Towards Improved Performance Objectives for Liquefaction Hazards in Seismic Design Guidelines

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ABSTRACT: Ground failure due to liquefaction in loose sand deposits poses substantial risks to the built environment and has caused substantial damage in past earthquakes to a wide range of infrastructure. Advances in the liquefaction hazard state of practice remains rooted in simplified procedures that ignore considerable uncertainties in liquefaction phenomena and are largely conditional on single-return period ground motions. As a result, engineering practice lacks liquefaction-specific design criteria or performance objectives. Presented herein is a roadmap for using probabilistic liquefaction hazard analysis (PLHA) to address many of these limitations and improve liquefaction design guidelines. PLHA incorporates hazard contributions from the full ground motion hazard space in conjunction with probabilistic liquefaction models, to produce hazard curves for various liquefaction-related demands. In this study, PLHA is utilized to assess the current liquefaction probabilities to which engineers are implicitly designing at 76 study sites throughout the United States using American Society of Civil Engineers (ASCE) 7 guidelines, by computing effective return periods of liquefaction factor of safety FS_L, and liquefaction potential index *LPI*. The results indicate broad variations in these return periods across different parts of the United States, from about 300 years in parts of California, to nearly 3,000 years on the Pacific Northwest coast and the southeastern United States. These results are also used to inform potential strategies for establishing liquefaction-specific design objectives in the future, based on return period averaging methods that weight the importance of a study site according to both the population and relative liquefaction hazard level.

1. INTRODUCTION

Earthquake-induced ground failure, resulting from liquefaction of loose sands and soft silts, poses significant risks to the built environment, causing significant damage in past earthquakes to buildings, bridges, embankments, and critical lifelines. Evaluation of liquefaction-related hazards generally consists of analyzing (1) the liquefaction susceptibility of soils at a site, (2) the likelihood of liquefaction triggering in those soils under earthquake loading given the seismic hazard of the site, and (3) the consequences of liquefaction (i.e., surface manifestation, ground deformation, damage to existing or planned structures).

Current United States (U.S.) seismic design guidelines, e.g., American Society of Civil Engineers (ASCE) 7 and American Association of State Highway and Transportation Officials (AASHTO), generally focus their design criteria on evaluating liquefaction triggering conditional on design ground motion parameters such as peak ground acceleration (*PGA*) and magnitude (M_w) that correspond to a single return period of ground shaking (e.g., 2,475 years for the ASCE 7 guideline; ASCE 2013). When compared to fully probabilistic liquefaction triggering analyses that consider contributions from all *PGA*-M_w scenarios, current code standards result in highly variable design return periods of liquefaction across the United States (e.g., Kramer and Mayfield 2007), which generally goes against the principle and goal of uniformly applied design standards. Such limitations can be addressed through the adoption of design ground motions for liquefaction that are tailored to performance objectives specific to liquefaction hazards, rather than conditional on uniform hazard ground shaking, and would result in more consistent design levels that bring geotechnical practice closer to uniform risk design criteria.

Presented herein are some practical considerations and potential impacts for adopting liquefaction-targeted design ground motions in U.S. building codes. Several different strategies are explored for establishing new design liquefaction return period targets. Candidate criteria include conventional return periods (e.g., 975 and 2,475 years), or return period targets based on current, implied liquefaction design return periods in the United States. Different design objectives based on different liquefaction limit states (e.g., triggering at the sublayer layer level and surface level, or surface manifestation) are also explored.

2. CONVENTIONAL & PROBABILISTIC LIQUEFACTION HAZARD ANALYSIS

In most practical applications, liquefaction hazard analysis involves an initial assessment of the liquefaction factor of safety FS_L. The FS_L approach is essentially a demand-capacity relationship for liquefaction, where the soil's cyclic resistance against liquefaction is compared against the expected seismic demands, represented by *PGA* and M_w (to account for the effects of duration). Within most U.S. building codes, a single combination of *PGA* and M_w is used based on uniform ground shaking hazard (or are deterministic considerations); in ASCE 7, the *PGA* value corresponds to a 2,475-year return period (or the Maximum Considered Earthquake), with the M_w specified to be the "controlling magnitude" under those shaking conditions and is usually obtained from hazard disaggregation.

