PERFORMANCE-BASED LIQUEFACTION POTENTIAL: A STEP TOWARD MORE UNIFORM DESIGN REQUIREMENTS

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Abstract

This paper describes a procedure for performance-based evaluation of liquefaction potential developed for the Washington State Department of Transportation. While conventional liquefaction hazard evaluation procedures focus on ground motions with a particular return period, a performance-based procedure allows consideration of ground motions with all return periods. The performance-based procedure and its results are compared and contrasted with those of conventional procedures for evaluation of liquefaction potential at 10 sites across the United States and on a grid of points across Washington state. Consistent application of conventional procedures is shown to produce inconsistent results when interpreted in terms of the actual probability of liquefaction. Given the implicit goal of codes and standards to produce designs with uniform risk across their regions of application, it is apparent that conventional procedures have some significant limitations. These limitations are discussed, and a framework for the use of performance-based procedures that better satisfy the goal of uniformity is presented.

Introduction

Methods for evaluation of liquefaction potential and criteria for liquefaction-resistant design are well established and have been used for many years. The development of probabilistic liquefaction models and of a probabilistic framework for performance-based earthquake engineering, however, allow a new approach to the specification of criteria for liquefaction-resistant design. This paper describes a recently developed performance-based procedure for evaluation of liquefaction potential and shows how it can be used to develop rational and consistent criteria for liquefaction-resistant design. The benefits and implications of such criteria are also discussed.

Liquefaction Potential

Liquefaction potential is generally evaluated by comparing consistent measures of earthquake loading and liquefaction potential. It has become common to base the comparison on cyclic shear stress amplitude, usually normalized by initial vertical

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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, $FS_L = CRR/CSR$. The procedure has been recommended by a number of organizations; for example, the California Division of Mines and Geology (CDMG, 1999) recommended using the National Center for Earthquake Engineering Research (NCEER) procedure described by Youd et al. (2001) with 475-yr peak ground accelerations (i.e. peak ground accelerations with a 10% probability of exceedance in a 50-yr period) and corresponding magnitudes. The CDMG report recommends that factors of safety should reflect site-specific conditions and the vulnerability of structures on the site; for loose, clean sands, it suggests factors of safety of 1.1 – 1.3 as being reasonable for sites at which liquefaction could cause settlement, lateral spreading, or related effects. For the purposes of this paper, the conventional criterion for a liquefaction-resistant site will be taken as $FS_L \geq 1.2$ for a 475-yr ground motion.

Although a number of approaches have been proposed in the literature, the NCEER procedure is most commonly used in practice. Recently, a detailed review and careful re-interpretation of liquefaction case histories (Cetin, 2000; Cetin et al., 2002; Cetin et al., 2004) was used to develop a new probabilistic procedure for evaluation of liquefaction potential. The Cetin et al. (2004) procedure can be used to predict the *CRR* value corresponding to a given probability of liquefaction, P_L, in the form

$$CRR = \exp\left[\frac{(N_1)_{60}(1+\theta_1 FC) - \theta_3 \ln M_w - \theta_4 \ln\left(\sigma'_{vo}/p_a\right) + \theta_5 FC + \theta_6 + \sigma_\varepsilon \Phi^{-1}(P_L)}{\theta_2}\right] \quad (1)$$

where $(N_1)_{60}$ = corrected SPT resistance, FC = fines content (in percent), 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, Φ^{-1} is the inverse standard normal cumulative distribution function, and θ_1-θ_6 are model coefficients obtained by regression. Mean values of the model coefficients and uncertainty, presented in Table 1, correspond to uncertainties that exist for a site investigated with a normal level of detail. Liquefaction resistance curves computed with Equation (1) are shown in Figure 1.

