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Probabilistic Liquefaction Analysis

U.S. Army Corps of Engineers

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This document provides technical bases for formulating probabilistic approaches to liquefaction evaluation. The three basic elements of probabilistic liquefaction analysis are: (1) uncertainty in the earthquake load, (2) uncertainty in the available resistance, and (3) uncertainty in the method of analysis. The probabilistic approach is built from the steps in a deterministic liquefaction analysis; however, the input parameters, such as penetration resistance, site stratigraphy, acceleration, and magnitude, are treated as random variables and the accuracy of the method of analysis is factored in as a part of a capacity-demand model. Uncertainty in the earthquake load is generally treated with a probabilistic seismic hazard analysis, which introduces time as a parameter. The site stratigraphy and engineering properties are generally treated as one-, two-, or three-dimensional random fields. Uncertainty in the method of analysis is generally estimated with logit regression analysis of the field performance data base. It is assumed that the reader has a working knowledge of probability theory, stochastic processes, liquefaction evaluation, and probabilistic seismic hazard analysis calculations.

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ABSTRACT

This document provides a technical basis for formulating probabilistic approaches to liquefaction evaluation. The three basic elements of probabilistic liquefaction analysis are described: (1) uncertainty in the earthquake load, (2) uncertainty in the available resistance, and (3) uncertainty in the method of analysis. The probabilistic approach is built from the steps in a deterministic liquefaction analysis; however, the input parameters, such as penetration resistance, site stratigraphy, acceleration, and magnitude, are treated as random variables and the accuracy of the method of analysis is factored in as part of a capacity-demand model. Uncertainty in the earthquake load is generally treated with a probabilistic seismic hazard analysis which introduces time as a parameter. The site stratigraphy and engineering properties are generally treated as one-, two-, or three-dimensional random fields. Uncertainty in the method of analysis is generally estimated with logit regression analysis of the field performance data base. It is assumed that the reader has a working knowledge of probability theory, stochastic processes, liquefaction evaluation, and probabilistic seismic hazard analysis calculations.

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PREFACE

The study described in this report was performed by the U.S. Army Engineer Waterways Experiment Station (WES) for the U.S. Nuclear Regulatory Commission (NRC) under Inter-Agency Agreement RES-95-008 during the period June, 1995, to January, 1999. The study was directed by Mr. Robert Kornasiewicz and Dr. Ernst Zurflueh, Office of Nuclear Regulatory Research, NRC.

The report was prepared by Dr. Mary Ellen Hynes, Chief, Earthquake Engineering and Geophysics Branch, Earthquake Engineering and Geosciences Division (EEGD), Geotechnical Laboratory (GL), WES. General supervision was provided by Dr. Lillian D. Wakeley, Acting Chief, EEGD, and Dr. William F. Marcuson III, Director, GL.

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At the time of preparation of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Robin R. Cababa, EN.

1 INTRODUCTION

The professional literature has numerous papers on probabilistic liquefaction analysis as shown in Appendix A. The three basic elements of probabilistic liquefaction analysis are: (1) uncertainty in the earthquake load, (2) uncertainty in the available resistance, and (3) uncertainty in the method of analysis. The probabilistic approach is built from the steps in a deterministic liquefaction analysis (see for example Koester, Sharp, and Hynes 1999); however, the input parameters, such as penetration resistance, site stratigraphy, acceleration, and magnitude, are treated as random variables and the accuracy of the method of analysis is factored in as part of a capacity-demand model (see for example Benjamin and Cornell 1970). Uncertainty in the earthquake load is generally treated with a probabilistic seismic hazard analysis (Cornell 1968, Frankel et al. 1996) which introduces time as a parameter. The site stratigraphy and engineering properties are generally treated as one-, two-, or three-dimensional random fields (see for example VanMarcke 1983, Journel 1989, Wackernagel 1995). Uncertainty in the method of analysis is generally estimated with logit regression analysis of the field performance data base (see for example Liao, Veneziano, and Whitman 1988; Liao 1996; and Liao and Lum 1998). A logit regression was developed in this study for a relatively new empirical liquefaction chart using Arias Intensity (Kayen 1993; Kayen and Mitchell 1997).

This report describes these three basic elements of probabilistic liquefaction analysis and the formulation of the capacity-demand model. More detail is provided on uncertainty in the method of analysis; probabilistic seismic hazard analysis and random fields are well covered elsewhere in the literature. This report does not discuss the consequences of liquefaction, such as increased pore water pressures, settlement, deformations, slope instability, and reduced strength, stiffness, and bearing capacity; however, probabilistic treatment of these consequences uses the same techniques as the probabilistic liquefaction analysis model developed in this report, namely a capacity/demand type of probabilistic model based on a more complex model of liquefaction and consequent soil behavior and deformations. It is assumed that the reader has a working knowledge of probability theory, stochastic processes, liquefaction evaluation, and probabilistic seismic hazard analysis calculations. This report provides a structured summary of the literature and current practice in this topic area, and presents some additional contributions, research in progress, and recommendations. There are many researchers who have contributed to this topic area; a limited selection of contributions are discussed in the text. In an effort to provide a broader list of contributions, a bibliography on probabilistic liquefaction analysis is provided in Appendix A.

2 UNCERTAINTY IN EARTHQUAKE LOAD

2.1 Apparatus

Probabilistic seismic hazard analysis (PSHA) is widely used in engineering practice and has evolved with increasing sophistication over the years since it was first formulated by Cornell (1968) to its current application in USGS earthquake hazard maps (Frankel et al. 1996). A PSHA is a model for estimating the frequency of occurrence of earthquakes of various sizes for each source zone that affects a site and combining their contributions to the probability of exceeding a particular ground motion parameter at a site within a specified time period. Although PSHA has advanced in its sophistication and use over the years, it is not a perfect descriptor of the natural process of earthquake occurrence. There are shortcomings in its formulation and the data base for input parameters. These shortcomings are well documented by Krinitzsky (1993a, 1993b, 1995a, and 1995b). Nevertheless, PSHA is the best tool available at this time for attempting to quantify uncertainty in earthquake ground motions in future time and space.

The basic steps in a PSHA are: (1) identify seismic source zones that could affect the site; (2) for each source zone, determine maximum or characteristic earthquake size and recurrence relationships which relate frequency of occurrence to earthquake size; (3) for each source zone and ground motion parameter of interest, select an attenuation relationship which accounts for changes in the parameter as a function of the distance from the source to the site; and (4) combine the probability of exceedance contributions from each source zone for the time period of interest.

Attenuation functions exist for many types of ground motion parameters, source types, and site conditions. Advances are being made in direct calculation of site motions from energy release at the source; however, development, validation, and quantification of uncertainty in these advanced seismology models were still in progress at the time of this study.

The ground motion parameter needed from the PSHA depends on the type of liquefaction analysis used. For example, Simos, Costantino, and Reich (1995) use power spectral density for stochastic modeling of the earthquake and probability density functions for the engineering properties of their two-phase (soil-water) analytical technique. Hwang and Lee (1991) use the Iwasaki et al. (1982) liquefaction potential index and moment magnitude. Todorovska (1998) uses seismic wave energy determined from the Fourier spectrum of strong motion velocity with an empirical liquefaction model. Cao and Law (1991) also use an energy approach with an empirical liquefaction model. Kayen's (1993) approach uses Arias Intensity of the acceleration in the liquefiable layer.

The most widely used and accepted type of liquefaction analysis is the Seed-Idriss approach (see for example Youd and Idriss 1997). The Seed-Idriss simplified method has evolved to a practical form since 1971 (Seed and Idriss 1971; Seed, Idriss, and Arango 1983; Seed et al. 1985; Idriss 1999). The simplified method uses the earthquake magnitude and peak site acceleration to estimate cyclic shear stresses in the soil deposit. Changes in site response with depth and magnitude are quantified by a factor r_D which has been determined primarily from numerical wave propagation calculations (Idriss 1999).

2.2 Probabilistic Site Response

Incoming earthquake ground motions are modified as they propagate through the soil deposits at a site. The resulting cyclic shear stresses and acceleration response in the soil deposit depend on the stratigraphy (thickness, depth, and areal extent of various soil units, depth to rock) and engineering properties of the soil units (such as soil type, density, stiffness, modulus changes with cyclic straining). Rigorous probabilistic modeling of all the parameters that affect site response to incoming ground motions has been attempted by various researchers. Popescu, Deodatis, and Prevost (1996) use multi-dimensional, multi-variate, non-Gaussian homogeneous stochastic fields to investigate the effect of varying soil stratigraphy and properties on site response. Arango et al. (1996) present an alternative approach that uses simplified one-dimensional site response calculations together with a simplified deaggregated seismic hazard to yield an estimate of probability of liquefaction. Other researchers have used fuzzy logic to incorporate uncertainty in site response (Rahman and El Zahaby 1997; Zhou, Su, and Fan 1992).

Three-dimensional modeling of the soil deposits at a site and the resulting physical distribution of response can provide very useful information for sites with complex geomorphology, topographic relief, and varying bedrock surface. Also, many parameters are physically and statistically correlated, which further complicates a rigorous analysis (for example, soil density, stiffness, and confining stress).

