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PERFORMANCE-BASED ASSESSMENT OF LIQUEFACTION HAZARDS

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ABSTRACT

Advances in the development of performance-based earthquake engineering, which seeks to predict the seismic performance of structures and facilities in ways that are useful to a wide variety of stakeholders, offer important opportunities for more rational, objective, and consistent evaluation of liquefaction hazards. Performance-based procedures for evaluation of liquefaction potential have been shown to provide more consistent and accurate indications of the actual likelihood of liquefaction in areas of different seismicity than conventional procedures. These procedures can be extended to include effects of liquefaction such as lateral spreading and post-liquefaction settlement. This paper reviews the evolution of performance-based earthquake engineering, discusses the notion of performance and its description, and describes a recently developed framework for performance-based liquefaction hazard evaluation. The integration of procedures for estimation of liquefaction susceptibility, potential, and effects are described. The procedures are illustrated with an example of the evaluation of post-liquefaction settlement hazards.

Introduction

Liquefaction hazards are generally assessed by evaluating (a) the susceptibility of a soil deposit to liquefaction, (b) the potential for initiation of liquefaction under some anticipated loading, and (c) the effects of liquefaction. The level of loading considered in the assessment is usually expressed in terms of a ground motion intensity measure at a single return period. In practice, liquefaction susceptibility, potential, and effects are usually evaluated deterministically.

The development of performance-based earthquake engineering concepts allow probabilistic evaluations of susceptibility, initiation, and effects to be combined with a probabilistic evaluation of ground motion hazards to produce more rational and consistent estimates of liquefaction hazards. The purpose of this paper is to show how performance-based concepts can allow uncertainties in susceptibility, initiation, and effects to be incorporated into liquefaction hazard assessments.

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Conventional Procedures for Assessment of Liquefaction Hazards

The assessment of liquefaction hazards can be broken down into three primary components (Kramer, 1995) – evaluation of liquefaction susceptibility, evaluation of the potential for initiation of liquefaction, and evaluation of the effects of liquefaction. If liquefaction-susceptible soils are not present, or if the anticipated level of shaking is not strong enough to initiate liquefaction, liquefaction hazards do not exist and there is no need to evaluate the potential effects of liquefaction. If the soil is susceptible and the shaking strong enough to trigger liquefaction, however, then the performance of structures, lifelines, and other facilities will be strongly influenced by the effects of liquefaction, which must be explicitly evaluated.

Liquefaction Susceptibility

The primary difficulty in evaluating liquefaction susceptibility at this time lies in the susceptibility of fine-grained soils of marginal plasticity and coarse-grained soils with high fines contents. Following recent observations of liquefaction in fine-grained soils for which the Chinese criteria (Seed and Idriss, 1982) indicated non-susceptibility, extensive research on the liquefaction susceptibility of fine-grained soils was undertaken. At this stage, two major studies have proposed criteria for evaluating the liquefaction susceptibility of fine-grained soils. These criteria are consistent for many conditions but differ for others; however, both indicate the unreliability of the Chinese criteria. Both were developed using the results of field observations and laboratory tests by well-respected leaders of the geotechnical engineering profession.

Boulanger and Idriss

Boulanger and Idriss (2005) reviewed case histories and laboratory tests involving cyclic loading of different fine-grained soils. Boulanger and Idriss identified two types of behavior that they described as “sand-like” and “clay-like” on the basis of stress normalization and stress-strain behavior. Soils exhibiting sand-like behavior were considered susceptible to liquefaction and soils exhibiting clay-like behavior were not, although Boulanger and Idriss pointed out that some (e.g., sensitive clays) may be susceptible to behavior that can lead to earthquake damage. Boulanger and Idriss found that soil plasticity characteristics determined whether an individual soil was likely to exhibit sand-like or clay-like behavior, and proposed that the distinction could be made based on plasticity index, *PI*.