Arango et al. (2004) found that the Cetin et al. (2004) and NCEER procedures yielded similar values of FS_L for a site in San Francisco when P_L in Equation (1) was 0.65. A similar exercise for a site in Seattle shows equivalence of FS_L when $P_L \approx 0.6$. Cetin et al. (2004) suggest use of a deterministic curve equivalent to that given by Equation (1) with $P_L = 0.15$, which would produce a more conservative result than the NCEER procedure. For the purposes of this paper, the terminology "NCEER-C" will be used to describe a deterministic approximation to the NCEER model using Equation (1) with $P_L = 0.6$; it should be noted that this is not equivalent to the deterministic procedure recommended by Cetin et al. (2004).

Table 1. Cetin et al. (2004) model coefficients with measurement/estimation errors (after Cetin et al., 2002).

Coefficient	Value
θ_1	0.004
θ_2	13.79
θ_3	29.06
θ_4	3.82
θ_5	0.06
θ_6	15.25
σ_ε	4.21

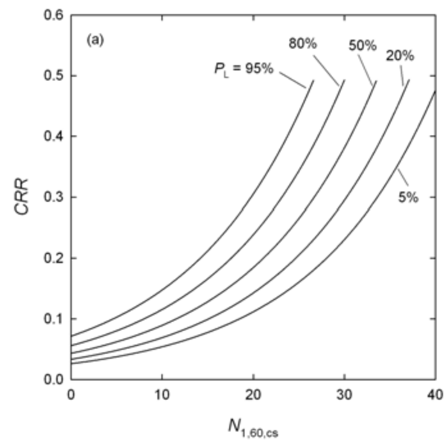

Figure 1. Curves of constant probability of liquefaction (Cetin et al., 2002).

Performance-Based Liquefaction Potential Evaluation

The Pacific Earthquake Engineering Research Center (PEER) has developed a probabilistic framework for performance-based earthquake engineering (PBEE) to evaluate the risk associated with earthquake shaking at a particular site (Cornell and Krawinkler, 2000; Krawinkler, 2002; Deierlein et al., 2003). The risk, expressed in terms of economic loss, fatalities or other measures, is computed as a function of ground shaking through a series of intermediate steps: the seismic hazard, characterized in terms of an *Intensity Measure*, *IM*, results in some form of response or demand hazard, characterized in terms of an *Engineering Demand Parameter*, *EDP*, at the site of interest. This *EDP* results in a damage hazard, which contributes to the overall risk. This paper focuses on the relationship between the *IM* hazard and the specific response hazard of liquefaction.

IM is a general term for describing the intensity of the earthquake shaking at the site of interest. In conventional liquefaction analyses, both peak ground acceleration and earthquake magnitude are required to estimate the liquefaction potential, thus there are two parts to the *IM*. The *EDP* can be represented by FS_L. By combining a probabilistic evaluation of FS_L with the *IM* from a probabilistic seismic hazard analysis, the mean annual rate of developing FS_L less than some selected factor of safety of interest, FS^*_L, can be computed as

$$\Lambda_{FS_L^*} = \sum_{i=1}^{N_{IM}} P[FS_L < FS_L^* \mid IM_i] \Delta\lambda_{IM_i} \tag{2}$$

where $\Delta\lambda_{IM_i}$ is the slope of the *IM* seismic hazard curve at IM_i. By integrating over the entire seismic hazard curve (approximated by the summation over i = 1, N_{IM}), the performance-based approach includes contributions from all return periods, not just the single return period mandated by typical codes or standards. The value of $\Lambda_{FS^*_L}$ should be interpreted as the mean annual rate (or inverse of the return period) at which the actual factor of safety, FS_L will be less than the selected factor of safety, FS^*_L. The mean annual rate of factor of safety non-exceedance is used because non-exceedance of a particular factor of safety represents an undesirable condition, just as exceedance of an intensity measure does; because lower case lambda is commonly used to represent mean annual rate of exceedance, an upper case lambda is used here to represent mean annual rate of non-exceedance. Since liquefaction is expected to occur when $CRR < CSR$ (i.e. when $FS_L < 1.0$), the return period of liquefaction corresponds to the reciprocal of the mean annual rate of non-exceedance of $FS^*_L = 1.0$, i.e. $T_{R,L} = 1/\Lambda_{FS^*_L=1.0}$.