The use of the 2,475-year *PGA* return period as a liquefaction design basis is largely an artifact of earlier iterations of ASCE 7 seismic performance objectives for structural design; whereas the uniform hazard design basis was eventually superseded by a uniform risk collapse objective (via the introduction of risk-targeted design motions in ASCE 7-10 [ASCE 2013]), similar improvements have yet to be made for analysis and mitigation of geotechnical hazards. As a result, current conventional liquefaction hazard analyses have substantial limitations; the use of a single return period of *PGA*, rather than the full hazard curve, as well as deterministic liquefaction triggering models that ignore considerable uncertainties inherent to liquefaction hazard estimation, have been shown to result in highly inconsistent design levels across different site conditions and seismotectonic environments (Kramer and Mayfield 2007; Makdisi 2021).

Many of these limitations can be addressed using a framework known as probabilistic liquefaction hazard analysis, or PLHA (Kramer and Mayfield 2007). PLHA is an application of the Pacific Earthquake Engineering Research Center (PEER) performance-based earthquake engineering framework (PBEE), and an extension of probabilistic seismic hazard analysis (PSHA) that is based on computing annualized rates of occurrence of various modes of liquefaction demand. The most common application of PLHA is used for computing the annualized non-exceedance rate of FS_L (Λ_{FS_L}) as follows:

$$\Lambda_{FS_L} = \sum_{j=1}^{N_m} \sum_{i=1}^{N_{pga}} \mathrm{P}\left[FS_L < fs_L \middle| pga_i, M_{w,j}\right] \cdot \cdot \mathrm{P}\left[M_w = m_{w,j} \middle| pga_i\right] \Delta\lambda_{pga_i} \quad ()$$

where $\mathrm{P}[FS_L < fs_L | pga_i, m_{w,j}]$ is the probability of non-exceedance of a certain value of fs_L given a particular *PGA* and M_w, $\Delta\lambda_{pga}$ is the incremental annualized exceedance rate of *PGA* (i.e., the numerical derivative of the hazard curve at a given *PGA* value); $\mathrm{P}[M_w = m_{w,j} | pga_i]$ is the conditional probability of a particular event magnitude given the exceedance of pga_i; and N_m and N_{pga} are the number of M_w and *PGA* values, respectively, over which the expression is integrated numerically.

The PLHA calculation is illustrated for a hypothetical soil profile, represented by cone penetration test (CPT) resistance data in Figure 1. The profile consists of a 2-m-thick crust of non-liquefiable material above the groundwater table, underlain by a 4-m-thick layer of liquefaction-susceptible soil, with a linearly increasing CPT resistance (q_{c1Ncs}) between 60 and 200 (corresponding to an estimated relative density range of approximately 45% to 85%).

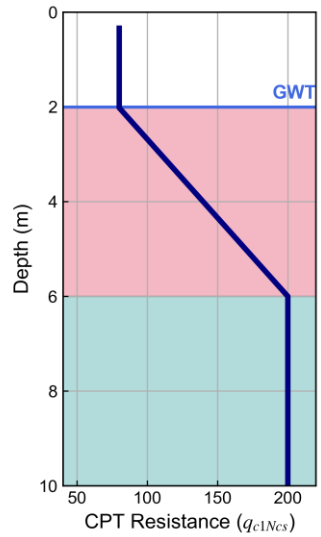

Figure 1: Hypothetical CPT profile, with susceptible layer shaded in red, with ground water table (GWT) located at 2-m depth.

The resulting FS_L hazard curves are shown, based on ground motion data from a site in San Francisco, California, in Figure 2 for a selected subset of soil elements at varying depths and relative densities. Several important pieces of information can be obtained from these curves. For a given soil element, a return period of liquefaction (i.e., the reciprocal of the annual rate of FS_L dropping below 1.0) can be computed; in this example the return period is 258 years at a depth of 4.0 m, corresponding to approximately an 18% likelihood of liquefaction in 50 years. Alternatively, one can specify a target factor of safety return period $T_{R,FS}$, and compute the corresponding FS_L value – in the case of the 5.0-m-deep element, the 1000-year FS_L value is roughly 1.23, corresponding to relatively low liquefaction potential. The second calculation is particularly useful, as it can be repeated at all soil depths to produce FS_L profiles with uniform return periods.