Consequently, a probabilistic site response analysis with many variable parameters and statistical regressions for estimation is not necessarily better than a more simplified approach.

2.3 Artificial Earthquake Records

Probabilistic techniques have been used to generate artificial earthquake records. Artificial earthquake records can be divided into two groups: those that derive from signal processing techniques and those that derive from physical modeling of wave propagation.

Frequency domain and time domain signal processing techniques have been widely used to generate artificial earthquake records that are consistent with prescribed power density spectra or response spectra; these records are particularly applicable for linear elastic analyses typically used in structural dynamics. These records are appealing to an analyst because they are easy to manipulate numerically. However, such artificial records have too much energy throughout the frequency spectrum compared to natural records. Such records should not be used for an analysis that includes nonlinear effects. Consequently, such records are not recommended for evaluation of liquefaction and consequent stability and nonlinear deformation analyses. If such records are used for a probabilistic liquefaction analysis, a reality check with actual records would help to evaluate confidence in the results.

Advances in seismology have resulted in three-dimensional wave propagation codes to estimate source to site transmission of earthquake motions. These codes generally are limited to low frequency output since the computational requirements to capture source to site characteristics are so demanding. Costantino (1999) cites an example in which the computations had to be limited to a two-dimensional configuration in order to even approach 2 Hz capability in the output, while mesh size characteristics severely limited site descriptions. Seed (1998) describes developments to statistically "roughen" the rupture/release of energy to capture higher frequency motions. Although the progress in this work is very promising, development, validation, and quantification of uncertainty in these advanced seismology models were still in progress at the time of this study, as noted above.

3 UNCERTAINTY IN LIQUEFACTION RESISTANCE

3.1 Calculation of Liquefaction Resistance

Almost any soil can be liquefiable. Typically, liquefaction is observed in the form of surface sand boils from loose, saturated, or nearly saturated, Pleistocene or younger, water-laid deposits at depths less than 50 ft. These observations do not preclude the occurrence of liquefaction at greater depths. Gravels, sands, silts, and high water content, low plasticity clays, and mixtures of these soil types can liquefy if shaken hard enough and long enough. Low permeability layers, if present, can trap developing high pore water pressures; consequently, even thin layers of potentially liquefiable materials may be damaging.

Probabilistic analysis of liquefaction resistance follows from the form of deterministic analysis used. Deterministic liquefaction evaluation methods are either analytical (constitutive models based on first principles of soil behavior), empirical (field observations and laboratory testing), or a combination of the two. The 1997 evolution of the Seed-Idriss procedure is the standard recommended for practice at the time of this writing (Youd and Idriss 1997), and is the focus of this section. Additional liquefaction evaluation methods are mentioned at the end of this section.

3. Uncertainty in Liquefaction Resistance

The liquefaction resistance of a soil in the empirical Seed-Idriss procedure is determined from $N_{1,60}$, defined as the Standard Penetration Test (SPT) blowcount corrected to a confining stress of 1 atm and an energy efficiency of 60 percent. The liquefaction resistance in terms of a cyclic shear stress ratio is:

$$\tau_{cy}/\sigma_v' = f(N_{measured} \times (E/60) \times C_N + K_F) \times K_M \times K_\sigma \times K_\alpha \times \ldots \tag{3.1}$$

where

$N_{measured}$ = SPT blowcount measured in the field
E = energy efficiency of the equipment used in the field
C_N = overburden correction to convert $N_{measured}$ to a confining stress of 1 atm
K_F = fines content correction
K_σ = overburden correction factor for cyclic shear strength
K_α = initial shear stress correction factor
K_M = magnitude correction factor
$f(\,)$ = is defined by the Seed-Idriss chart
τ_{cy} / σ_v' = cyclic shear strength / vertical effective confining stress

Note that ($N_{measured} \times (E / 60) \times C_N$) is equal to $N_{1,60}$, and ($N_{measured} \times (E / 60) \times C_N + K_F$) is equal to an equivalent clean sand $N_{1,60}$. Each of the factors noted in Equation 1 can be treated as a random variable; however $N_{measured}$, C_N , K_F , K_σ and K_α are correlated to each other. The factors K_σ and K_α, estimated from laboratory tests, are applied to the chart strength values to extend the chart beyond the field performance data base. Additional factors have been developed for age, overconsolidation, and other aspects that influence liquefaction resistance, but they are not usually applied in practice.

The magnitude correction factor, K_M, has been studied by a number of researchers since its introduction by Seed and Idriss in 1982 (Ambraseys 1988, Idriss 1999, Arango 1996, Youd and Noble 1997, Youd and Idriss 1997, and Liao and Lum 1998). For a magnitude 6 earthquake, K_M ranges from 1.32 (Seed and Idriss 1982) to 2.92 (Youd and Noble 1997). In their statistical analyses, Liao and Lum concluded that the Seed and Idriss (1982) relationship for K_M best fits the Liao and Whitman (1986) database. Youd and Idriss (1997) recommended using K_M values similar to Idriss (1997); for a magnitude 6 earthquake, K_M = 1.76, compared to 1.32 from Seed and Idriss (1982). Values for K_M are still under study. Liao and Lum (1998) provide an example of the impact of K_M on probabilistic analysis of liquefaction and risk analysis for Keenleyside Dam.

Silva and Costantino (1999) have examined liquefaction data from the Northridge and Kobe earthquakes and recommend an additional correction factor for distance. They note that empirical liquefaction correlations are based mainly on distant events and cannot easily treat close-in "fling effects," or exaggerated vertical ground motions as measured in the Northridge and Kobe earthquakes, for which the fault ruptures occurred very near the recording instruments.

Cyclic shear strength can also be estimated with laboratory tests that can be used to develop site specific relationships for K_F , K_σ and K_α , and better define C_N for the materials and stress range of interest at the site (see for example Koester, Sharp, and Hynes 1999). Like blowcounts, laboratory tests are not perfect measurements of in situ cyclic shear strength. The test results are altered by unavoidable disturbance that occurs during sampling, changes in density and stress state in the sample as it is extracted from the field and prepared for testing, and loading and boundary conditions in the testing apparatus that do not perfectly simulate field conditions and earthquake loads.

Reconstituted specimens have been widely used by researchers to investigate liquefaction potential. These specimens are constructed either by pluviation or moist-tamped or moist-vibrated layers. Laboratory test results are available for gravels, sands, silts, clays, and mixtures of these soil types. Hynes and Olsen (1998) have shown that reconstituted pluviated specimens underestimate in situ cyclic shear strength by a factor of two to three at a confining stress of one atm; this error becomes smaller as confining stresses increase. Hynes and Olsen (1998) have shown that specimens constructed by moist-tamped or moist-vibrated layers may overestimate in situ cyclic strength at low relative densities and underestimate cyclic strength at high relative densities. The problem is that the reconstruction process does not duplicate or preserve stress history, fabric, or aging effects that are present in situ. Pluviated specimens may be representative of recently dredged or recently liquefied materials, but with time, the deposit will densify, develop stress history, etc., and its cyclic strength will usually increase. Consequently, laboratory testing of reconstructed specimens is not a panacea to the deterministic or probabilistic modeling problem.

At this time, there is no clearly established "best" procedure for modeling these uncertainties in cyclic shear strength. In spite of the many uncertainties and correction factors involved, the Seed-Idriss-Arango 1983 chart for liquefaction based on SPT measurements has performed well in distinguishing liquefiable from nonliquefiable materials during the 16 years of field observations since 1983. The implication is that a rigorous probabilistic treatment of all contributing factors to uncertainty in calculating liquefaction resistance may overestimate actual uncertainty. An alternative is to incorporate uncertainty estimates from expert elicitation; this approach has its problems (Kahneman, Slovic, and Tversky 1982; Hynes and VanMarcke 1975), but can introduce engineering and geological expertise that is missing in a purely mathematical analysis.

3.2 Spatial Modeling of the Soil Deposit

For simplicity, let us assume that if $N_{measured}$ is known, then an equivalent clean sand $N_{1,60}$ can be determined with certainty. The problem now becomes modeling the variability in $N_{measured}$ throughout the volume of the soil deposit, since only a few locations will have actual measurements. Geostatistics (Journel 1989, Wackernagel 1995) and random fields (VanMarcke 1983; Popescu, Deodatis, and Prevost 1996) provide numerical techniques for stochastic modeling of this three- dimensional stratigraphy problem, which includes thickness, orientation, and continuity of layers, depth to rock, surface topography, and varying blowcounts. Variograms, kriging, and autocorrelation are techniques used in a stochastic model to estimate properties and uncertainties for materials between measured points in the three-dimensional space of the deposit. A shortcoming of this purely mathematical treatment of geomorphologic data is the absence of knowledge of the processes of deposition and erosion which formed the deposit being modeled. This geologic knowledge and expertise reduces the degree of uncertainty below the values calculated in stochastic models.