Because *CSR* depends on both peak acceleration and magnitude, calculation of the mean annual rate of exceeding some factor of safety against liquefaction, FS^*_L, can be modified as

$$\Lambda_{FS^*_L} = \sum_{j=1}^{N_M} \sum_{i=1}^{N_{a_{\max}}} P[FS_L < FS^*_L \mid a_{\max i}, m_j]\Delta\lambda_{a_{\max i}, m_j} \tag{3}$$

where N_M and $N_{a_{\max}}$ are the number of magnitude and peak acceleration increments into which the "hazard space" is subdivided and $\Delta\lambda_{a_{\max i}, m_j}$ is the incremental mean annual rate of exceedance for intensity measure, $a_{\max i}$, and magnitude, m_j. The conditional probability term in Equation (3) can be calculated using the Cetin et al. (2004) model with $CSR = CSR_{eq,i}FS^*_L$ (where $CSR_{eq,i}$ = cyclic stress ratio without any magnitude correction computed from $a_{\max,i}$) and $M_w = m_j$, i.e.

$$P[FS_L < FS^*_L \mid a_{\max i}, m_j] = \Phi\left[-\frac{(N_1)_{60}(1+\theta_1 FC) - \theta_2 \ln\left(CSR_{eq,i}\, FS^*_L\right) - \theta_3 \ln m_j - \theta_4 \ln\left(\sigma'_{vo}/p_a\right) + \theta_5 FC + \theta_6}{\sigma_\varepsilon}\right] \tag{4}$$

Another way of characterizing liquefaction potential is in terms of the SPT resistance required to produce a desired level of performance. For example, the SPT value required to resist liquefaction, N_{req}, can be determined at each depth of interest. Given that liquefaction would occur when $N < N_{req}$, or when $FS_L < 1.0$, then $P[N < N_{req}] = P[FS_L < 1.0]$. The PBEE approach can then be applied in such a way as to produce a mean annual rate of exceedance for N^*_{req}

$$\lambda_{N^*_{req}} = \sum_{j=1}^{N_M} \sum_{i=1}^{N_{a_{\max}}} P[N_{req} > N^*_{req} \mid a_{\max i}, m_j] \Delta \lambda_{a_{\max i}, m_j} \tag{5}$$

where

$$P[N_{req} > N^*_{req} \mid a_{\max i}, m_j] = \Phi\left[-\frac{N^*_{req}(1+\theta_1 FC) - \theta_2 \ln CSR_{eq,i} - \theta_3 \ln m_j - \theta_4 \ln\left(\sigma'_{vo}/p_a\right) + \theta_5 FC + \theta_6}{\sigma_\varepsilon} \right]$$

The value of N^*_{req} can be interpreted as the SPT resistance required to produce the desired performance level for shaking with a return period of $1/\lambda_{N^*_{req}}$.

Illustration of Performance-Based Approach

Potentially liquefiable sites in different locations have different likelihoods of liquefaction due to differences in site conditions (which strongly affect liquefaction resistance) and local seismic environments (which strongly affect loading). The effects of seismic environment can be isolated by considering the liquefaction potential of a single soil profile placed at different locations.

Figure 2 shows the subsurface conditions for an idealized site with corrected SPT resistances that range from relatively low ($(N_1)_{60}$ = 10) to moderately high ($(N_1)_{60}$ = 30). In order to illustrate the effects of different seismic environments on liquefaction potential, the idealized site was assumed to be located in each of 10 U.S. cities. For each location, the local seismicity was characterized by the probabilistic seismic hazard analyses available from the U.S. Geological Survey using 2002 interactive deaggregation link with listed latitudes and longitudes. In addition to being spread across the United States, these locations represent a wide range of seismic environments; the total seismic hazard curves for each of the locations are shown in Figure 3.