As with almost any application of PBEE, the advantages of PLHA lie in its modularity and ability to extend the probabilistic analysis beyond a particular demand parameter. In liquefaction hazard applications, FS_L profiles are often used as inputs to subsequent procedures for estimating the consequences of liquefaction. Liquefaction manifestation indices (MI), such as the liquefaction potential index (LPI, Iwasaki et al. 1978), provide a useful means for assessing the cumulative influence of a liquefiable profile on the effects of liquefaction at the ground surface, and can guide engineers on how and when to refine their analysis methods for estimating liquefaction consequences. Such indices are based on integrating the factor of safety profile with depth generally as follows:

$$MI = \int_z F(FS_L) \cdot w(z) dz \tag{1}$$

where $F(FS_L)$ is a function that expresses the influence of factor of safety on the severity of surface manifestation, and $w(z)$ weights the influence of depth, with different indices using different methods to develop each weighting function.

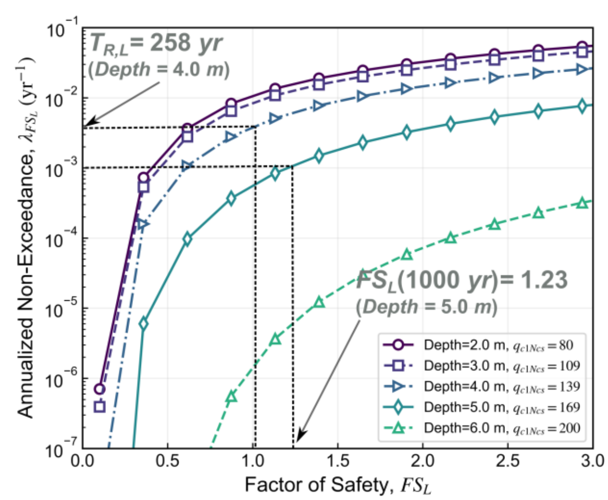

Figure 2: FS_L hazard curves for a hypothetical soil profile and a site in San Francisco, California.

Within the PLHA framework, hazard curves for manifestation indices can be easily computed from the FS_L hazard curves by calculating the *MI* value per Equation (1) multiple times for a set of FS_L profiles encompassing a broad range of $T_{R,FS}$ values. For example, the 2,475-year *LPI* value (i.e., an annualized rate of exceedance of 4.04×10^{-4} yr^{-1}) would be calculated from the FS_L profile corresponding to $T_{R,FS}$ =2,475 years. The resulting hazard curves for *LPI* for the example CPT profile are shown in Figure 3.

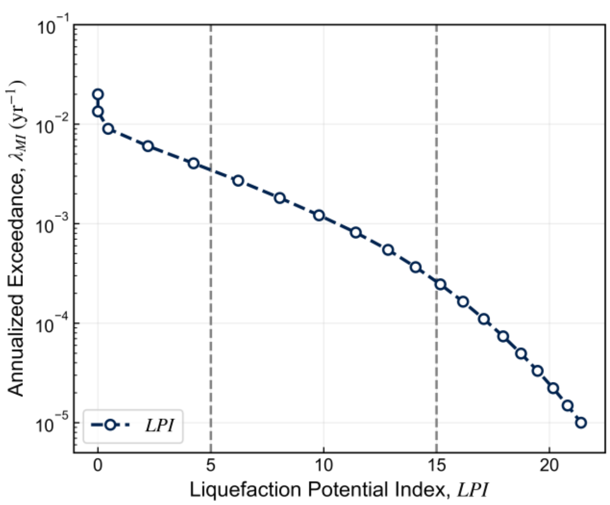

Figure 3: Example hazard curves for liquefaction potential index for a hypothetical soil profile located in San Francisco, California.

In addition to the benefits discussed previously, the PLHA framework and the resulting FS_L or *LPI* hazard curves also have the potential to improve liquefaction design guidelines themselves by serving as the basis for developing design criteria specifically tailored to liquefaction hazards, resulting in performance objectives that are anchored to specified return period of FS_L or *LPI*. In subsequent sections, several strategies for establishing liquefaction-specific performance objectives are explored using PLHA calculations.