If the subsurface geology is not very complex and the location, thickness, and extent of a potentially liquefiable, low blowcount layer can be defined, the problem is greatly simplified. What is needed is a probability density function that represents the uncertainty and variability of the average $N_{1,60}$ over the low blowcount layer. The average $N_{1,60}$ values in the low blowcount layers for liquefied and nonliquefied sites are the values used to develop the Seed-Idriss chart. The probability distribution for an average $N_{1,60}$ has a smaller standard deviation than the distribution for a point value of $N_{1,60}$.

Safety factors in a Seed-Idriss liquefaction analysis are used to estimate residual excess pore pressures for follow-on analyses such as pore pressure redistribution and dissipation, slope stability, bearing capacity, and deformations. The concept is that safety factors against liquefaction greater than unity may still result in damaging levels of residual excess pore pressure from cyclic shaking. Vasquez-Herrera and Dobry (1989) found that if residual excess pore pressures of 40 percent or greater, but less than 100 percent, were generated in a liquefiable soil (strain softening stress-strain curve under undrained conditions), then the soil would creep (deformation under constant load) until a triggering strain level was reached; then the soil would liquefy and its strength would drop to the steady-state or residual strength. Ishihara (1985) and Hynes (1988) show that safety factors against liquefaction of about 1.15 to 1.25 correspond to residual excess pore pressures of 40 percent. Consequently, in a risk analysis, the probability of liquefaction may be centered on a safety factor of 1.2 (± 0.05) rather than unity.

Since $N_{1,60}$ typically has considerable variability in a natural deposit, liquefaction at a single point has a relatively large probability of occurrence (given sufficient loading). Liquefaction over a surface or volume of sufficient size to cause a problem to a structure has a lower probability. However, due to the communication of high pore water pressures to the surrounding material, the effective size of the liquefied zone may be larger than its geometric boundaries. This is observed in the results of effective stress analyses for design of remedial measures that involve rapid dissipation of earthquake-induced residual excess pore pressures (for example, stone columns to increase permeability and shorten the drainage path; see Ledbetter and Finn (1993); Finn, Ledbetter, and Marcuson 1994); these analyses indicate that a liquefaction-resistant layer may develop high residual excess pore pressures if adjacent to liquefied layers. An assessment of the effective volume of potentially liquefiable materials from effective stress analyses may assist in developing cross sections for follow-on stability and deformation analyses and in a risk analysis.

4 UNCERTAINTY IN METHOD OF LIQUEFACTION ANALYSIS

4.1 Uncertainty in the Seed-Idriss Chart

The 1997 evolution of the Seed-Idriss procedure is the standard recommended for practice (Youd and Idriss 1997). Lay, Shich, and Lee (1991) developed a misclassification model for evaluating probability of soil liquefaction with the Seed-Idriss chart. Liao et al. (1988), Youd and Noble (1997) and Liao and Lum (1998) present results of logistical regression of the observational database for occurrence and non-occurrence of liquefaction for the Seed-Idriss chart. As discussed by Youd and Noble (1997), the results from Liao et al. (1988) have been applied in a number of studies to estimate the probability of liquefaction (see for example Budhu et al. 1987; Arango et al. 1996; and FEMA (1997) for loss estimation studies in the HAZUS code).

Logit regression on the liquefaction data base is a technique to quantify the uncertainty in the empirical liquefaction analysis procedure. If P_L is the probability of liquefaction given $N_{1,60}$ or given $N_{1,60}$ and magnitude, the logit transformation is used to yield Q_L, where:

$$Q_L = Logit\ (P_L) = \ln[P_L/(1-P_L)] \tag{4.1}$$

Because probabilities vary from 0 to 1, the logit transformation changes the variable P_L to Q_L, which varies monotonically from $-\infty$ to $+\infty$ as P_L varies from 0 to 1. Q_L can then be expanded through a regular polynomial regression. From Q_L P_L is computed as follows:

$$P_L = 1/[1 + \exp(-Q_L)] \tag{4.2}$$

Liao et al.(1988) conducted regression analyses on a data set of 278 points (CSRN = cyclic shear stress ratio generated at the depth of interest (CSR) normalized to a magnitude of 7.5; $N_{1,60}$). The data were divided into two sets: relatively clean sands with fines less than or equal to 12 percent and silty sands with fines greater than 12 percent. The Seed and Idriss (1982) magnitude scaling factors were used to normalize CSR to CSRN. The logit equation developed by Liao et al. (1988) for fines less than 12 percent (182 data points) is:

$$Q_L = 16.477 - 0.39760\ N_{1,60,CS} + 6.4603\ \ln(CRRN) \tag{4.3}$$

Logit results for Equation 4.3 are shown in Figure 4.1 with the data. The logit equation developed by Liao et al. (1988) for fines greater than or equal to 12 percent (96 data points) is:

$$Q_L = 6.4831 - 0.18190\ N_{1,60,CS} + 2.6854\ \ln(CRRN) \tag{4.4}$$

Figure 4.2 shows the logit results from Equation 4.4 compared to the Seed-Idriss curves for 35 and 15 percent fines. Note that Liao et al. (1988) used the minimum value of $N_{1,60}$ reported rather than an average; consequently, these logit results need to be shifted slightly (to increase $N_{1,60}$) for comparison with the Seed-Idriss chart.

Youd and Noble (1997) re-examined the observational database and added new data for a total of 369 points. They added data from earthquakes with magnitudes less than 7 and performed the logistical regressions with magnitude as an independent variable. They used a fines content correction proposed by Idriss and Seed (see Youd and Idriss 1997) to correct all the data to an equivalent clean sand blowcount value, $N_{1,60,CS}$. Loertscher and Youd (1994) and Youd and Noble (1997) detected points in the database that may have been misclassified in the original construction of the Seed-Idriss baseline curve. The regression equation developed by Youd and Noble (1997) is:

$$Q_L = -7.633 + 2.256\ M_W - 0.258\ N_{1,60,CS} + 3.095\ \ln(CRR) \tag{4.5}$$

The Youd and Noble (1997) logit results for magnitudes from 7.25 to 7.75 are shown in Figure 4.3. With the changes in the data set, the Youd and Noble (1997) logit results plot below those of Liao et al. (1988), as shown in Figure 4.4 along with the Seed-Idriss baseline curve.

Work in progress at the University of California, Berkeley, by Dr. Raymond B. Seed (Seed 1998) at the time of this study indicated that the baseline curve would be moved slightly downward as suggested by Youd and Noble (1997). Additional logit studies by Dr. Seed indicate a bias may exist in the reported observations, favoring occurrences of liquefaction and under-representing occurrences of no liquefaction. The study in progress uses a weight of 1.5 on the no-liquefaction data points to better represent the actual field performance. Dr. Seed used the average of the $N_{1,60}$ values in the low blowcount zone. Dr. Seed incorporated 165 additional data points, mainly from Loma Prieta, Northridge, and Kobe events, and deleted 5 case histories from the Seed et al. (1984) data set due to poor quality in the data and documentation. This is a work in progress, but interim results from Dr. Seed are provided in Figure 4.5 for $N_{1,60,CS}$ and in Figure 4.6 for fines contents of <5, 15, and >35 percent.

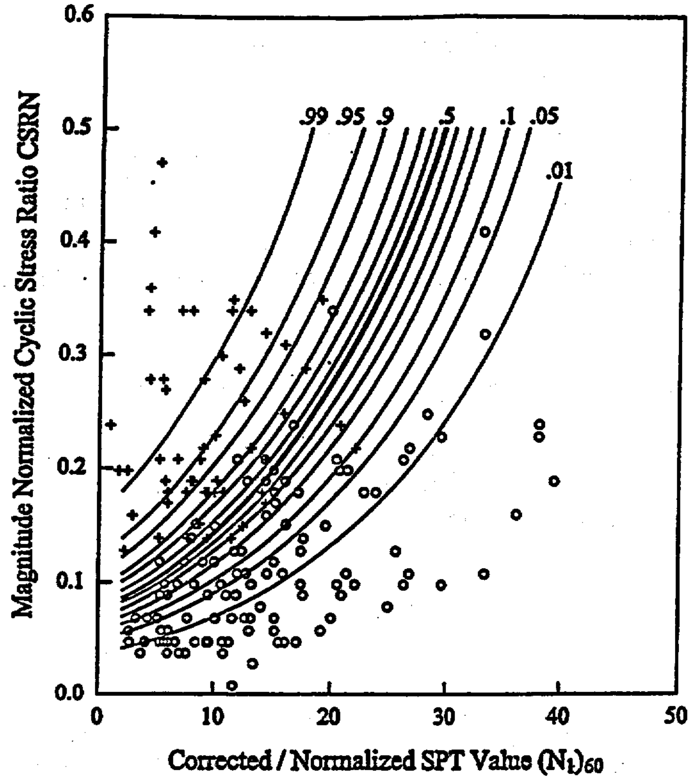


Figure 4.1. Logit results for sands with <12 percent fines after Liao et al. (1988) and Liao (1996) (figure from Youd and Noble 1997)