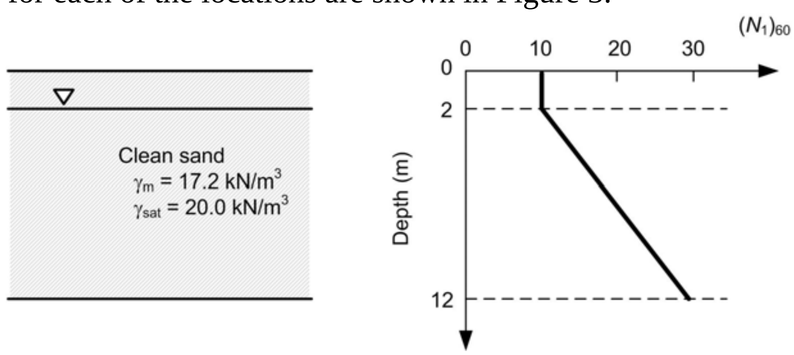

Figure 2. Subsurface profile for idealized site.

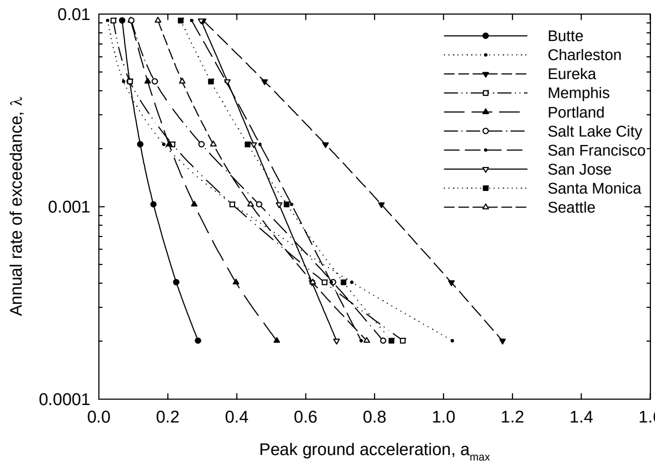

Figure 3. USGS total seismic hazard curves for different site locations.

The performance-based approach, which allows consideration of all ground motion levels and computation of liquefaction hazard curves, was applied to each of the site locations. In all analyses, the peak soft rock outcrop accelerations obtained from the USGS 2002 PSHAs were adjusted to approximate unconsolidated surface conditions using a Quaternary alluvium amplification factor (Stewart et al., 2003),

$$F_a = \frac{a_{\max,surface}}{a_{\max,rock}} = \exp\left[-0.15 - 0.13\ln a_{\max,rock}\right] \tag{6}$$

Figure 4 illustrates the results of the performance-based analyses for an element of soil at a depth of 6 m, at which $(N_1)_{60}$ = 18 for the hypothetical soil profile. Figure 4(a) shows factor of safety hazard curves, and Figure 4(b) shows hazard curves for N_{req}^{PB}, the SPT resistance required to resist liquefaction as calculated using the performance-based procedure.

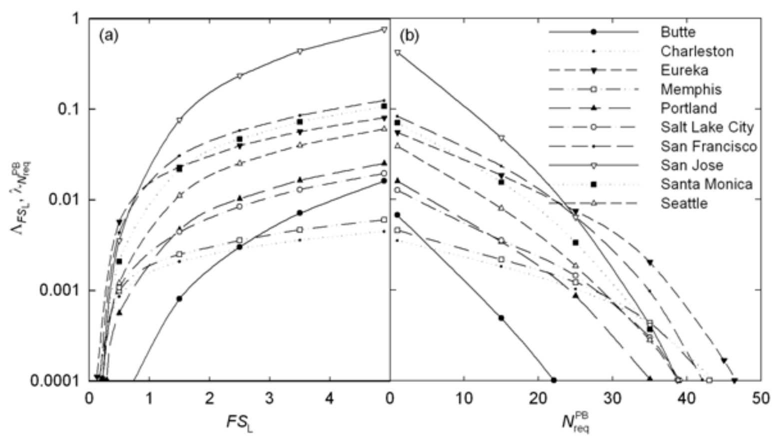

Figure 4. Seismic hazard curves for (a) factor of safety against liquefaction, FS_L, and (b) required SPT resistance, $N_{\mathbf{req}}^{\mathbf{PB}}$, at 6 m depth.

