UCLA

Title

Performance-based earthquake engineering applied to a bridge in liquefied and laterally spreading ground.

Permalink

Authors

Brandenberg, SJ
Kashighandi, P.

Publication Date

Peer reviewed

PERFORMANCE-BASED EARTHQUAKE ENGINEERING APPLIED TO A BRIDGE IN LIQUEFIED AND LATERALLY SPREADING GROUND

Scott J. Brandenberg[1], and Pirooz Kashighandi[2]

1) Assistant Professor, Dept. of Civil and Env. Engineering, University of California, Los Angeles, United States

2) Graduate Student, Dept. of Civil and Env. Engineering, University of California, Los Angeles, United States

sjbrandenberg@ucla.edu, pirooz@ucla.edu

Abstract： Fragility functions developed for liquefaction and lateral spreading against typical classes of bridges are integrated with a probabilistic lateral spreading ground displacement methodology in a performance-based earthquake engineering example problem. A site near UCLA is selected and a probabilistic seismic hazard analysis is performed to obtain a hazard curve expressing mean annual rate of exceedance of peak ground acceleration. The disaggregation of the seismic hazard curve is also computed. A liquefiable soil profile is selected, and hazard curve expressing mean annual rate of non-exceedance of factor of safety against liquefaction is computed from the seismic hazard curve and disaggregation. A hazard curve expressing mean annual rate of exceedance of lateral spreading ground displacement is then computed using a semi-empirical probabilistic framework. The ground displacement hazard curve is compared with a typical approach wherein a probabilistic ground motion is selected and the engineering calculations are performed deterministically. Finally, the fragility functions are applied and a hazard curve is computed that expresses the mean annual rate of exceedance of various engineering demand parameters. This example problem shows how performance-based earthquake engineering can be applied to liquefaction problems to better communicate uncertainty and risk to decision- and policy- makers.

1. INTRODUCTION

As of 2008, more than half of the earth's population lives in cities. Growth of our urban centers has placed a premium on sites with marginal or poor quality soils that had previously been considered inappropriate for development. This problem is particularly pertinent to geotechnical earthquake engineering because (1) a large fraction of the world's urban centers are in seismically active regions, (2) loose or soft soils have exhibited poor behavior due to cyclic failure and liquefaction in past earthquakes, causing death and billions of dollars in economic damages, and (3) our understanding of the seismic behavior of these soils is not well calibrated with meaningful experience because (thankfully) earthquakes are rare occurrences and few designers live to see how design-level shaking affects their projects. Geotechnical engineers strive to learn as much as possible from earthquakes as they occur around the world, but our evaluation procedures remain fraught with uncertainty. Considering how much uncertainty

geotechnical engineers encounter, an outsider would be justified in assuming that we are experts at quantifying and communicating risk to our clients. After all, other related fields with similar levels of uncertainty have embraced probabilistic methods. For example, consider the widespread adoption of probabilistic seismic hazard analysis in building codes. However, we have been reticent to adopt probabilistic methods in geotechnical engineering in large part because (1) there is a poor understanding of risk assessment among our community, and (2) we haven't automated our engineering evaluation procedures to permit the large number of realizations often required to integrate uncertainty in our calculations. As a result we are overly-reliant on "engineering judgment" that is not adequately calibrated with meaningful experience for earthquake applications, and we don't fully understand the risk associated with our design recommendations.