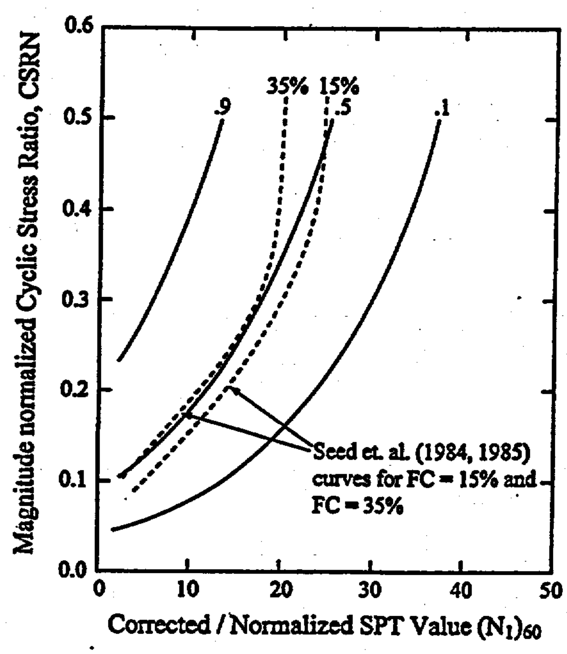


Figure 4.2. Logit results for sands with >12 percent fines after Liao et al. (1988) and Liao (1996) (figure from Youd and Noble 1997)

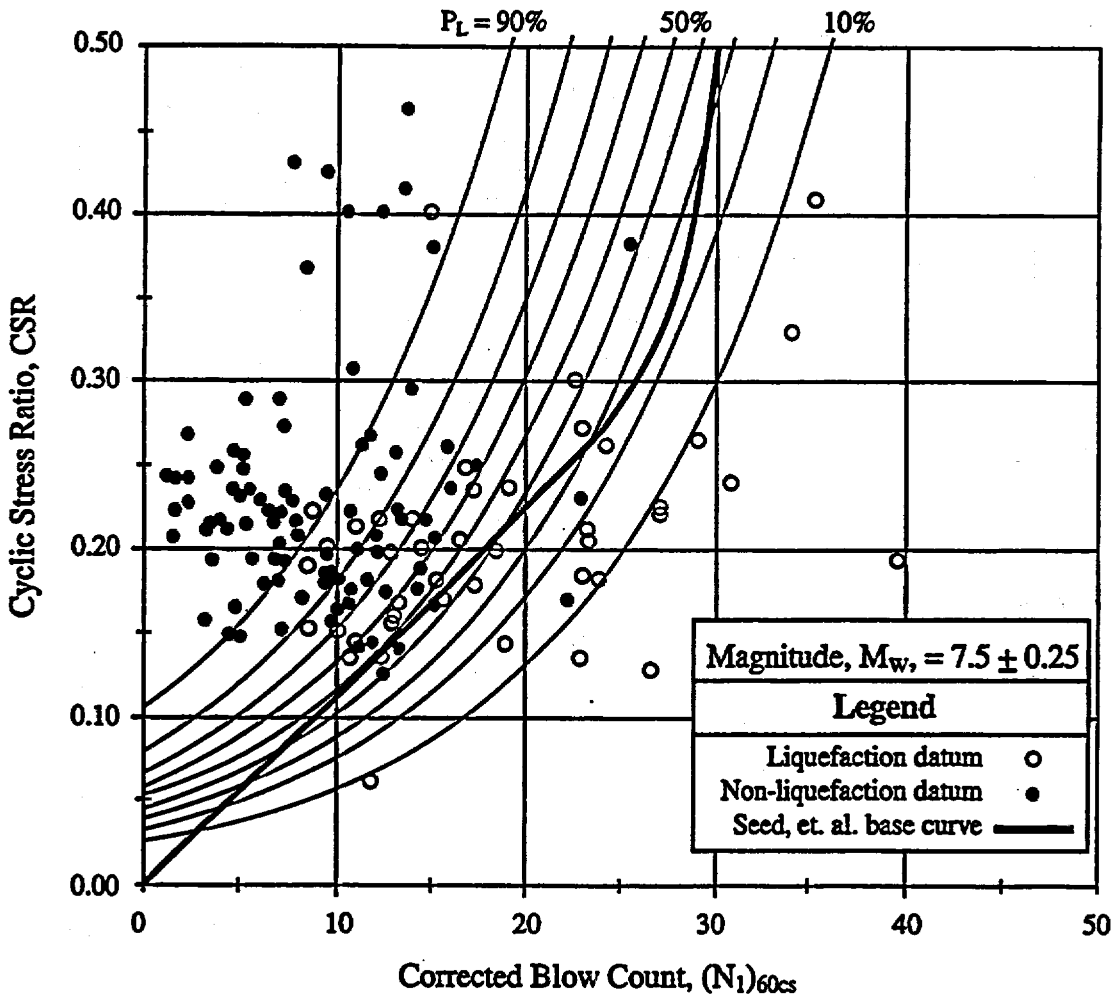


Figure 4.3. Logit results from Youd and Noble (1997)

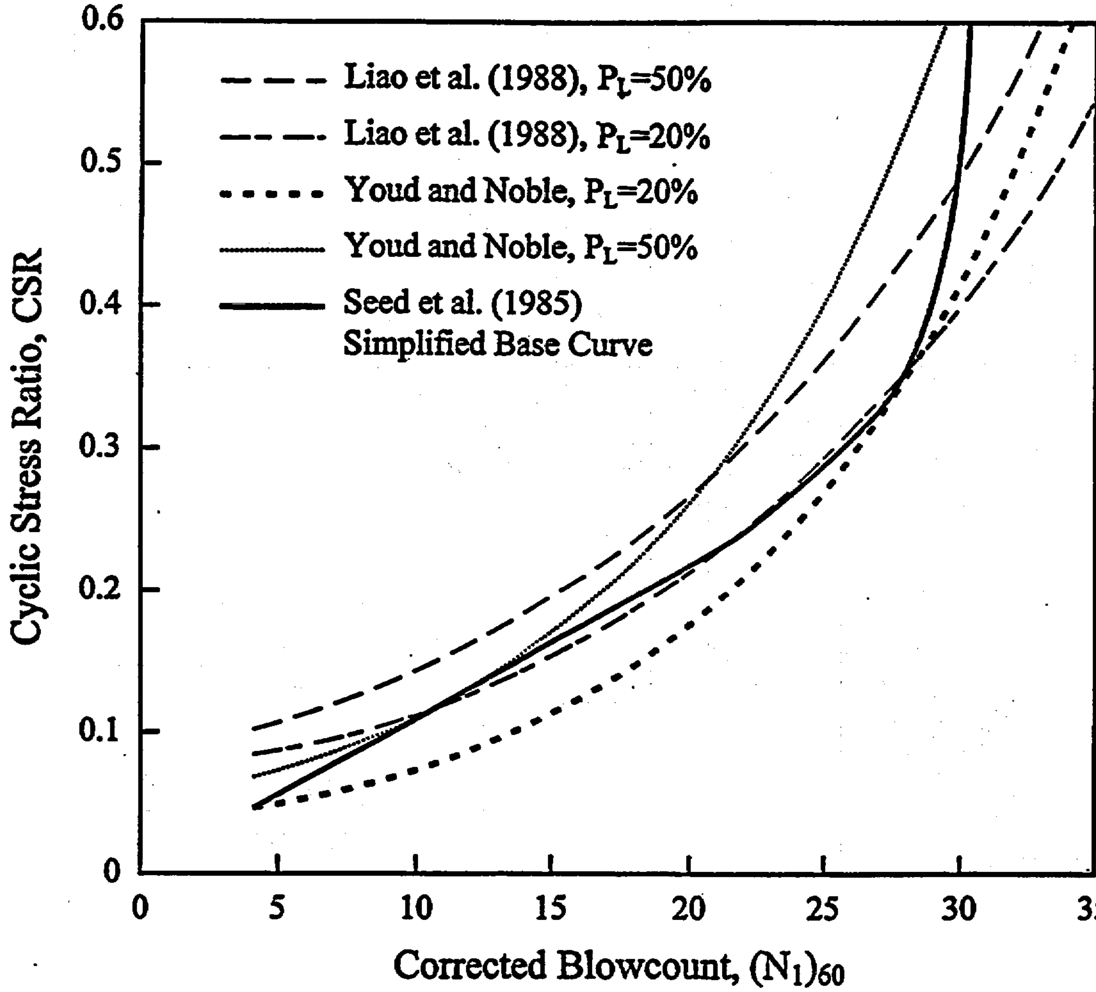


Figure 4.4. Liao et al. (1988) and Youd and Noble (1997) logit results plotted against Seed et al. (1985) curve for clean sands (after Youd and Noble 1997)