To quantify the transitional nature of observed sand-like to clay-like behavior, a numerical relationship can be established. The *PI* transition from clay-like to sand-like behavior from Boulanger and Idriss (2005) can be described using a susceptibility index, defined as

$$S_{BI} = \left[1 + \left(\ln PI/1.843\right)^{11.483}\right]^{-2.0} \qquad (1)$$

which has a value of 0.0 for clay-like (non-susceptible) behavior and 1.0 for sand-like (susceptible) behavior, and varies in a manner consistent with that shown graphically by Boulanger and Idriss (2005).

Bray and Sancio (2006)

Bray and Sancio (2006) investigated fine-grained soils that liquefied during 1994 Northridge, 1999 Kocaeli, and 1999 Chi-Chi earthquakes and proposed new compositional criteria for liquefaction susceptibility evaluation. In addition to plasticity index, Bray and Sancio found the ratio of water content to liquid limit (w_c/LL) to also influence liquefaction susceptibility.

A function similar to that used to approximate the Boulanger and Idriss criterion can be developed to quantify Bray and Sancio's susceptibility criteria. The equation is simply the product of two terms which have same general form as Equation 1, i.e.,

$$S_{BS} = \left[1 + (\ln PI/2.778)^{33.077}\right]^{-2.0}\left[1 + (4.401/\ln(w_c/LL))^{360.471}\right]^{-2.0} \quad (2)$$

These equations were determined by assuming the boundary between susceptibility and non-susceptibility to be uniformly distributed within the 'moderately susceptible' zone of Bray and Sancio, and fitting a function that would have the same mean and variance with respect to both PI and w_c/LL. As in Equation 1, a value of 0.0 indicates non-susceptibility and 1.0 indicates susceptibility.

Initiation of Liquefaction

Liquefaction potential is generally evaluated by comparing consistent measures of earthquake loading and liquefaction resistance. It has become common to base the comparison on cyclic shear stress amplitude, usually normalized by initial vertical effective stress and expressed in the form of a cyclic stress ratio, *CSR*, for loading and a cyclic resistance ratio, *CRR*, for resistance. The potential for liquefaction is then described in terms of a factor of safety against liquefaction,

$$FS_L = CRR/CSR \quad (3)$$

Characterization of Earthquake Loading

The cyclic stress ratio is most commonly evaluated using the "simplified method" first described by Seed and Idriss (1971), which can be expressed as

$$CSR = 0.65\frac{a_{\max}}{g}\cdot\frac{\sigma_{vo}}{\sigma'_{vo}}\cdot\frac{r_d}{MSF} \quad (4)$$

where a_{max} = peak ground surface acceleration, g = acceleration of gravity (in same units as a_{max}), σ_{vo} = initial vertical total stress, σ'_{vo} = initial vertical effective stress, r_d = depth reduction factor, and MSF = magnitude scaling factor, which is a function of earthquake magnitude. It should be noted that two pieces of ground motion information, a_{max} and magnitude, are required for estimation of the cyclic stress ratio.

Characterization of Liquefaction Resistance

The cyclic resistance ratio is generally obtained by correlation to insitu test results, principally from standard penetration (SPT), cone penetration (CPT), or shear wave velocity (V_s) tests. Of these, the SPT has been most commonly used and will be used in the remainder of this paper. A number of SPT-based procedures for deterministic (Seed and Idriss, 1971; Seed et al., 1985; Youd et al., 2001, Idriss and Boulanger, 2004) and probabilistic (Liao et al., 1988; Cetin et al., 2004) estimation of liquefaction resistance have been proposed.

Effects of Liquefaction

The initiation of liquefaction can have several effects that can be damaging to buildings, bridges, dams, embankments, lifelines and other facilities. The most significant of these are usually associated with permanent horizontal and vertical deformations that develop as a result of the softening and/or weakening of soils associated with excess porewater pressure development. Permanent vertical deformations in the form of post-earthquake settlement will be used to illustrate performance-based liquefaction hazard evaluation in the remainder of this paper.

The most common form of liquefaction-induced settlement is that which results from the volumetric compression that occurs when excess porewater pressures dissipate under level-ground conditions (i.e., when shearing deformations are insignificant). The contractive nature of sands subjected to vibratory loading has been recognized for many years.