3. CONSISTENCY-TARGETED DESIGN FOR LIQUEFACTION HAZARDS

Ideally, performance objectives for seismic design would be based on relatively rigorous cost-benefit analysis, involving some degree of characterization of the frequencies of earthquake hazard occurrence, engineering response, damage, and annualized costs in the absence of mitigation, as well as the reduction in losses weighed against the costs of various levels of mitigation. Variations in such an analysis across different geographic regions, site conditions, and engineering systems ideally would also be assessed before establishing an acceptable level of risk and, subsequently, a uniform design objective.

The reality, however, is that existing guidelines have been in place for decades, based on ground shaking hazard levels that have produced relatively consistent design bases over the course of building code cycle updates. As a result, any changes to design guidelines generally would be evaluated against their impacts to current design, feasibility, and construction costs. Such design impacts could be evaluated, for example, if the 2,475-year *PGA* design basis for liquefaction were to be replaced by a 2,475-year FS_L design basis (i.e., moving from uniform ground shaking hazard to uniform liquefaction triggering hazard). The map in Figure 4 illustrates the average difference in resulting FS_L values within the liquefaction-susceptible layer in the example profile in Figure 1, using conventional and PLHA-based analyses. The conventional FS_L values were computed using design *PGA* parameters from ASCE 7 and benchmarked against the PLHA-based, 2475-year FS_L values, at 76 sites throughout the United States. For the PLHA calculations, the underlying *PGA* hazard data were obtained from the 2018 U.S. Geological Survey (USGS) National Seismic Hazard Model (NSHM) web-services (Petersen et al. 2020) at each geographic location, for a site shear wave velocity V_{s30} of 183 m/s. The ASCE 7 design *PGA* data were obtained from the USGS web-services for a Site Class D/E condition (which similarly

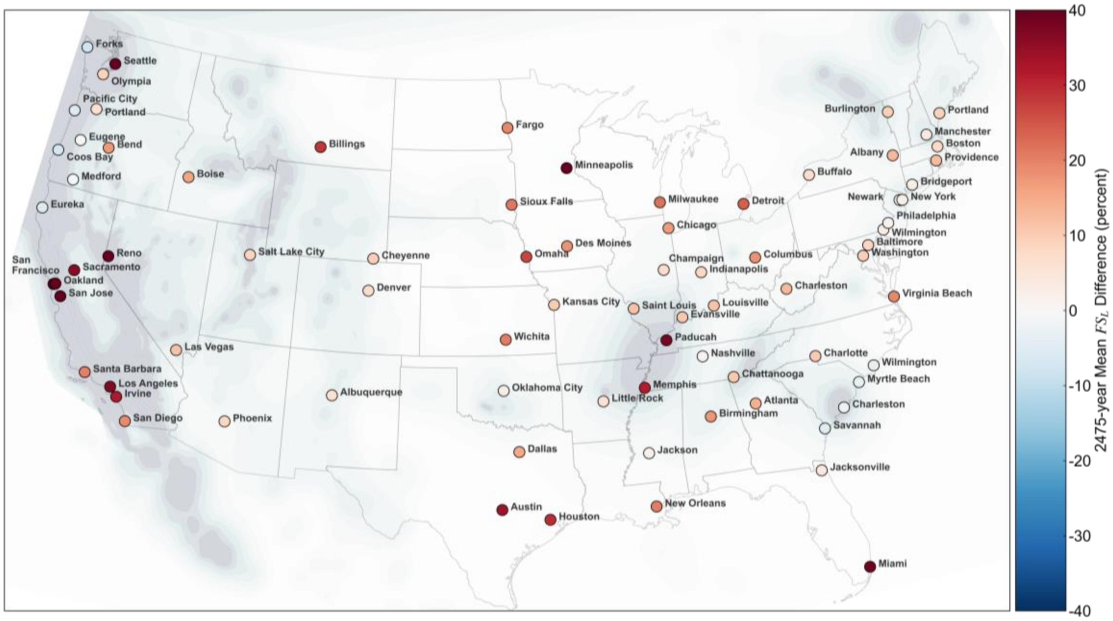

Figure 4: Average percent difference in computed FS_L values in an example soil profile using current ASCE 7 design procedures, compared to 2,475-year FS_L values obtained from PLHA-based liquefaction hazard curves. Data overlaid on greyscale contour map showing relative seismic hazard from the USGS 2018 NSHM.