Liquefaction-induced lateral spreading and its effect on bridges is a geotechnical topic where statistical methods can help us quantify uncertainty and make better decisions. Lateral spreading occurs in gentle slopes or near a free face where soil is under a static driving shear stress, and an earthquake induces liquefaction in the loose saturated soil deposit and ground displacements accumulate in the direction of static driving shear stress during shaking. Ground displacements are often in the range of tens of centimeters to meters. When the static driving shear stress exceeds the undrained strength of the liquefied material, a flow slide occurs and the ground deformations can be very large (e.g., on the order of tens of meters). Lateral spreading hazard is particularly
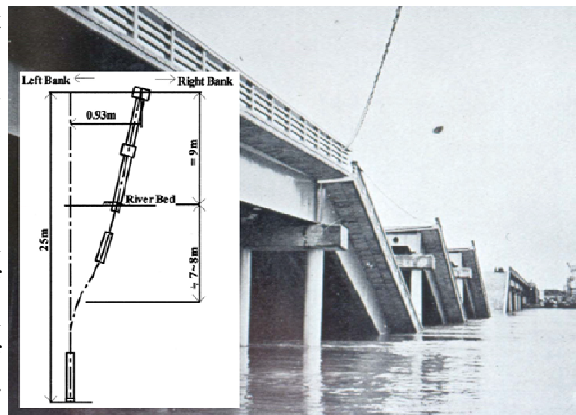

Figure 1: Collapse of Showa Bridge due to lique- faction and lateral spreading during 1964 Niigata earthquake (Yasuda and Berrill 2000).

pertinent for bridges that often cross bodies of water where liquefiable soil deposits are prevalent. Fig. 1 shows the Showa Bridge, which collapsed due to unseating of the simply supported spans caused by liquefaction and lateral spreading of the soils in which the piles were founded. Other bridges have performed reasonably well in lateral spreads. For example, the Landing Road Bridge suffered only moderate, repairable damage as a result of 2m of lateral spreading of the adjacent soil (Berrill et al. 2001), and several bridges were only slightly damaged due to tens of centimeters of ground displacements during the 2007 Niigata Ken Chuetsu-Oki earthquake (Kayen et al. 2007). This range of performance levels underscores the need for improved methodologies for predicting how much damage is anticipated due to liquefaction.

This paper demonstrates how the performance-based earthquake engineering can be applied to predict liquefaction-induced damage to bridges. An example problem consisting of a site with a corresponding seismic hazard curve and disaggregation is combined with a liquefiable soil profile to compute a hazard curve defining mean annual rate of exceedance of lateral spreading ground displacement. The ground displacement hazard curve is combined with recently-developed fragility functions to compute mean annual rate of exceedance of various bridge engineering demand parameters due to liquefaction and lateral spreading.

2. SITE AND SEISMIC HAZARD ANALYSIS

A site in Santa Monica, CA, (118.492°W, 34.015°N) was selected for this example problem. This is the same Santa Monica site analyzed by Kramer and Mayfield (2007), which provides a convenient means of validating the liquefaction hazard curve with their results. A probabilistic seismic hazard analysis was performed using OpenSHA (Field et al. 2003), with Vs_{30} = 300 m/s. The seismic hazard curve and magnitude disaggregation are shown in Figs. 2 and 3. The soil profile at the site consists of a 2m thick nonliquefied crust with unit weight γ = 18 kN/m³ lies over a clean liquefiable sand with $(N_1)_{60}$ = 10. The ground gently slopes at an angle of β = 2° and can be reasonably represented as an infinite slope.
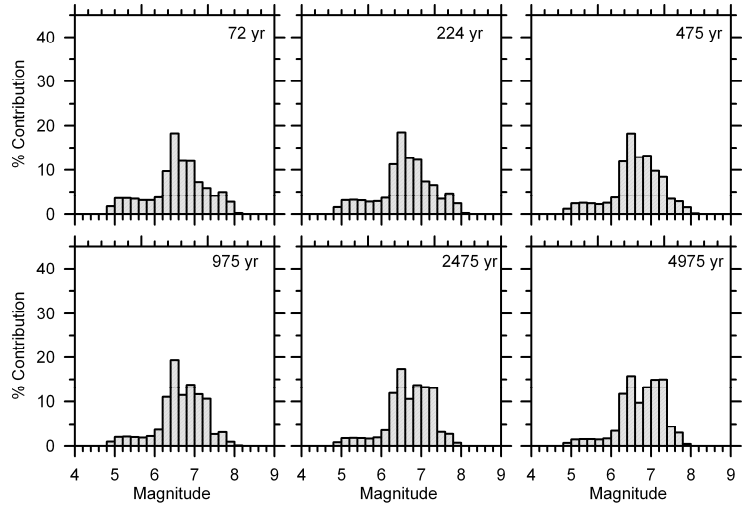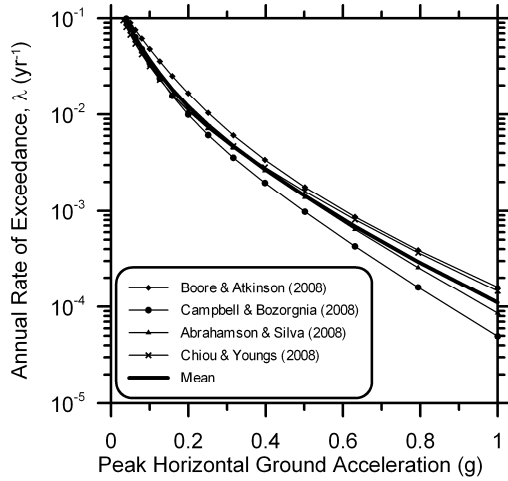