Tokimatsu and Seed (1987) reviewed previous laboratory test data, which showed post-liquefaction volumetric strain to be related to relative density and peak shear strain. They then related relative density to SPT resistance and peak shear strain to cyclic stress ratio to develop curves relating volumetric strain to $(N_1)_{60}$ and *CSR* (Figure 1a). The curves in Figure 1(a) show that post-liquefaction volumetric strain increases with increasing loading and decreasing SPT resistance and suggest that volumetric strains can be as large as 10 percent in extremely loose sands. They also show that, for strong levels of shaking, the soil reaches a limiting volumetric strain. The Tokimatsu and Seed procedure computes ground surface settlement by integrating volumetric strain over the depth of the liquefiable layer, i.e., as

$$\Delta H = \int \varepsilon_v dz \qquad (5)$$

Other investigators (Ishihara and Yoshimine, 1992; Shamoto et al., 1998; Wu and Seed, 2004) have proposed similar relationships; that of Wu and Seed (2004) is shown in Figure 1(b). This relationship, like several others, does not suggest the existence of a limiting volumetric strain for moderately dense soils (corrected SPT resistances above about 10). Recently, Cetin et al. (2009) proposed a probabilistic procedure for estimation of volumetric strain following liquefaction that indicates lower uncertainty than the preceding procedures.

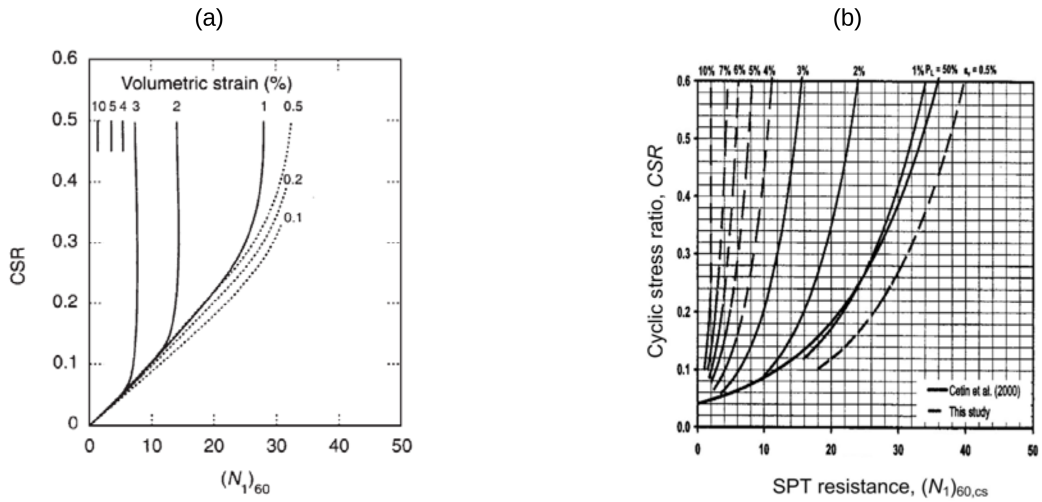

Figure 1. Variation of volumetric strain with corrected SPT resistance and cyclic stress ratio (a) Tokimatsu and Seed (1987), and (b) Wu and Seed (2004).

Performance-Based Liquefaction Hazard Assessment

In practice, liquefaction hazards are usually evaluated for a single hazard level, for example, for ground motions with a 10% probability of exceedance in a 50-yr period (475-yr return period). In some cases, two hazard levels may be considered, but different performance objectives (e.g. minimum factors of safety) would typically be required for the different hazard levels. For a given performance objective, only one hazard level is usually considered. In reality, liquefaction hazards can be caused by a wide range of ground shaking levels ranging from weaker motions that occur relatively frequently to stronger motions that occur only rarely.

Performance-Based Earthquake Engineering (PBEE) is generally formulated in a probabilistic framework to evaluate the risk associated with earthquake shaking at a particular site. The risk can be expressed in terms of economic loss, fatalities, or other measures. Pacific Earthquake Engineering Research (PEER) Center has developed a probabilistic framework for PBEE (Cornell and Krawinkler, 2000; Krawinkler, 2002).