corresponds to a V_{s30} of 183 m/s). Forty-eight of the test locations were selected to be the most populous city in each conterminous U.S. state, with an additional 28 sites selected due to their proximity to areas of high or unique seismicity, as well as their status as population centers. The results in Figure 4 reveal striking differences between current and probabilistic liquefaction hazard analyses that vary substantially across the country. Areas dominated by highly active crustal seismicity, such as the San Francisco Bay area, southern California, and the Seattle metropolitan area, have the largest positive differences in FS_L, corresponding to an under-prediction of the true liquefaction hazard of 40% to more than 100%. It should be noted that many of these sites are subjected to deterministic capping considerations in current ASCE 7 standards; adoption of PLHA-based liquefaction design criteria may be subjected to similar considerations (or, alternatively, tolerance of higher acceptable liquefaction hazard in such areas [Stewart et al. 2020]). Areas along the Pacific Northwest coast, where the hazard is dominated more by subduction earthquakes, have much lower FS_L errors on the order of 0 to –10% (corresponding to slight over-predictions of the liquefaction hazard). In the central and eastern United States (CEUS), differences around the Charleston Fault Zone (e.g., Charleston, Myrtle Beach) are generally within +/-5%, while cities in the New Madrid area (e.g., Memphis, Paducah, St. Louis) would see FS_L differences of roughly +10 to +30%. The remaining parts of the United States not discussed in this section are generally located in areas of low seismicity and have average FS_L values well above 1.5 for this particular profile, and thus are generally not considered impactful from a design standpoint.

The results shown in Figure 4 generally imply that a direct adoption of a uniform FS_L return period design objective (especially one that is quite arbitrary – a 2,475-year return period was simply selected to follow conventional ground motion return periods and has little basis in actual liquefaction hazards) would produce substantial differences in liquefaction triggering analysis results across the United States. Moreover, these impacts would be highly non-uniform with respect to geographic location and tectonic environment.

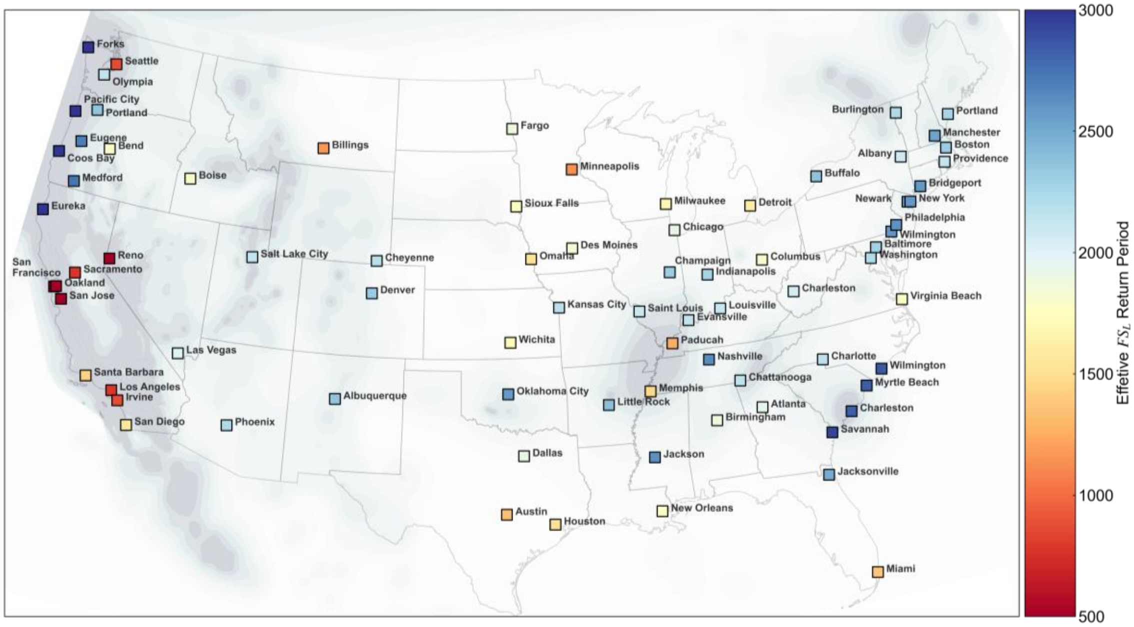

Figure 5: Average effective design FS_L *return period for an example soil profile using current ASCE 7 design procedures and ground motions, computed from PLHA-based liquefaction hazard curves. Data overlaid on greyscale contour map showing relative seismic hazard from the USGS 2018 NSHM.*