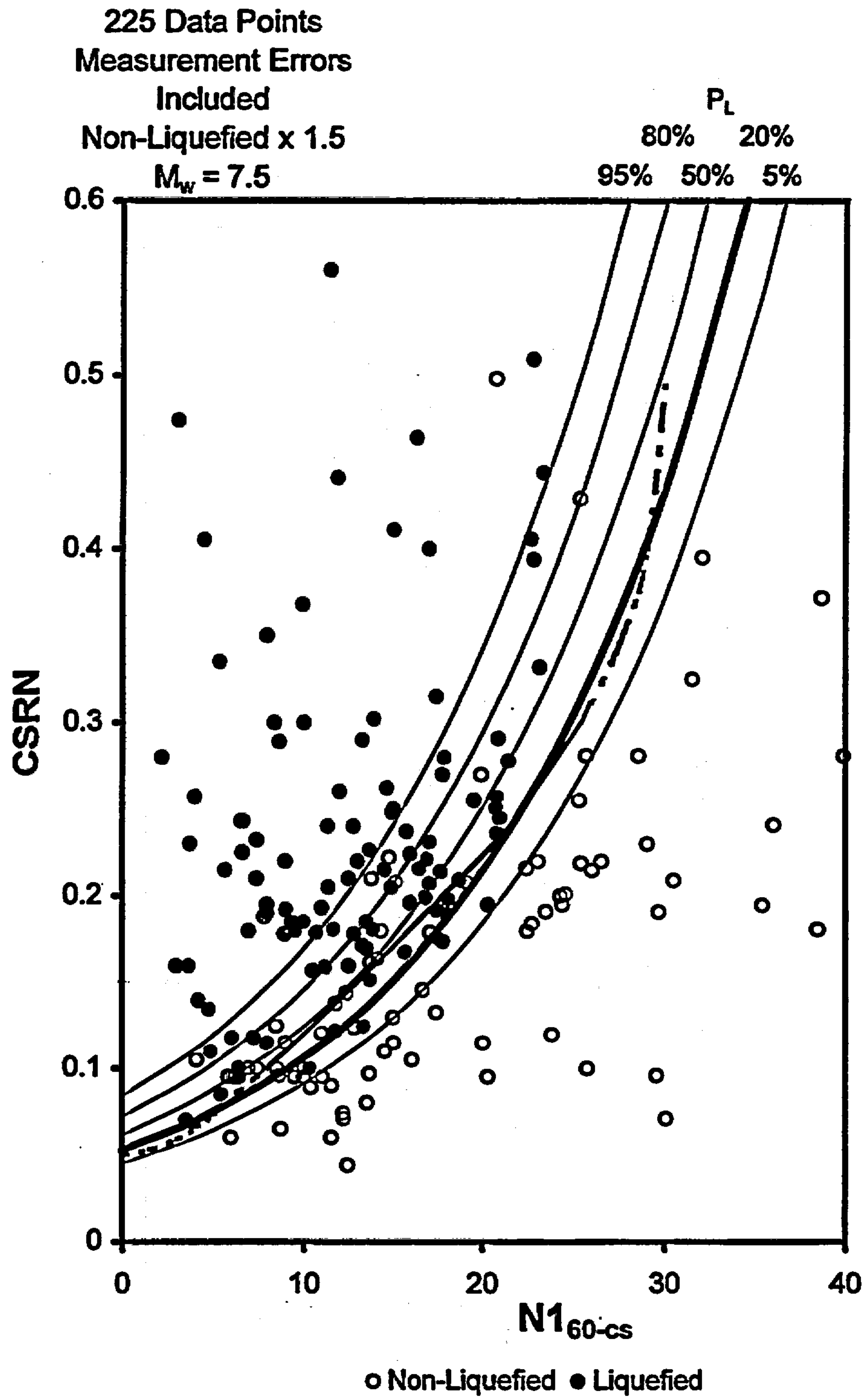


Figure 4.5. Interim logit results from Dr. Seed (1998) for clean sands

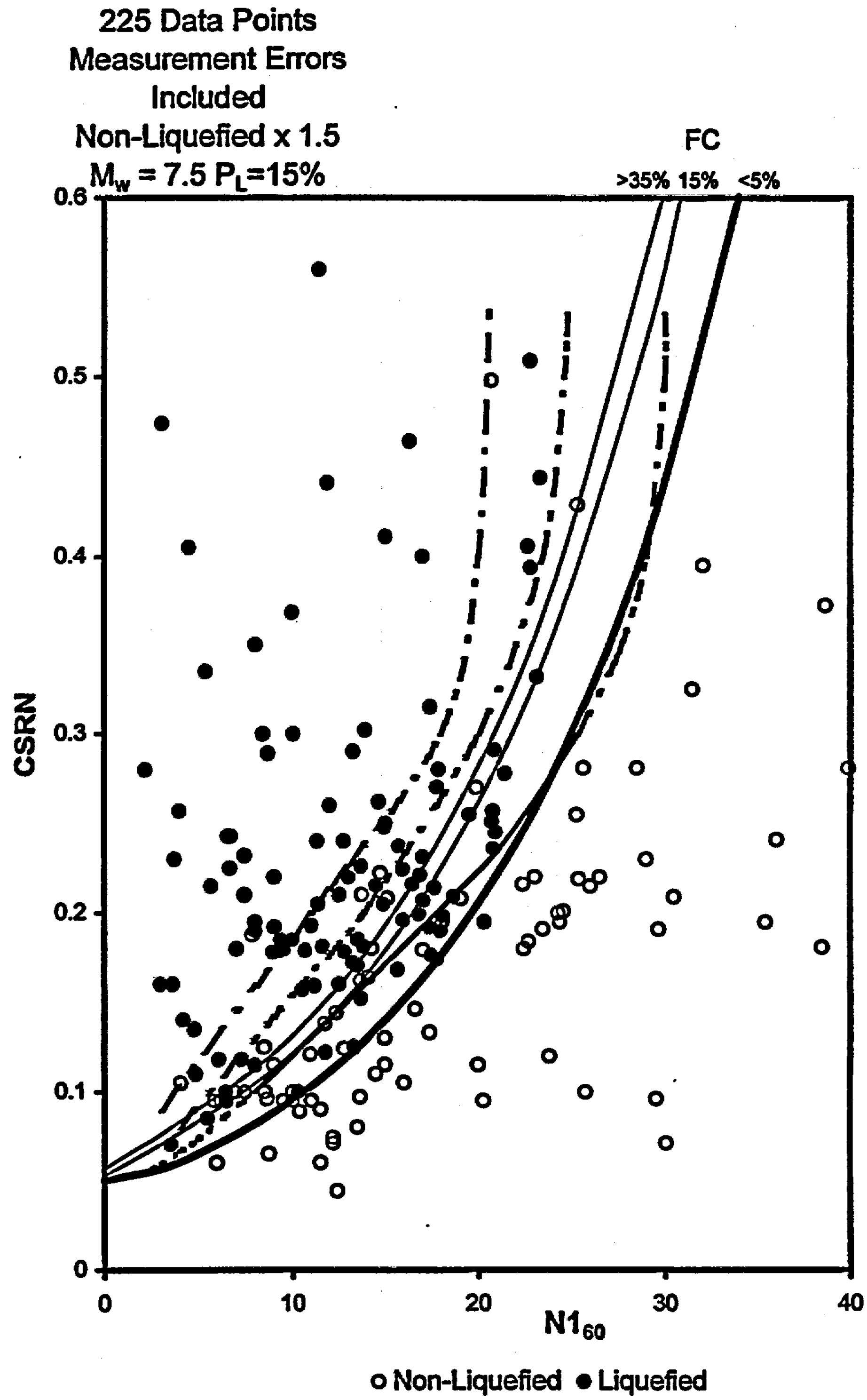


Figure 4.6. Interim logit results from Dr. Seed (1998) for <5, 15, and >35 percent fines

4.2 Other Types of Analysis and Inputs

4.2.1 Cone Penetration Test

Other empirical approaches for liquefaction analysis exist that are based on field observations of liquefaction, laboratory tests, and the type of field measurement used to assess engineering properties. Approaches using Cone Penetration Test (CPT) results (Olsen 1997, Robertson and Wride 1997) are widely used. The CPT is less expensive to perform than SPT and provides a continuous record of penetration resistance of the cone tip and friction resistance of the sleeve behind the tip. Equipment can be mounted on the CPT to measure pore pressure during and between pushes, velocity, electrical resistance, and other properties. Only a few sites that have liquefied have been investigated with the CPT and the data collected has not been made available in usable, digital form to the research community. As with the SPT, careful equipment calibration and field procedures need to be used to minimize the influence of equipment and operator error in measurements. Since the database for direct comparison of CPT to field observations of liquefaction and no liquefaction is quite limited, probabilistic evaluations have relied primarily on translating the CPT to an equivalent SPT $N_{1,60}$, and then using the SPT procedure (see for example Arango et al. 1996).

4.2.2 Shear Wave Velocity, Becker and Large Penetrometer Tests

Shear wave velocity is also an indicator of liquefaction potential. The database for this technique is documented by Andrus and Stokoe (1997) and could be used for a probabilistic model; however, the shear wave velocity technique is still developing and the supporting database needs to be enlarged. Shear wave velocity is also used in the "threshold strain" approach (Dobry et al. 1982, Hynes 1988); however, this approach is a conservative, preliminary screening tool. Shear wave velocity is particularly useful for investigation of coarse-grained deposits where SPT and CPT are not practical. Alternative penetrometers such as the Becker Penetration Test (BPT; Harder 1988; Sy, Campanella, and Stewart 1995; Harder 1997) and Large Penetrometer Test (LPT) (see Ishihara 1996) have been used to investigate liquefaction potential in gravelly soils. These approaches are primarily aimed at translating the BPT and LPT results to equivalent values of $N_{1,60}$.