The PEER PBEE framework computes risk as a function of ground shaking through the use of several intermediate variables. The ground motion is characterized by an *Intensity Measure, IM,* which could be any of a number of ground motion parameters. The effects of the *IM* on a system of interest are expressed in terms used primarily by engineers in the form of *Engineering Demand Parameters,* or *EDP*s. For a liquefiable site, the geotechnical engineer's initial contribution to this process for evaluating liquefaction hazards comes primarily in the evaluation of the conditional probability distribution of *EDP* given *IM*. In the PEER framework, the mean annual rate of exceeding some $EDP = edp$ is given by

$$\lambda_{EDP}(edp) = \sum_{i=1}^{N_{IM}} P[EDP > edp \mid IM = im_i]\Delta\lambda_{IM}(im_i) \tag{6}$$

Within the context of the previously described susceptibility/initiation/effects evaluations for liquefaction hazard assessment, the effects of liquefaction (e.g., settlement) would be taken as the *EDP*, and the susceptibility and initiation considerations would need to be addressed as intermediate steps in the evaluation of liquefaction effects. The evaluation of the conditional probability of *EDP* given *IM* would then take the following form

$$P[EDP > edp \mid IM] = P[EDP > edp \mid IM, initiation] P[initiation \mid IM, susceptibility] P[susceptbility] \quad (7)$$

The evaluation of this conditional distribution therefore requires evaluation of the probability that a given element of soil is susceptible to liquefaction, the probability that liquefaction will be triggered by ground motions of various intensities, and the probability that some level of effects will be reached given the intensity level and the fact that liquefaction has been initiated.

Susceptibility

The susceptibility indices given by Equations 1 and 2 can be interpreted in different ways. Neither were developed in a sufficiently formal manner as to represent probabilities of liquefaction susceptibility. However, they could be interpreted as degrees of belief, or subjective probabilities, that a soil would be susceptible to liquefaction in that they have values of zero for conditions in which Boulanger and Idriss (2005) and Bray and Sancio (2006) indicate non-susceptibility and value of 1.0 where they indicate susceptibility. Differences between the two approaches could be treated as epistemic uncertainty using a logic tree.

Initiation

As indicated previously, a number of probabilistic liquefaction initiation models have been proposed over the past 20 years. The model of Cetin et al. (2004) provides a good example of such methods. For a soil of a given density, the Cetin et al. (2004) model allows computation of a probability of liquefaction initiation for a liquefaction-susceptible soil as

$$P[initiation \mid IM, susceptbilty] = \Phi\left[-\frac{(N_1)_{60}(1+\theta_1 FC) - \theta_2 \ln CSR_{eq} - \theta_3 \ln M_w - \theta_4 \ln\left(\sigma'_{vo}/p_a\right) + \theta_5 FC + \theta_6}{\sigma_\varepsilon}\right] \quad (8)$$

where Φ is the standard normal cumulative distribution function, $(N_1)_{60}$ = corrected SPT resistance, *FC* = fines content (in percent), CSR_{eq} = cyclic stress ratio (Equation 4 without *MSF*, which serves as the *IM* in this case), M_w = moment magnitude, σ'_{vo} = initial vertical effective stress, p_a is atmospheric pressure (in same units as σ'_{vo}), σ_ε is a measure of the estimated model and parameter uncertainty, and θ_1-θ_6 are model coefficients obtained by regression. In this form, *CSR* and M_w form a vector *IM*.

Effects

Huang (2008) developed a probabilistic post-liquefaction settlement model based on interpretation of previous laboratory test results presented by Tokimatsu and Seed (1987), Ishihara and Yoshimine (1992), Shamoto et al., (1998), and Wu and Seed (2004). The relationship between SPT resistance, cyclic stress ratio, and median vertical strain, $\hat{\varepsilon}_v$, based on Wu and Seed's graphical model was expressed as