Figure 2: Hazard curves from probabilistic seismic hazard analysis of Santa Monica site.

Figure 3: Distributions of magnitude contributions to seismic haz-

3. LIQUEFACTION TRIGGERING EVALUATION

The next step in the analysis is computing the annual rate of exceedance of triggering of liquefaction. To simplify, the mean hazard curve from Fig. 2 will be used. Kramer and Mayfield (2007) outlined a framework for computing annual rate of non-exceedance of liquefaction that is adopted in this study. The approach is based on the probabilistic liquefaction triggering framework developed by Cetin et al. (2004), using the regression constants that account for measurement/estimation errors. Eq. 1 defines probability of factor of safety against liquefaction (FS_L) dropping below a value (FS_L^*) given $(N_1)_{60}$, fines content FC cyclic stress ratio CSR_{eq} moment magnitude M_w and vertical effective stress σ_{vo}'. The cyclic stress ratio is defined as $CSR_{eq} = 0.65(PGA/g)(\sigma_v/\sigma_{vo}')r_d$, where the stress reduction factor r_d was treated deterministically (Golesorkhi 1989). Uncertainty in r_d is anticipated to have negligible effect on the hazard analysis since the site is so shallow and r_d is near unity.

$$P\left[FS_L < FS_L^* \mid PGA, M_w\right] = \Phi\left[\frac{(N_1)_{60}(1+0.004FC) - 13.79\ln CSR_{eq} - 29.06\ln M_w - 3.82\ln(\sigma_{vo}'/p_a) + 0.06FC + 15.25}{4.21}\right] \tag{1}$$

$$\Lambda_{FS_L^*} = \sum_{j=1}^{N_{M_w}} \sum_{i=1}^{N_{PGA}} P\left[FS_L < FS_L^* \mid PGA_i, M_{w_j}\right] \Delta\lambda_{PGA,M_w} \qquad (2)$$

Peak horizontal ground acceleration (PGA) is not sufficient to characterize liquefaction triggering, and magnitude appears as well due to the influence of duration and frequency content. Hence, the hazard calculation must be integrated over PGA and M_w, which requires the disaggregation shown in Fig. 3. Eq. 2 defines the probability of non-exceedance of factor of safety against liquefaction, where the summations indicate discrete numerical integration over an adequate range of PGA and M_w values using the binning method wherein the probability density functions are divided into small slices for numerical integration (after Kramer and Mayfield 2007). Fig. 4 shows the mean annual rate of non-exceedance of factor of safety against liquefaction, which is similar to the Santa Monica site presented by Kramer and Mayfield (see Fig. 9 in their paper). The return period for $FS_L<1$ is about 100 years (i.e. Λ=0.01 yr^{-1}).
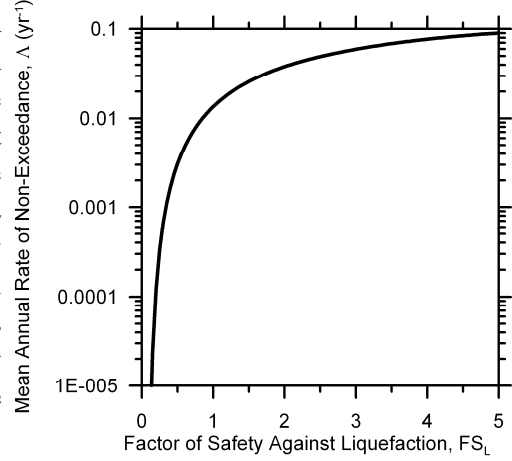
Fig. 4: Liquefaction triggering hazard curve.