4.2.3 Arias Intensity and SPT

As part of this study, a logit analysis was conducted for the Arias Intensity-SPT liquefaction chart developed by Kayen (Kayen 1993; Kayen and Mitchell 1997). The total horizontal Arias Intensity, I_h, is defined as:

$$I_h = I_{xx} + I_{yy} = (\pi/(2g))\int_0^t [a_x^2(t) + a_y^2(t)]dt \tag{4.6}$$

The Kayen chart uses Arias Intensity at the depth of the liquefied layer, I_{hb}. This is computed as the product of I_h and an Arias Intensity depth reduction factor r_d (see Youd and Idriss 1997).

The Arias Intensity approach internalizes earthquake magnitude corrections since I_{hb} is an indicator of the energy applied at the location of the liquefied material in the soil profile. Attenuation functions for Arias Intensity were being developed at the time of this study. Consequently, PSHA results for Arias Intensity are not readily available, but may be in the near future. The logit results developed in this study are plotted in Figure 4.7. The $N_{1,60}$ values (average values in the low blowcount zone as used by Seed and Idriss 1982) were converted to equivalent clean sand values, $N_{1,60,CS}$, with the fines content correction developed by Olsen (1997). The regression developed for the Kayen (1993) data set is:

$$Q_L = Logit\ (P_L) = \ln[P_L/(1-P_L)] = 8.3324 + 3.97718 \times \ln(I_{hb}) - 0.381227\ N_{1,60,CS} \qquad (4.7)$$

and, as above, the probability of liquefaction given the blowcount information can be computed with Equation 4.2.

4.2.4 Liquefiable Fine-Grained Soils

High water content, low plasticity silts, and clays are also potentially liquefiable. The empirical database for this was published by Wang (1981) for several sites in China for earthquakes ranging in intensity from MMI VII to IX (note: MMI VII could correspond to earthquake magnitudes as low as 5.4, from Krinitzsky 1995c). Criteria for assessing liquefaction potential of these soils was published by Seed and Idriss (1982). The input properties for the procedure are index tests, namely gradation (including hydrometer), liquid and plastic limits, and natural water content. A probabilistic model could be developed to characterize the database and typical testing errors in laboratory measurement of index properties. Vane shear testing has also been an informative technique for estimating residual strength of these soils. Laboratory testing of these materials is also possible; however, the materials are extremely difficult to sample, cannot be frozen, and are subject to large volume changes when reconsolidated in the laboratory to in situ stress conditions. The laboratory tests do not perfectly simulate the loading and boundary conditions in the field, as mentioned earlier.

4.2.5 Steady-State Liquefaction Evaluation

Kramer (1989) developed a probabilistic model for dealing with the uncertainties in steady-state approaches to liquefaction and flow failure (Poulos, Castro, and France 1985). The steady-state approach involves laboratory testing of soil samples, careful measurement of in situ densities, and calculation of initial stress state and earthquake-induced cyclic and permanent shear strain. Liquefaction and flow occur if the material is contractive under added shear strain and if the earthquake-induced shear strain exceeds the triggering strain level determined from the laboratory tests. This description of the steady-state approach indicates the types of parameters that could be considered as random variables in a probabilistic formulation.

4.2.6 Advanced Constitutive Models and Deformation Codes

Another type of liquefaction evaluation technique is an advanced constitutive model in a numerical stress, deformation, and wave propagation analysis code. The liquefaction evaluation procedures discussed previously are decoupled approaches (such as SHAKE, FLUSH, QUAD4; Schnabel, Lysmer, and Seed 1972; Lysmer et al. 1973; and Idriss et al. 1973, respectively), since the load is decoupled from the resistance throughout the analysis. In partially coupled or direct models (POROSLAM, FLAC, TARA; Simos et al. 1996; Itasca 1991; Finn et al. 1986, respectively), the load is decoupled from resistance for a portion of a time increment or stress cycle in a time domain calculation. In a fully-coupled model (DYNAFLOW; Prevost 1981), the load and resistance are related throughout the calculation. Published probabilistic assessments of the accuracy or bias in this class of numerical liquefaction analysis codes were not found during this study.

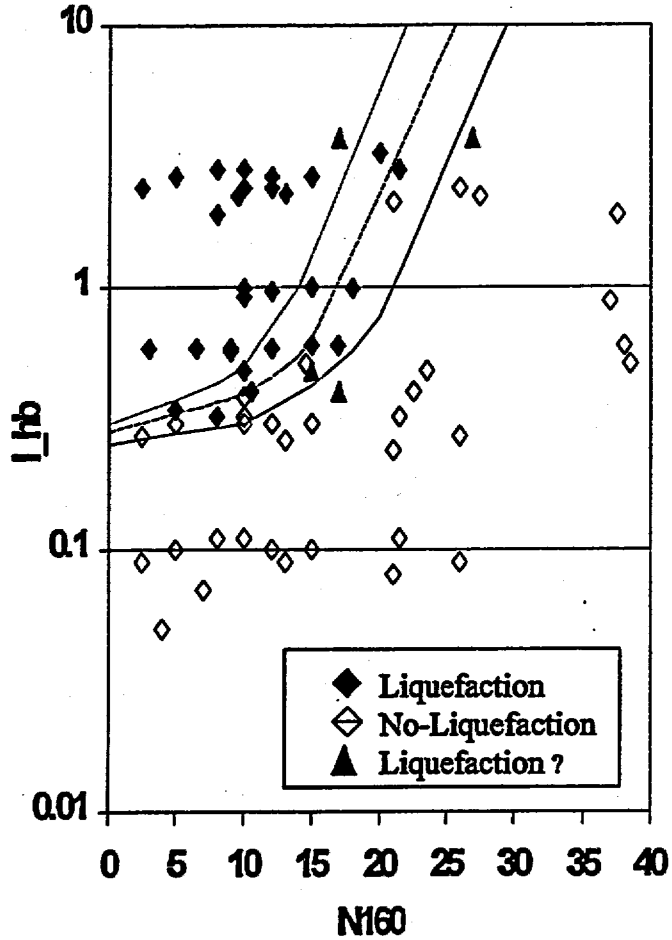


Figure 4.7. Logit results for Arias Intensity (this study) from data by Kayen and Mitchell (1997) (reprinted with permission from ASCE)

5 CAPACITY-DEMAND MODEL

The numerical formulation to compute probability of liquefaction comes from a classic capacity-demand model (Benjamin and Cornell 1970), also known as interference theory in reliability computations (Kapur and Lamberson 1977). The demand is the earthquake cyclic shear stress (or strain) or cyclic stress ratio (CSR), the capacity is the soil cyclic shear strength (or triggering strain level) or cyclic resistance ratio (CRR), and the method of liquefaction analysis defines the threshold values of load that will result in liquefaction. The probability of liquefaction is then:

$$P[liquefaction] = P[CSR > CRR] \tag{5.1}$$

In a simplified analysis, CSR can be considered a function of acceleration and magnitude, and CRR can be considered a function of $N_{1,60}$. Uncertainty in CSR can be determined from a PSHA deaggregated on acceleration and magnitude. The uncertainty in $N_{1,60}$ can be estimated from analysis of variability of data from the site, modified by expert opinion, such as geologic knowledge of the deposit and past performance of the site and similar deposits during earthquakes.

Note that the CSR and the CRR are actually not independent quantities. The cyclic stress ratio at a point or over a zone in a deposit is a function of the energy delivered to the deposit by the earthquake (Arias Intensity or combinations of acceleration and magnitude) and the mass and stiffness of the soil deposit. The cyclic resistance ratio is related to the density and stiffness of the deposit, and is indicated by shear wave velocity, density, and blowcounts. However, for simplicity, if it is assumed that CSR and CRR are independent, then the probability of liquefaction can be determined as:

$$P[liquefaction] = \int\limits_{\substack{all \\ N_{1,60}}} \int\limits_{\substack{all \\ CRR}} G_{CSR}(crr)\, f_{CRR|N1,60}(crr|N_{1,60} = n_{1,60})\, F_{N1,60}(n_{1,60})\; dcrr\; dn_{1,60} \tag{5.2}$$

where the complementary cumulative distribution for CSR is G_{CSR}, the conditional probability density function for CRR given $N_{1,60}$ is $f_{CRR\,|\,N1,60}$ from the logit analyses, and the probability density function for $N_{1,60}$ is $f_{N1,60}$.

A threshold chart based on the 1997 evolution of the Seed-Idriss SPT procedure (which corresponds to a probability of liquefaction of about 15 percent according to Seed 1998) is shown in Figure 5.1 for $N_{1,60,cs}$ values of 5,10, 15, and 20. For a given value of $N_{1,60,cs}$, deaggregation matrices from a PSHA can be used to compute the annual probability of exceeding safe threshold combinations of PGA and M.