Development of a Performance-Based Liquefaction Criterion

As illustrated in Figure 3, different geographic regions have seismic hazard curves with different positions and shapes. The ground motion hazard curves for Portland and Memphis, for example, intersect at a point corresponding to a return period of about 475 yrs, which suggests that they would have similar values of FS_L and N_{req} using conventional liquefaction analyses. However, the hazard curve slopes are different and, as Figure 4(b) illustrates, the return periods of liquefaction (using $(N_1)_{60}$ = 18 at 6 m depth) are significantly different. This observation suggests that consistent rates of occurrence could be achieved by the use of performance-based criteria for liquefaction potential, i.e. by specifying adequate performance with respect to liquefaction in terms of a maximum rate of occurrence or, as preferred here, a minimum return period for liquefaction.

Benefits

The development and use of performance-based liquefaction criteria would have a number of benefits: (1) by considering ground motions corresponding to *all* return periods, regional differences in seismicity would be accounted for rationally and consistently; (2) by considering the contributions of all combinations of peak acceleration and magnitude at each return period, the issue of how to use magnitude de-aggregation information in a liquefaction evaluation becomes moot; (3) by allowing de-aggregation of the magnitudes that contribute to liquefaction hazard (rather than just magnitudes that contribute to peak acceleration), the magnitude values required by empirical models for prediction of the effects of liquefaction can be more logically selected; and (4) a single parameter such as N_{req} for a desired return period, which reflects all the performance-based calculations described in the previous section, can be mapped across large areas and adjusted using a simple calculation procedure to account for site-specific conditions.

Issues

Development of liquefaction criteria based on a minimum return period for liquefaction raises two primary questions: (1) what should that minimum return period be? and (2) which liquefaction evaluation procedure should the performance-based criterion be based on?

Design-Level Return Period

As engineering design has moved from deterministic to reliability-based criteria (e.g. LRFD), such questions have usually been explored by calibration exercises which involve determining the "failure" rate associated with accepted deterministic criteria. For the site locations considered in this paper, deterministic analyses were performed to evaluate N_{req}^{det}, i.e. the value of N_{req} that would produce FS_L = 1.2 using the NCEER-C model with 475-yr ground motions and mean magnitudes. Those $N_{req}^{\det}$ values were then used with Figure 4(b) to compute the actual return period of liquefaction corresponding to the deterministic criterion. The results of these analyses, shown in Figure 5, show that the actual return periods vary significantly. Hence, *consistent application of conventional liquefaction criteria can produce significantly inconsistent actual liquefaction hazards.*

Examination of the computed return periods for liquefaction indicates that the average return period implied by conventional criteria (average of return periods at 6 m depth) is 474 years, which indicates an average 10% probability of liquefaction in a 50-yr period. Recognizing that this applies to the 10 somewhat arbitrarily selected locations, and that the average could be different if other locations were considered, it nevertheless indicates a degree of consistency between the average return period of liquefaction and the return period of the ground motion on which the conventional liquefaction criterion is based. However, the return periods at the individual site locations range from about 340 to 580 yrs.

