2006). For that purpose a "HyperbolicGapMaterial" was developed and added to OpenSees by Matthew Dryden of the University of California at Berkeley, as a part of his PhD study under the supervision of Professor Gregory Fenves.

The cyclic model follows the hyperbolic backbone curve for virgin loading but adds an unloading and reloading stiffness K_{ur} for subsequent cycles (Figures 6 and 7). This model assumes that if the bridge deck pushes the abutment backwall into the backfill and then retreats, the backwall essentially remains at its furthest penetration (the small soil cohesion helps the deformed backfill to retain its shape). On subsequent loading cycles (Figure 7), it is assumed that the abutment loses its resisting capacity up to the point of prior unloading. A value of K_{ur} = 21500 kN/m (per meter of width) was estimated based on the initial stiffness and instances of unloading and reloading in the static pushover experiments (Wilson 2008). For the implemented model, $K_{ur} = K_{max}$ (Table 2) may also be adopted as the unloading and reloading stiffness.

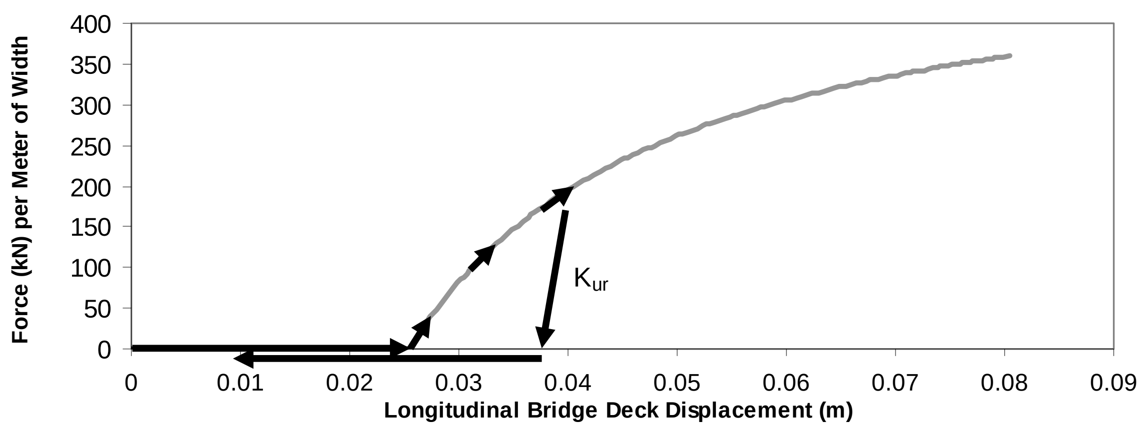

Figure 6: First cycle of representative abutment model

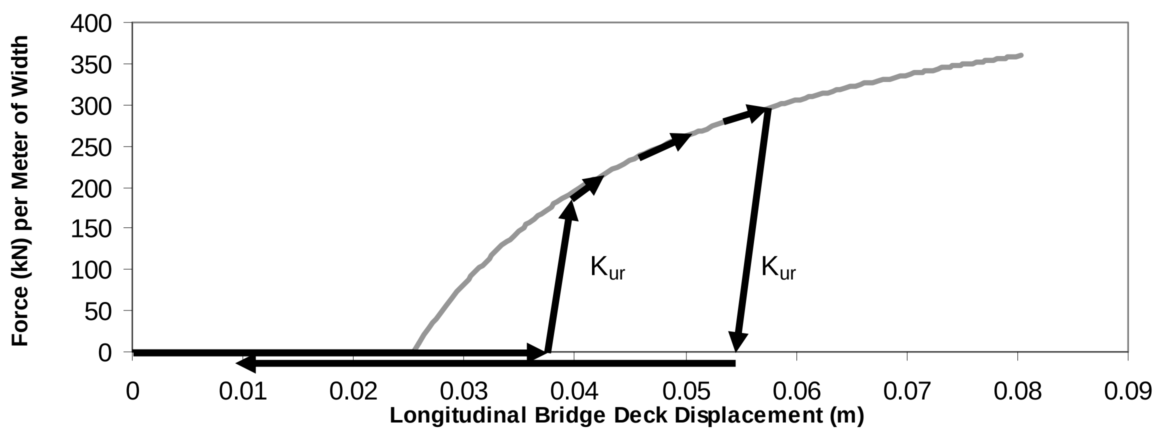

Figure 7: Subsequent cycle of representative abutment model

5. CONCLUSIONS

Recent full scale experiments to determine the bridge abutment longitudinal force-displacement resisting capacity have been described in this article. Static pushover test results provided a basis for developing hyperbolic model parameters to represent the abutment in monotonic pushover analyses. A cyclic loading model was described to employ the hyperbolic force-displacement relationships in dynamic computational analyses. Dynamic excitation test results suggest that earthquake induced inertial forces on the backfill may affect the abutment force-displacement resistance. This effect may however be small at shaking levels up to 0.5g. Finite element simulations are currently being employed to further investigate this issue.

ACKNOWLEDGEMENTS

Support for this research was provided by the National Science Foundation (NSF) NEESR grant number 0420347 under the overall direction of Professor M. Saiid Saiidi of the University of Nevada at Reno (UNR). The authors acknowledge Matthew Dryden and Professor Gregory Fenves (grant Co-PI) of the University of California at Berkeley for their implementation of the described cyclic loading model in OpenSees.

REFERENCES

AASHTO (2007). Proposed Guide Specifications for LRFD Seismic Bridge Design. American Association of State Highway and Transportation Officials, Washington D.C.

Caltrans (1999). "Standard specifications." California Dept. of Transportation, Sacramento, CA.

Caltrans (2004). "Seismic design criteria, version 1.3." California Dept. of Transportation, Sacramento, CA.

Cole, R., and Rollins, K. (2006). "Passive earth pressure mobilization during cyclic loading." *Journal of Geotechnical and Geoenvironmental Engineering*, 132(9), 1154-1164.

Duncan, M., and Mokwa, R. (2001). "Passive earth pressures: theories and tests." *Journal of Geotechnical and Geoenvironmental Engineering*, 127(3), 248-257.

Mazzoni, S., McKenna, F., and Fenves, G. (2006). *Open System for Earthquake Engineering Simulation User Manual.* Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA, USA.

Saiidi, M. S. (2004). "Seismic Performance of Bridge Systems with Conventional and Innovative Design." <http://nees.unr.edu/4-spanbridges/index.html> (Nov. 10, 2007).

Shamsabadi, A., Rollins, K., and Kapuskar, M. (2007). "Nonlinear soil-abutment-bridge structure interaction for seismic performance-based design." *J. of Geotechnical and Geoenvironmental Eng.*, 133(6), 707-720.

Terzaghi, K., Peck, R., and Mesri, G. (1996). *Soil mechanics in engineering practice, 3rd edition*, John Wiley and Sons, Inc. New York, NY.

Wilson, P. (2008). "Large-scale measurement and simulation of lateral earth pressure for bridge abutment and retaining wall seismic design." Ph.D. thesis, Dept. of Structural Eng., University of California, San Diego, CA (in progress).