An alternative approach for establishing new liquefaction design objectives that may help minimize short- to intermediate-term design impacts is to select a target return period that is reflective of the current *implied* liquefaction design levels, answering the question of: what FS_L return periods do we actually obtain using uniform hazard or deterministically capped design ground motions?

3.1. Assessment of Current Liquefaction Triggering Design Levels (FS_L)

The implied, or "effective" FS_L design return periods can be obtained through the same PLHA calculations used to produce the data in Figure 5, simply by computing the conventional FS_L value using ASCE 7 design ground motions and interpolating the corresponding PLHA-based FS_L hazard curve to estimate the effective return period for the same soil element at a given site. These return periods can be visualized in Figure 5 in a similar manner to Figure 4. The same spatial inconsistencies are observed, with average FS_L return periods as high as nearly 3,000 years on the Pacific Northwest coast, and as low as 1,000 years in non-capped areas in California such as Sacramento, Irvine, and Santa Barbara. Effective return periods are as low as 200–400 years in deterministically-capped sites in California.

One approach for establishing new FS_L design targets could be based on the average of the effective return periods (e.g., Figure 5), the motivation for which would be to broadly minimize impacts on current design levels. In this example case, the average in the Western United States (WUS) is around 1,700 years (corresponding to a roughly 3% in 50-year non-exceedance rate of FS_L) – this average return period, however, masks huge variations in the WUS, ranging from about 600 (mainly in deterministically capped areas) to 2,700 years. In the CEUS, the average return period is around 2,020 years (or about 2.4% in 50-year non-exceedance), ranging from 1,700 to 2,400 years.

Although these arithmetic averages indicate that a design basis of about 2.5 to 3.0% FS_L non-exceedance rate in 50 years may be appropriate as a starting point for future liquefaction design guidelines, the effective return period estimate could be further refined by giving more weight to study areas with larger populations and higher liquefaction hazard. Weighting factors for the average effective return periods within the soil profile at each site were developed according to population, and the reciprocal of the average FS_L to represent the liquefaction hazard.

The resulting effective FS_L return periods, using arithmetic, population-weighted, and combined population and hazard-weighted averages are summarized for the WUS and CEUS sites in Table 1. The combined weighting scheme produced the largest weights for cities such as Los Angeles, San Jose, Seattle, and Portland in the WUS; and Memphis, St. Louis, Charleston, and Evansville in the CEUS – this result generally tracks with the areas that are studied more heavily in the development of building codes. Although the refined averages generally produced similar design level estimates in the CEUS, we see that the FS_L design levels in the WUS are highly sensitive to the selected weighting scheme, and these levels were reduced substantially when considering the effects of both population and hazard level.

Table 1. Average effective return periods of FS_L in years for an example profile in WUS and CEUS.

Region	**Average Effective FS_L Return Period (% 50-year non-exceedance rate)**		
	Arithmetic	**Population Weighted**	**Population & Hazard Weighted**
Western United States	1,724 (2.9% in 50 yr)	1,413 (3.5% in 50 yr)	1,187 (4.1% in 50 yr)
Central & Eastern United States	2,021 (2.4% in 50 yr)	1,929 (2.6% in 50 yr)	2,012 (2.5% in 50 yr)