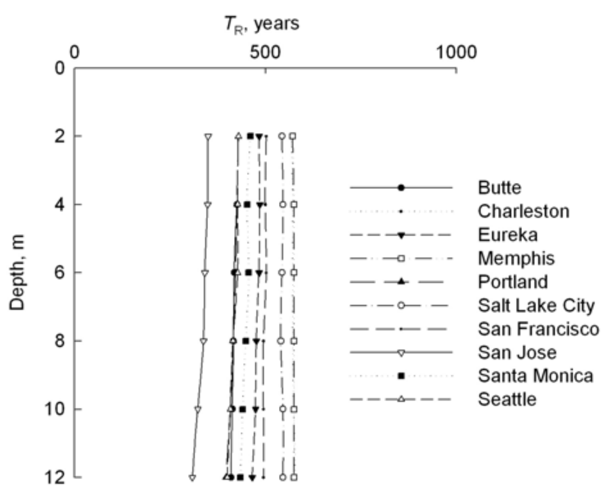

Figure 5. Profiles of return period of liquefaction for sites with equal liquefaction potential as evaluated by NCEER-C procedure.

Table 2. Required penetration resistances for 475-yr liquefaction hazard in element of soil at 6 m depth in hypothetical site at different site locations, and ratios of equivalent factors of safety.

Location	$N_{req}^{\det}$	N_{req}^{PB}	$\dfrac{CRR\left(N_{req}^{\det}\right)}{CRR\left(N_{req}^{PB}\right)}$
Butte	6.6	7.3	0.95
Charleston	15.9	12.1	1.32
Eureka	34.9	34.8	1.01
Memphis	19.4	15.7	1.31
Portland	17.9	18.8	0.94
SLC	22.5	21.1	1.11
San Fran.	31.8	31.5	1.02
San Jose	28.3	29.6	0.91
Santa Mon.	27.3	27.5	0.99
Seattle	23.5	24.2	0.95

Liquefaction Potential Model

The performance-based methodology described in this paper makes use of a recently developed procedure (Cetin et al., 2004) for estimation of the probability of liquefaction. While this procedure is very well suited for implementation into the performance-based methodology, other probabilistic liquefaction procedures could also be used. Because a performance-based criterion requires integration of PSHA and probabilistic liquefaction evaluation, either a consensus must be developed on a preferred probabilistic liquefaction model or on the use of a logic tree approach, in which multiple models are used with their results weighted as is routinely done in PSHA.

Implications

The previous analyses suggest that a performance-based criterion of a 475-yr return period for liquefaction would be consistent with the average results produced by conventional liquefaction criteria for the small sample of site locations considered in this paper. Obviously, much more extensive calibration calculations would need to be performed before adopting a specific return period criterion, but the 475-yr value will be assumed reasonable for the purposes of this paper.

The SPT resistances required to satisfy the conventional, deterministic criterion at a depth of 6 m are listed in Table 2. The SPT resistances required to satisfy the performance-based criterion at the same depth are also shown in Table 2. For those locations at which the return periods for liquefaction in Figure 4 are less than 475 yrs, the SPT resistances required for liquefaction with an actual return period of 475 yrs are increased, and vice versa for locations at which the Figure 4 return periods were greater than 475 yrs. For a location like Memphis, the relative conservatism in the conventional approach means that the required SPT resistance of 19.1 for FS_L = 1.2 with the 475-yr motion (conventional criterion) is reduced to an SPT resistance of 14.3 for a 475-yr return period of liquefaction (performance-based criterion). For Portland, the relative unconservatism in the conventional approach means that the required SPT resistance increases slightly from 17.0 (conventional criterion) to 17.5 (performance-based criterion).

Because cyclic resistance ratio varies nonlinearly with SPT resistance, it is also useful to consider the difference between the deterministic and performance-based approaches from a factor of safety standpoint. Since FS_L is proportional to *CRR*, the ratio of the *CRR* values corresponding to the N_{req} values in Table 2 can be thought of as factor of safety ratios that describe the "extra" liquefaction resistance required by the deterministic criterion relative to that required by the performance-based criterion. These ratios, also listed in Table 2, range from 0.91 to 1.32; higher values are associated with locations where conventional deterministic procedures produce more conservative results. In Memphis and Charleston, for example, the deterministic criterion would result in a factor of safety some 30% higher than that required for an actual liquefaction return period of 475 yrs. At several other locations, the deterministic criterion results in factors of safety *lower* than those required for the same actual liquefaction hazard.