4. GROUND DISPLACEMENT EVALUATION

The next step in the procedure is computing the mean annual rate of exceedance of lateral spreading ground displacement for this site. A number of methods for estimating lateral spreading displacements exist, and including multiple approaches is important for quantifying the effects of epistemic uncertainty. However, for simplicity only a single approach is utilized in this paper, though the methodology can easily be extended to other methods. The approach by Bray and Travasarou (2007) for computing permanent ground displacements is combined with the approach by Olson and Stark (2002) for estimating undrained residual strength of liquefied sand. For $(N_1)_{60} = 10$, the mean value of $s_r/\sigma_v' = 0.1$ based on the Olson and Stark suggestion, hence $\mu_{sr} = 0.1(2m)(18kN/m^3) = 3.6kPa$. Furthermore, the standard deviation is $\sigma_{sr} = 0.025(2m)(18kN/m^3) = 0.9kPa$. The static driving shear stress is $\tau_{stat} = (2m)(18kN/m^3)\sin(2°) = 1.3kPa$. If the static driving shear stress exceeds the undrained residual strength, then a flow slide occurs and ground displacement is large. Assuming that s_r is log-normally distributed, the probability of a flow slide can be computed using Eq. 3, where Φ is the standard normal cumulative distribution function.

$$P\left[Flow\,Slide \mid Liq\right] = \Phi\left[\frac{\ln\tau_{stat} - \ln s_r}{\sqrt{1 - \frac{\sigma_{sr}^2}{\mu_{sr}^2}}}\right] \qquad (3)$$

For cases when a flow slide does not occur, the lateral spreading ground displacement is be computed using the methodology of Bray and Travasarou (2007) defined in Eqs. 4, where k_y is the yield acceleration. For an infinite slope, $k_y = (s_r - \tau_{stat})/\gamma H\cos\beta$.

The probability of lateral spreading ground displacement exceeding some value, d, conditioned on the occurrence of liquefaction is given in Eq. 5, where the summation indicates numerical integration by the

$$P(D=0) = 1 - \Phi\big(-1.76 - 3.22\ln(k_y) + 3.52\ln(PGA)\big)$$

$$P(D>d \mid D>0) = 1 - \Phi\left(\frac{\ln(d) - \ln(\hat{d})}{0.66}\right)$$

$$\ln(\hat{d}) = -0.22 - 2.83\ln(k_y) - 0.333(\ln(k_y))^2 + 0.566\ln(k_y)\ln(PGA)$$
$$+ 3.04\ln(PGA) - 0.244(\ln(PGA))^2 + 0.278(M_w - 7)$$

$$(4)$$

$$P(D>d \mid Liq) = \sum_{i=1}^{N_{k_y}} [1 - P(D=0)][P(D>d \mid D>0)][1 - P(FlowSlide \mid Liq)]\Delta k_y + P(FlowSlide \mid Liq) \qquad (5)$$

binning method over the random variable k_y, which depends on random variable s_r.