An example problem is provided to illustrate the steps in a probabilistic liquefaction calculation. For simplicity, the problem is discretized and expert opinion is used to assign probability mass values to the average $N_{1,60}$ in a loose foundation deposit. Deaggregated hazard curves are shown in Figure 5.2, adapted from a PSHA for a site in California with a relatively quiet near-field seismic source capable of a magnitude 6.5 earthquake and the San Andreas fault zone as a far-field seismic source. Two magnitude bins are used to describe the seismic hazard, magnitude 6 ± 0.5 and magnitude 8 ± 0.5. For this example, the magnitude bin 7 ± 0.5 had extremely low probability values, several orders of magnitude less likely than the other two bins.

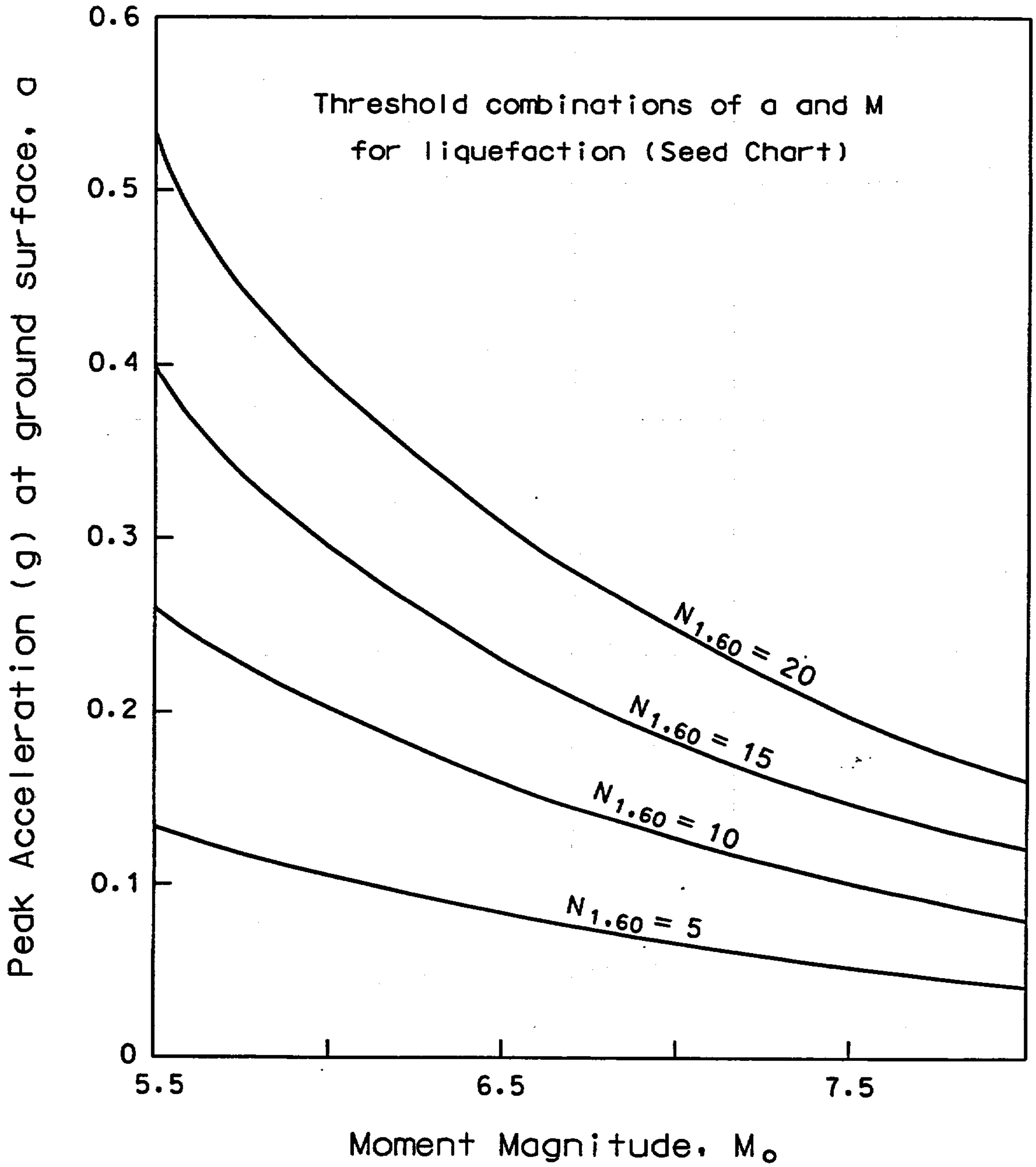


Figure 5.1. Triggering combination of peak ground acceleration and magnitude given $N_{1,60}$

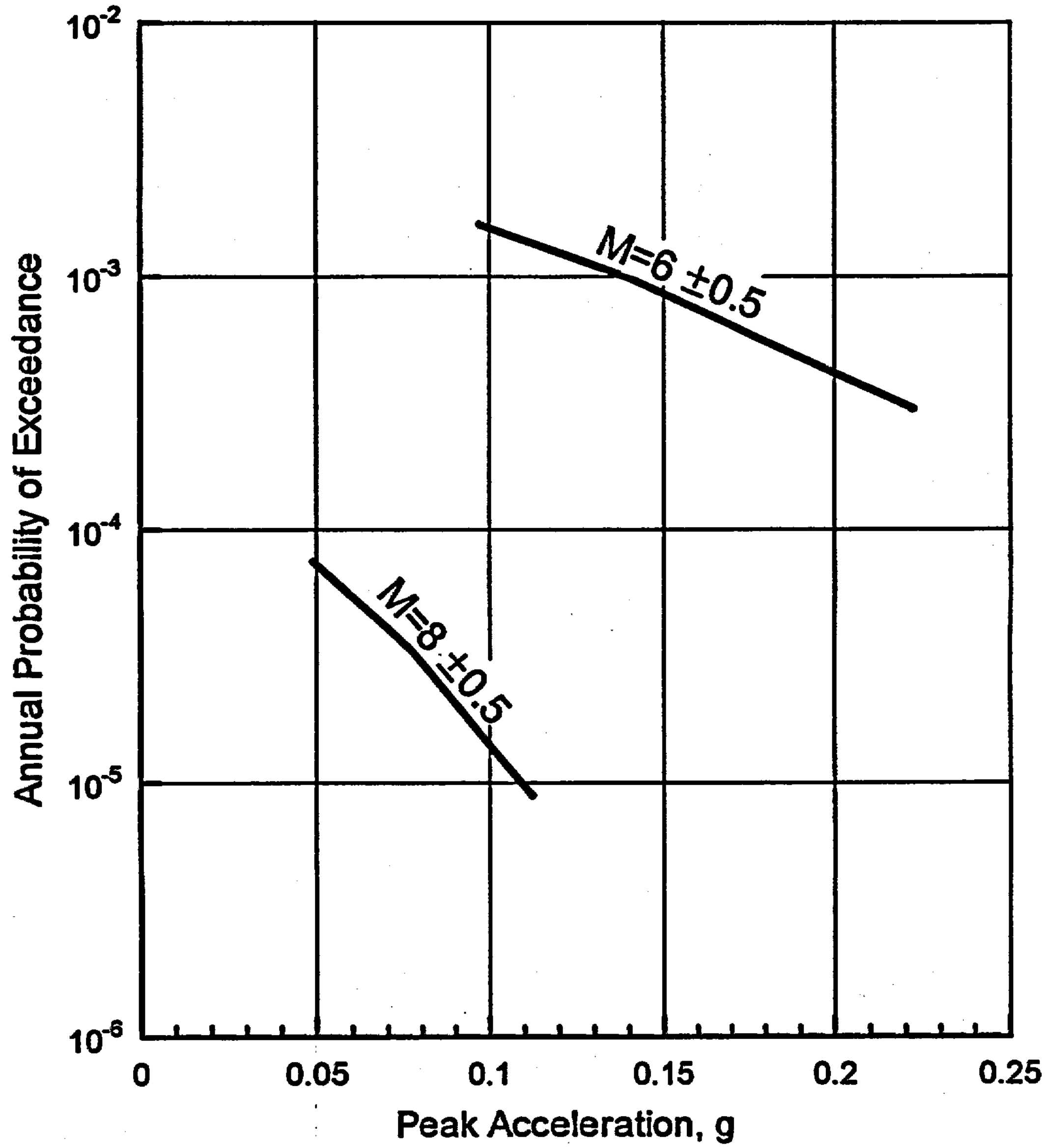


Figure 5.2. Deaggregated PSHA hazard curves for example problem

The cyclic stress ratio is computed with the Idriss (1999) simplified method:

$$CSR = 0.65\ PGA\ [\sigma_v / \sigma_v'] \ r_d \tag{5.3}$$

where

PGA = peak ground acceleration (g)
σ_v = total vertical stress
σ_v' = effective vertical stress
r_d = depth reduction factor, a function of magnitude

The deposit for this example consists of a hydraulically-placed soil with little or no fines. Because of the method of deposition and recent age of the deposit, the relative density of the material is quite uniform with depth. Examination of numerous borings indicates average values of $N_{1,60}$ of about 8. Subjective assignment of probability mass yields $P[N_{1,60} = 6 \pm 1] = 0.05$, $P[N_{1,60} = 8 \pm 1] = 0.75$ and $P[N_{1,60} = 10 \pm 1] = 0.20$.