Within Washington State, the benefits of return period-based criteria can be seen by comparing the distributions of N_{req} given by the deterministic and performance-based approaches. Figure 6(a) shows contours of the values of N_{req} for FS_L = 1.2 based on deterministic analyses using 475-yr peak acceleration and mean magnitude values. For

Seattle, the resulting value of N_{req} = 22 can be shown (using Figure 4(b)) to have a return period of 400 yrs. If that return period is taken as a reasonable criterion for acceptable performance, i.e. if the current level of resistance in Seattle is considered to be appropriate for all of the state, a map of N_{req} with a 400-yr return period (Figure 6(b)) can be constructed. Comparing Figures 6(a) and (b) shows little difference in N_{req} along and east of the I-5 corridor (I-5 runs through Vancouver, Olympia, Seattle, and Bellingham. To the west, however, the N_{req} values produced by the deterministic approach are significantly higher than those produced by the performance-based approach. The differences in required SPT resistance, i.e. $\Delta N = N_{req}^{\det} - N_{req}^{PB}$ are shown in Figure 7(a). These differences can be represented in terms of an equivalent factor of safety, contours of which are shown in Figure 7(b). Figure 7(b) shows that the use of conventional, deterministic procedures on the coast would produce a design with an apparent factor of safety 1.5 times higher than that produced by the same procedures in Seattle..

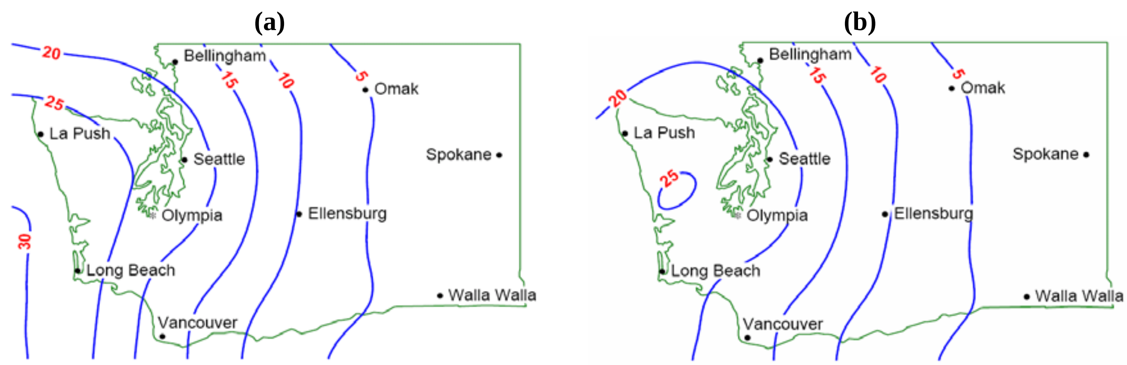

Figure 6. Contours of N_{req} for (a) deterministic analyses based on 475-yr peak acceleration and magnitude, and (b) 400-yr return period of liquefaction.