3.2 Assessment of Current Liquefaction Manifestation Design Levels (LPI)

Rather than relying on average FS_L values within a susceptible stratum, this analysis can be repeated at the profile level by considering design levels of a manifestation index such as *LPI*. Table 2 summarize the average *LPI* return periods at the 75 test sites for the example profile, with similar schemes for weighting by population and liquefaction hazard level. Note that in this instance the hazard weight is based on the computed *LPI* rather than the reciprocal of FS_L; in the CEUS, only 7 out of the 55 sites had computed 2,475-year *LPI* values greater than zero, resulting in average *LPI* return periods that are strongly influenced by just a handful of sites. Regardless, we see that the average *LPI* design levels for both WUS and CEUS are quite similar to those summarized in Table 1 for FS_L. Again, it is important to emphasize that the average *LPI* return periods in WUS mask extremely large variations, particularly between the San Franisco Bay area and Pacific Northwest coast, due to an outsized influence of deterministically-capped ground motions in the San Francisco Bay area that result in *LPI* return periods of 200-500 years for the example profile.

Table 2 Average effective return periods of LPI in years for an example profile in WUS and CEUS.

Region	**Average Effective *LPI* Return Period (% 50-year exceedance rate)**		
	Arithmetic	**Population Weighted**	**Population & Hazard Weighted**
Western United States	1,797 (2.7% in 50 yr)	1,103 (4.4% in 50 yr)	1,173 (4.1% in 50 yr)
Central & Eastern United States	2,135 (2.3% in 50 yr)	2,106 (2.3% in 50 yr)	1,934 (2.6% in 50 yr)

4. SUMMARY AND FUTURE WORK

The state of practice for the analysis of and design against liquefaction hazards has lagged considerably behind other areas of earthquake engineering in the incorporation of performance-based earthquake engineering principles. The uniform hazard *PGA* remains the design basis, and there is little incentive or guidance for engineers to incorporate other source of uncertainty into liquefaction problems, resulting in highly simplistic design objectives that are not tied to liquefaction-related performance.

Probabilistic liquefaction hazard analysis (PLHA) addresses many of these limitations by incorporating the full *PGA*-M_w hazard space in conjunction with probabilistic models that capture many uncertainties inherent to liquefaction hazard estimation, resulting in hazard curves for various liquefaction-related demands, such as FS_L and *LPI*. Furthermore, PLHA provides a roadmap to improving building codes towards liquefaction-oriented performance objectives by anchoring design criteria to a uniform, annualized likelihood of some liquefaction demand being exceeded.

In this study, PLHA was utilized to show that current, implied liquefaction design levels stemming from ASCE 7 guidelines can be highly inconsistent throughout the United States, with effective return periods of FS_L and *LPI* ranging from roughly 200 years (over 20% exceedance in 50 years) to nearly 3,000 years (1.5% in 50 years) in the WUS, and about 1,900 to 2,000 years in the CEUS (about 2.3 to 2.5% in 50-year exceedance). Average liquefaction return periods for the WUS and CEUS were computed using various methods for weighting the effects of population and relative hazard level. This framework has the potential to inform the development of PLHA-based design levels for consideration in future building code iterations (targeting, for example, a specified and consistent return period of FS_L or *LPI*) in a way that minimizes design impacts from one cycle update to the next.

In many respects, the analysis and results presented here are a proof of concept, rather than an endpoint, and there are several planned areas to extend this work. One clear need is to expand this analysis beyond the example profile in Figure 1 and consider a more realistic suite of profiles representing a broader range of soil conditions. It may also be useful to refine the evaluation of the liquefaction design levels to consider the ranges of effective return periods rather than the averages – in all likelihood, suitable design criteria that relatively minimize design impacts may lie at the upper and lower ends of the current WUS and CEUS design levels, respectively. More information may also be gleaned from refining the study regions (given the especially large within-region differences in WUS design levels).

Additionally, the use of deterministic capping for design ground motions in ASCE 7 and its effects on the resulting design levels has been addressed only in a limited sense in this study. Any future updates to liquefaction design guidelines would ideally consider whether deterministically capped sites should be excluded from the effective return period characterization, or whether alternatives to deterministic capping can and should be explored for liquefaction design purposes.

Finally, and most importantly, the ultimate goal of this framework is to advance liquefaction design guidelines to be based on performance objectives of more relevant engineering demands, such as surface manifestation, lateral spreading, or building settlement. An advantage of PLHA is its modularity and extensibility – as new fragilities for these demands are developed, this same framework can be utilized for developing and implementing liquefaction demand-specific performance objectives in the future.

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