The mean annual rate of exceedance of free-field lateral spreading ground displacement is computed by inserting the conditional probability defined in Eq. 5 in the hazard integral, as defined in Eq. 6.

$$\lambda_D = \sum_{j=1}^{N_M} \sum_{i=1}^{N_{PGA}} P(D>d \mid Liq)P(Liq \mid PGA, M_w)\Delta \lambda_{PGA,M_w} \qquad (6)$$

Fig. 5 shows the lateral spreading ground displacement hazard curve for the example problem, which was computed using 30 bins for PGA and k_y, and 17 bins for magnitude, for a total of 15,300 computations. Also shown in Fig. 5 are several values of ground displacement computed deterministically by taking the PGA associated with some hazard level combined with the modal magnitude (M_w = 6.5 in this case), mean k_y value, and mean lateral spreading displacement value computed using Eq. 4. In this case the deterministic approach underestimates the true ground displacement hazard primarily because (1) the modal magnitude was used and higher magnitudes contribute to larger displacements according to a nonlinear relation, and (2) the mean value of the liquefied undrained strength

Fig. 5: Free-field lateral spreading ground displacement haz-

was used and lower undrained strengths produce larger displacements according to a nonlinear relation. Kramer and Mayfield (2007) also showed how inconsistencies between the probabilistic and deterministic approaches to liquefaction triggering evaluation arise due to nonlinearities in the equations, and the mismatch depends on the slope of the hazard curve. These observations indicate that the return period associated with a design level ground motion may not be the same as the return period for a deterministically-computed engineering response parameter, and utilizing the performance-based approach is the only way to provide consistency.
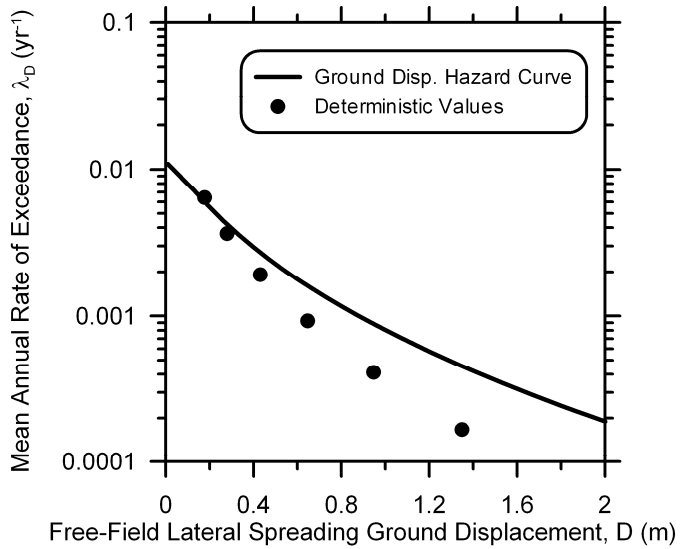
5. BRIDGE ENGINEERING DEMAND PARAMETER EVALUATION

Co-author Kashighandi recently defended his Ph.D. dissertation, which will be filed in the next few months. The topic of his dissertation was developing fragility functions for bridges in liquefied and laterally spreading ground to be used as a screening tool for identifying which set of Caltrans bridges is most susceptible to liquefaction. He developed demand fragility surfaces that define probability of exceeding some engineering demand parameter (EDP) (e.g., pier column curvature ductility, pile cap displacement, abutment displacement) as a function of free-field lateral spreading ground displacement. The fragility functions were developed using equivalent static global analysis, and the details are beyond the scope of this paper. Example demand fragility surfaces are shown in Fig. 6 for bridges constructed after 1971 with simply-supported spans, seat-type abutments, and 24" Cast in Drilled Hole deep foundations supporting the pile caps and abutments.