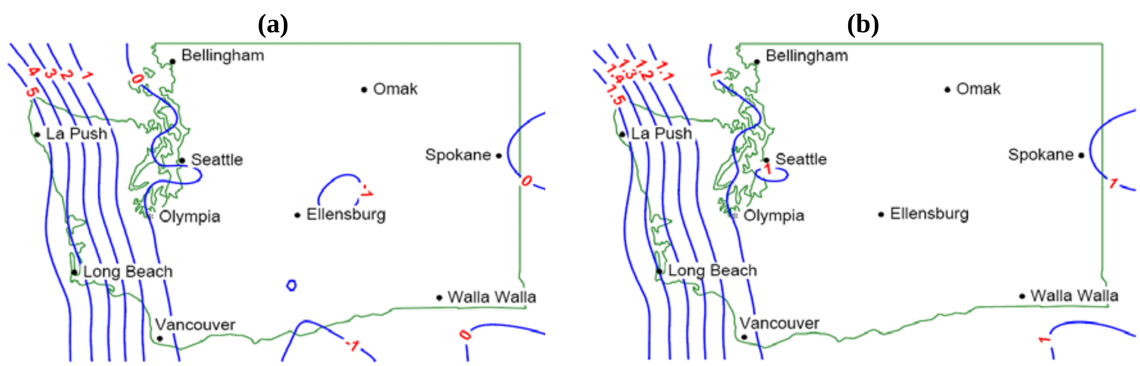

Figure 7. Contours of (a) difference in N_{req} from deterministic and performance-based analyses, and (b) additional factor of safety (relative to Seattle) produced by conventional deterministic analyses.

Tools

The computations required for performance-based liquefaction hazard evaluations are extensive. For a given element of soil, the numerical integration of Equation (2) is performed over 20 magnitude and 100 peak acceleration values for each SPT value; the hazard curves shown in Figure 4 were computed for 500 SPT values thereby requiring a total of one million probabilistic liquefaction analyses. Each of these evaluations requires hazard curve and de-aggregation information.

With the support of the Washington State Department of Transportation, a computational tool for performance-based liquefaction hazard evaluation is being developed. The WSDOT Liquefaction Hazard Evaluation System performs liquefaction hazard analyses using single-scenario, multiple-scenario, and performance-based approaches. The system contains a complete database of PSHA and de-aggregation data for Washington State. The single-scenario approach allows the user to select a single pair of $a_{\max}$ and M_w values or to look them up based on latitude and longitude for any return period. The multiple-scenario approach allows the user to specify a return period for a particular site with liquefaction potential calculated using the corresponding $a_{\max}$ value and all M_w values contributing to $a_{\max}$ (from the de-aggregation data) at that return period. Finally, the performance-based option allows computation of liquefaction hazard curves in terms of both FS_L and N_{req}. This tool allows the considerable benefits of performance-based liquefaction hazard evaluation to be realized with no more effort than that currently expended on conventional liquefaction analyses.

Summary and Conclusions

Conventional criteria for judging the liquefaction potential of a particular site are based on a deterministically-determined factor of safety using a single probabilistically-determined level of ground shaking. A performance-based approach, which combines probabilistically-determined liquefaction potential with probabilistically-determined levels of ground shaking, has been described. By considering all possible levels of ground shaking, the performance-based approach provides a more complete and accurate evaluation of the actual risk of liquefaction.

The performance-based approach can be used to define an alternative criterion for acceptable liquefaction potential – one based on a minimum return period for the occurrence of liquefaction rather than on the minimum return period for the seismic loading. Such a criterion would ensure consistency of actual risk of liquefaction across all seismic environments. Application of the performance-based approach to 10 locations across the United States led to the following conclusions:

1. The actual potential for liquefaction, considering all levels of ground motion, is influenced by the position and slope of the peak acceleration hazard curve and by

the distributions of earthquake magnitude that contribute to peak acceleration hazard at different return periods.

2. Consistent application of conventional liquefaction potential criteria (i.e. based on a minimum factor of safety for a single ground motion level) to sites in different seismic environments can produce highly inconsistent estimates of actual liquefaction hazards.

3. Criteria that would produce more uniform liquefaction hazards at locations in all seismic environments could be developed by specifying a standard return period for liquefaction. Performance-based procedures such as the one described in this paper could be used to evaluate individual sites with respect to such criteria.

4. Using a performance-based criterion of a minimum 475-yr return period for liquefaction, the burden placed on owners of sites in some areas would be reduced relative to the burden imposed by current criteria, in some cases dramatically. In other areas, the burden would be increased.

5. Analyses of a variety of sites in different locations using different liquefaction models followed by careful review by a range of stakeholders will be required to identify a minimum return period that would provide a suitable criterion for liquefaction potential.

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