$$(N_1)_{60,cs} = \frac{CSR + D}{A + B(CSR + D)} \tag{9}$$

where $A = 0.002152\hat{\varepsilon}_v + 0.003322$, $B = b_1\hat{\varepsilon}_v{}^2 + b_2\hat{\varepsilon}_v + b_3$, $b_1 = -0.00003725\hat{\varepsilon}_v^2 - 0.0001581\hat{\varepsilon}_v + 0.004152$, $b_2 = 0.0007936\hat{\varepsilon}_v^2 - 0.002052\hat{\varepsilon}_v + 0.002547$, $b_3 = -0.001089\hat{\varepsilon}_v^2 - 0.0006875\hat{\varepsilon}_v + 0.01696$, and $D = -0.0123652\hat{\varepsilon}_v + 0.02709$. Equation 9 requires an iterative solution for $\hat{\varepsilon}_v$ as a function of SPT resistance and cyclic stress ratio. The mean value of ln ε_v can then be computed as $\mu_{\ln \varepsilon v} = \ln(\hat{\varepsilon}_v)$, which allows the vertical strain fragility relationship for a given *CSR* and $(N_1)_{60}$ to be described by

$$P[\varepsilon_v > \varepsilon_v^* \mid CSR\ , N] = \Phi\left[\frac{\mu_{\ln \varepsilon v} - \ln \varepsilon_v^*}{\sigma_{\ln \varepsilon_v}}\right] \tag{10}$$

where $\Phi(\cdot)$ is the standard normal cumulative distribution function.

The fact that the Wu and Seed (2004) curves do not become vertical at high *CSR* levels implies that volumetric strain continues to increase without bound with increasing *CSR*. However, considerable experimental evidence suggests that the continued vibration of soil leads to densification only to some limiting void ratio or density. Therefore, there must exist some limiting volumetric strain for a soil of a given initial density. Because the performance-based approach integrates response over all levels of ground motion hazard (Equation 6), extrapolating the Wu and Seed (2004) curves to the very high *CSR* values that can exist in areas of high seismicity can lead to unrealistically high volumetric strain levels. Huang (2008) reviewed data from compaction, minimum void ratio, drained cyclic tests, and consolidation tests following cyclic loading to suggest a tentative relationship between maximum volumetric strain and SPT resistance. The relationship can be approximated by

$$\bar{\varepsilon}_{v,\max}(\%) = 9.765 - 2.427\ln[(N_1)_{60,cs}] \tag{11}$$

Recognizing the approximate nature of this relationship, Huang (2008) assumed that $\varepsilon_{v,\max}$ was uniformly distributed over a range of $0.5\,\bar{\varepsilon}_{v,\max}$ to $1.5\,\bar{\varepsilon}_{v,\max}$.

Example

To illustrate the performance-based settlement evaluation procedure, a simple, hypothetical site in Seattle, Washington (Figure 2) is assumed. The site consists of 6 m of loose, silty sand with a groundwater level 1 m below the surface. The sand has a corrected clean sand SPT resistance, $(N_1)_{60,cs} = 15$ and PI = 5.

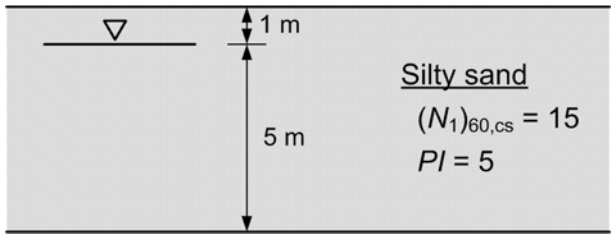

Figure 2. Hypothetical soil profile in Seattle, Washington.

To illustrate the roles of susceptibility, initiation, and maximum volumetric strain, the post-liquefaction settlement hazard for the site is computed for four different cases. Case 1 assumes that P[*susceptibility*] = 1.0, P[*initiation*] = 1.0, and that no maximum volumetric strain exists. For this case, the curves of Wu and Seed (2004) are extrapolated to large *CSR* values. Case 2 is identical to Case 1, except that the distribution of maximum volumetric strain is taken into account. Case 3 is the same as Case 2, except that the probability of initiation of liquefaction is also computed and accounted for. Finally, Case 4 accounts for the probability of liquefaction susceptibility, the probability of initiation, and the existence of a maximum volumetric strain.