The conditional probabilities defined in the demand fragility surfaces were inserted into the hazard integral to define the mean annual rate of exceedance of the three EDP values (Eq. 7). The EDP hazard curves are plotted in Fig. 7. The 10% in 50 year EDP values ($\lambda = 2.1 \times 10^{-3}$ yr^{-1} and return period = 475yr) are pile cap displacement = $0.18m$, the pier column remains elastic, and abutment displacement = $0.15m$. These EDP hazard curves provide for better decision-making compared with the standard-of-practice approach of selecting a probabilistic ground motion and performing engineering calculations deterministically.
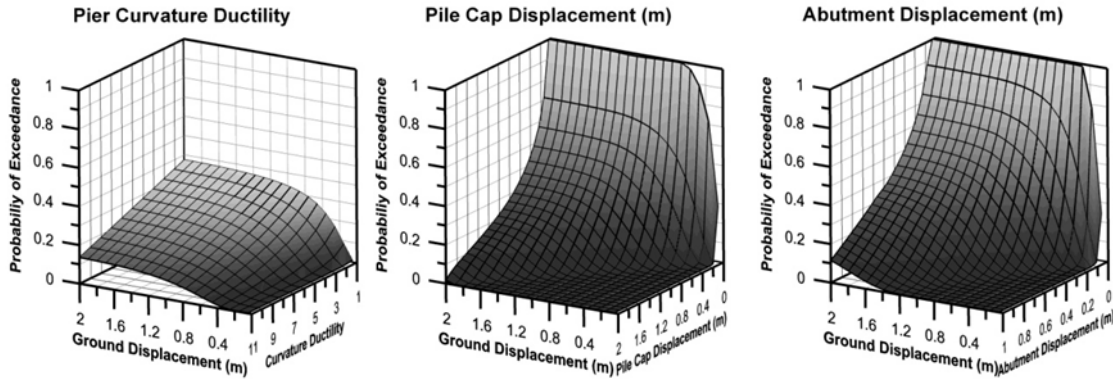

Fig. 6: Demand fragility surfaces for post-1971 bridges with simply-supported spans, seat-type abutments, and 24" CIDH piles.

$$\lambda_{EDP} = \sum_{j=1}^{N_M} \sum_{i=1}^{N_{PGA}} P(EDP > edp \mid D)P(D > d \mid Liq)P(Liq \mid PGA, M_w)\Delta\lambda_{PGA,M_w} \tag{7}$$

6. CONCLUSIONS

In this paper, a seismic hazard curve was integrated to obtain mean annual rate of non-exceedance of factor of safety against liquefaction, mean annual rate of exceedance of free-field lateral spreading ground displacement, and mean annual rate of exceedance of several engineering demand parameters that are meaningful for bridges. The procedure could be taken further, as defined in the framework of the Pacific Earthquake Engineering Research Center, to define a damage measure based on the engineering demand
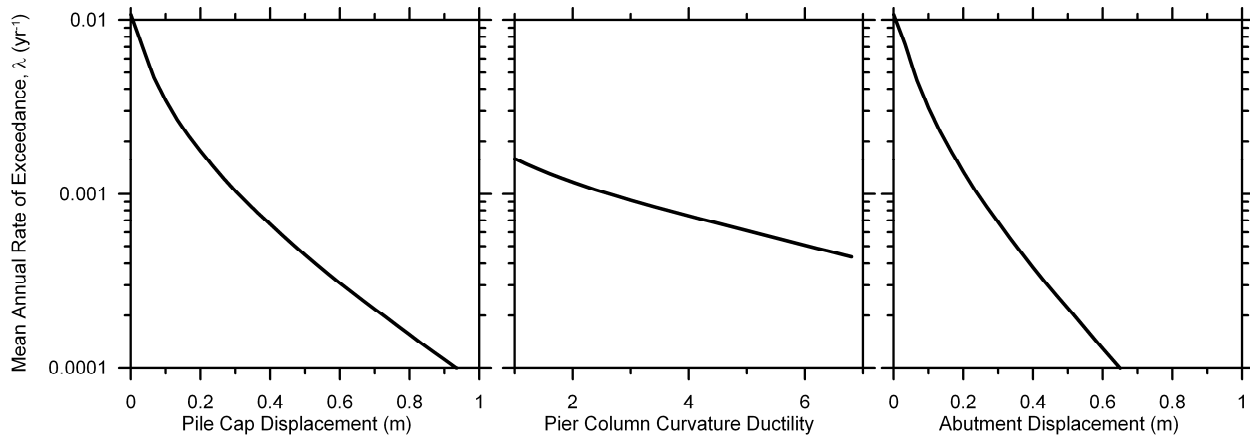
Fig. 7: Mean annual rate of exceedance of pile cap displacement, pier column curvature ductility, and abutment displacement.