The computed settlement hazard curves for all four cases are shown in Figure 3. For Case 1, the settlement hazard curve shows steadily increasing settlement with increasing return period (decreasing mean annual rate of exceedance). At long return periods, the unbounded settlement relationship implies settlement of over 0.5 m, which would correspond to average volumetric strains (over the 5 m saturated thickness) in excess of 10%. This volumetric strain would exceed that which has been observed in laboratory tests. The mean value of $\varepsilon_{v,\max}$ predicted by Equation 11 for such a soil would be approximately 3.2%, so the upper limit according to Huang (2008) would be less than 5%.

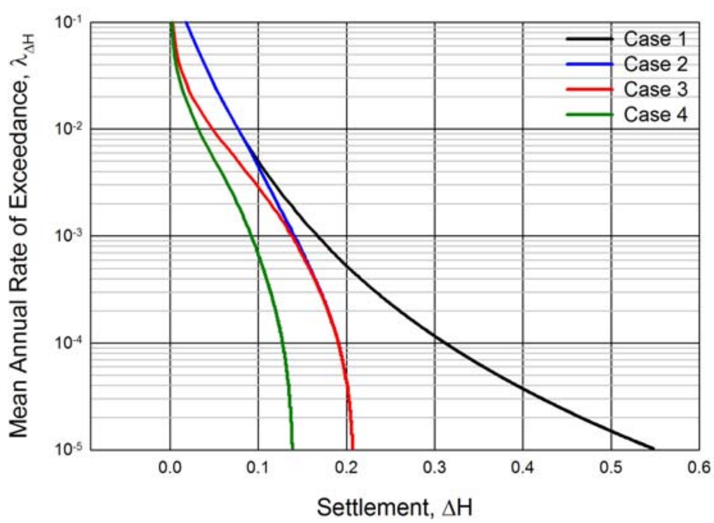

Figure 3. Post-liquefaction settlement hazard curves.

For Case 2, in which the existence of a maximum volumetric strain was considered, the settlement hazard curve matches the Case 1 curve at short return periods where volumetric strains do not approach the maximum value. At return periods beyond about 250 yrs, however, the shaking becomes strong enough that the maximum volumetric strain begins to affect the settlement hazard. At longer return periods, the Case 2 settlement hazard curve drops below the Case 1 curve and eventually becomes asymptotic to a maximum settlement slightly greater than 0.2 m. This settlement would correspond to an average maximum volumetric strain of about 4% under the strongest possible shaking, a value that is consistent with the results of laboratory tests.

The Case 2 curve assumes that liquefaction is initiated at all return periods. For the relatively weak shaking at short return periods, however, the probability of liquefaction may be less than 1.0. The Case 3 curve accounts for the probability of liquefaction initiation, and therefore drops below the Case 2 curve at short return periods. For return periods longer than about 1,000 years, the Case 3 curve converges to the Case 2 curve, as expected.

Finally, the settlement hazard curves for the first three cases all assume that the $PI = 5$ soil is susceptible to liquefaction. This PI level, however, would be in the transitional zone identified by Boulanger and Idriss (2005) and would be assigned a value of $S_{BI} = 0.68$ according to Equation 1. Taking S_{BI} as a subjective probability of susceptibility, the expected value of settlement drops to the Case 4 curve shown in Figure 3.

Summary and Conclusions

Performance-based concepts can allow objective and consistent evaluation of liquefaction hazards. By properly combining uncertainties in ground motions with uncertainties in liquefaction susceptibility, initiation, and effects, the actual return period for a given level of effects can be computed. The application of these concepts to the problem of post-liquefaction settlement prediction is illustrated in the paper.

The fact that performance-based settlement estimates are based on all levels of ground motion, including very strong levels that may only occur very rarely, requires the consideration of bounding values on volumetric strain. An approximate model for maximum volumetric strain was developed and implemented, and found to have a significant effect on post-liquefaction settlement at long return periods.

The implementation of this probabilistic settlement model into a performance-based framework that allows probabilistic characterization of liquefaction susceptibility and the potential for initiation of liquefaction allows the most consistent and objective estimates of post-liquefaction settlement hazards to be evaluated.

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