parameters (e.g., exceeding a pier column curvature ductility of 7 results in total damage of the pier column), and a decision variable based on the damage measure (e.g., the cost of replacing a totally damage pier column is $X). Decisions could then be made based on mean annual rate of exceedance of dollar loss, which is a much more intuitive decision tool compared with making decisions based on mean annual rate of exceedance of PGA, lateral spreading displacement, or EDP's.

7. ACKNOWLEDGEMENTS

8. REFERENCES

Abrahamson, N., and Silva, W. (2008). "Summary of the Abrahamson & Silva NGA ground motion relations." *Earthquake Spectra*, EERI, 24(1), 67-98.

Berrill, J.B., Christensen, S.A., Keenan, R.P., Okada, W., and Pettinga, J.R. (2001). "Case study of lateral spreading forces on a piled foundation." Geotechnique, 51(6). 501-517.

Boore, D.M., and Atkinson, G.M. (2008). "Ground-motion prediction equations for the average horizontal component of PGA, PGV, and 5%-damped PSA at spectral periods between 0.01 s and 10.0 s." *Earthquake Spectra*, EERI, 24(1), 99-138.

Bray, J.D., and Travasarou, T. (2007). "Simplified procedure for estimating earthquake-induced deviatoric slope displacements." *J. Geotech. Geoenviron. Eng.*, ASCE, 133(4), 381-392.

Campbell, K.W., and Bozorgnia, Y. (2008). "NGA ground motion model for the geometric mean

horizontal component of PGA, PGV, PGD and 5% damped linear elastic response spectra for periods ranging from 0.01 to 10 s." *Earthquake Spectra*, EERI, 24(1), 139-172.

Cetin, K.O., Seed, R.B., Der Kiureghian, A., Tokimatsu, K., Harder, L.F. Jr., Kayen, R.E., and Moss, R.E.S. (2004). "Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential." *J. Geotech. Geoenviron. Eng.*, ASCE, 130(12), 1314-1340.

Chiou, B.S.-J., and Youngs, R.R. (2008). "An NGA model for the average horizontal component of peak ground motion and response spectra." *Earthquake Spectra*, EERI, 24(1), 173-216.

Field, E.H., T.H. Jordan, and C.A. Cornell (2003). OpenSHA: A Developing Community-Modeling Environment for Seismic Hazard Analysis", Seismological Research Letters Volume 74, Number 4, pages 406-419.

Golesorkhi, R. (1989). "Factors influencing the computational determination of earthquake-induced shear stresses in sandy soils." PhD. Dissertation, Univ. of California, Berkeley, Calif.

Kayen, R., Collins, B., Abrahamson, N., Ashford, S., Brandenberg, S.J., Cluff, L., Dickenson, S., Johnson, L., Tanaka, Y., Tokimatsu, K., Kabeyasawa, T., Kawamata, Y., Koumoto, H., Marubashi, N., Pujol, S., Steele, C., Sun, J. I., Tsai, B., Yanev, P., Yashinsky, M., and Yousok, K. (2007). Investigation of the M6.6 Niigata-Chuetsu Oki, Japan, earthquake of July 16, 2007: U.S. Geological Survey Open-File Report 2007-1365, 230 p.

Kramer, S.L., and Mayfield, R.T. (2007). "Return period of soil liquefaction." *J. Geotech. Geoenviron. Eng.*, ASCE, 133(7), 802-813.

Olson, S.M. and Stark, T.D. (2002). "Liquefied strength ratio from liquefaction case histories." *Canadian Geotechnical Journal*, 39, 629-647.