The logit chart from Seed (1998) was used to estimate $f_{CRR|N1,60}$ in a discretized form with probability masses of 0.2, 0.6, and 0.2 for the nine values of CRR corresponding to the three values of average $N_{1,60}$. A magnitude correction factor of 1.46 for magnitude 6 and 0.89 for magnitude 8 were used, from Idriss (1999). Given the $N_{1,60}$, the corresponding CRR can be computed. The CSR is set equal to CRR, and unsafe combinations of magnitude and PGA are determined from Equation 5.3. A factor of safety of one against liquefaction was used in this example.

The computations are summarized in Table 5.1. The result is a return period of 1140 yrs for a damaging level earthquake sufficient to cause liquefaction. If the standard 1997 Seed-Idriss procedure (Figure 5.1) is used and uncertainty in the Seed-Idriss chart and $N_{1,60}$ are ignored, the resulting return period is 1380 yrs. Arango et al. (1996) and other papers listed in the bibliography provide further examples.

A similar approach can be developed for Arias Intensity, as illustrated in Figure 5.3. At this time, probabilistic seismic hazard maps for Arias Intensity are not readily available, but may be in the near future.

6 DISCUSSION AND CLOSURE

Conclusions and recommendations from this review of existing work on probabilistic liquefaction analysis are:

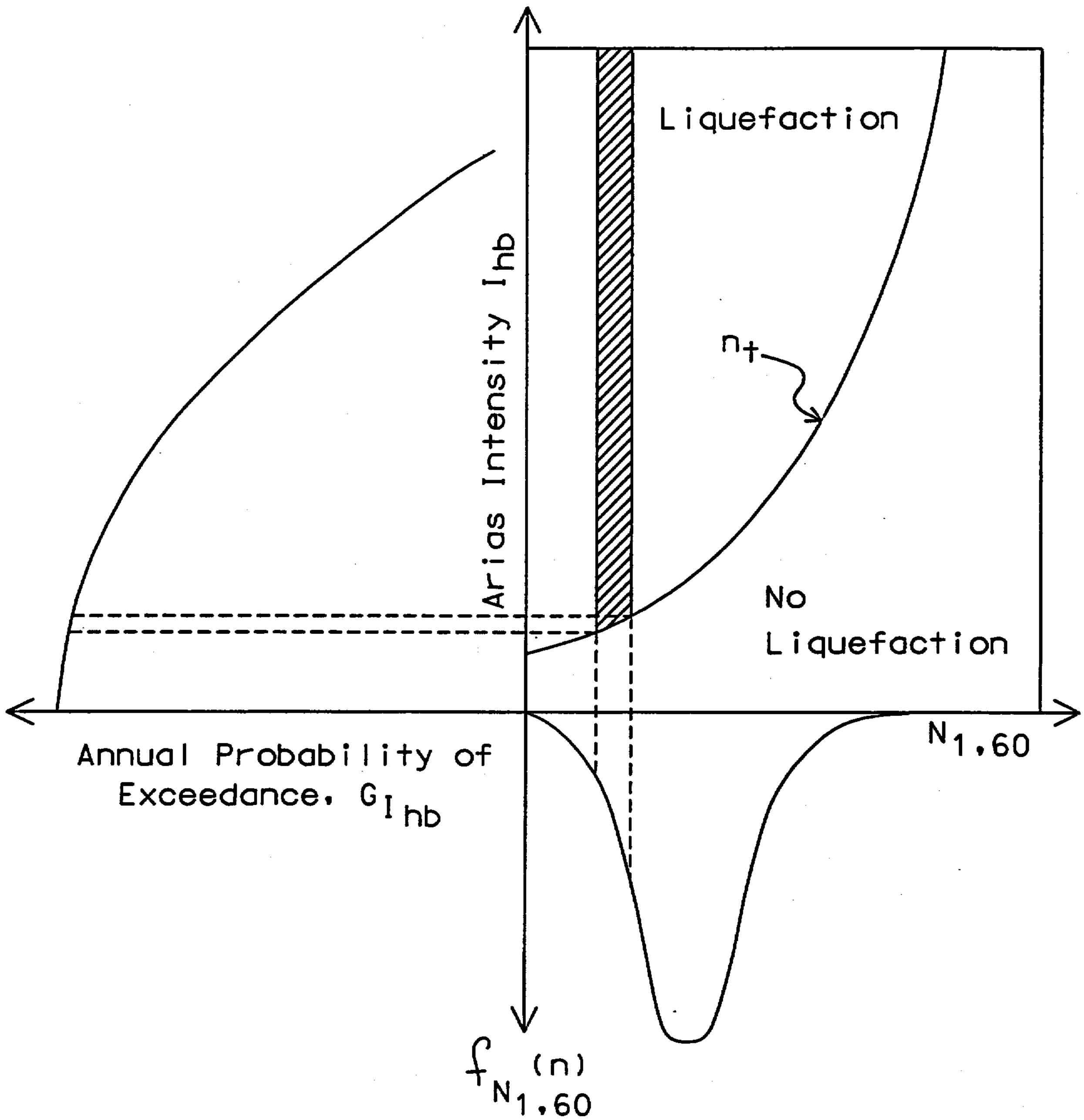
(a) There is no single, well established procedure extant at this time for modeling and quantifying uncertainties in liquefaction evaluations.

(b) Evaluation of the probability of liquefaction should build upon commonly accepted deterministic procedures and incorporate the uncertainties in demand and capacity.

(c) Modeling a large number of input parameters as random variables in more sophisticated models does not necessarily yield better results.

Table 5.1 Probability of Liquefaction Calculation for Example Problem

In-situ $N_{1,60}$ ($P[N_{1,60}]$)	Cyclic Resistance CRR ($P[CRR \mid N_{1,60}]$)	Triggering Earthquake Load for Liquefaction, CSR		Annual Probability of Exceedance			
		magnitude	acceleration g	magnitude contribution	sum over magnitude given CRR	weighted sum over CRR	weighted sum over $N_{1,60}$
6 (0.05)	0.07 (0.20)	6 ±0.5	0.097	1.6×10^{-3}	1.675×10^{-3}	1.164×10^{-3}	0.877×10^{-3}
		8 ± 0.5	0.049	7.5×10^{-5}			Annual return period =
	0.09 (0.60)	6 ± 0.5	0.125	1.1×10^{-3}	1.15×10^{-3}		$(0.877 \times 10^{-3})^{-1}$
		8 ± 0.5	0.063	5.0×10^{-5}			= 1140 yrs
	0.12 (0.20)	6 ± 0.5	0.166	6.7×10^{-4}	6.95×10^{-4}		
		8 ± 0.5	0.084	2.5×10^{-5}			
8 (0.75)	0.08 (0.20)	6 ± 0.5	0.111	1.5×10^{-3}	1.56×10^{-3}	0.903×10^{-3}	
		8 ± 0.5	0.056	6.0×10^{-5}			
	0.11 (0.60)	6 ± 0.5	0.153	8.0×10^{-4}	8.3×10^{-4}		
		8 ± 0.5	0.077	3.0×10^{-5}			
	0.14 (0.20)	6 ± 0.5	0.194	4.5×10^{-4}	4.65×10^{-4}		
		8 ± 0.5	0.098	1.5×10^{-5}			
10 (0.20)	0.09 (0.20)	6 ± 0.5	0.125	1.1×10^{-3}	1.15×10^{-3}	0.709×10^{-3}	
		8 ± 0.5	0.063	5.0×10^{-5}			
	0.12 (0.60)	6 ± 0.5	0.166	6.7×10^{-4}	6.95×10^{-4}		
		8 ± 0.5	0.084	2.5×10^{-5}			
	0.16 (0.20)	6 ± 0.5	0.222	3.0×10^{-4}	3.09×10^{-4}		
		8 ± 0.5	0.112	9.0×10^{-6}			



$$\text{Annual Probability of Liquefaction} = \int_{\text{all } N_{1,60}} G_{I_{hb}}(n_t)\, f_{N_{1,60}}(n_t)\, dn$$

Figure 5.3. Approach to determine annual probability of liquefaction using Arias Intensity

(d) A fairly simplified analysis that focuses on the key parameters that control liquefaction may yield practical, reasonable results. The level of sophistication needed for solving the deterministic problem can be used as a guide for developing the probabilistic model.

(e) The probabilistic study needs to build a logical development of the approach used and show that the dominant parameters have been modeled.

(f) The three main elements to be modeled are:

1. Uncertainty in the earthquake load ("demand" from a PSHA in terms such as acceleration and magnitude or earthquake-induced cyclic shear stress ratio at depth or Arias Intensity at depth).

2. Uncertainty in the earthquake resistance ("capacity" from modeling the spatial distribution and variability of cyclic shear strength inferred from laboratory or in situ test measurements).

3. Uncertainty in the method of liquefaction evaluation (such as logit results).

(g) Estimates of probability should be obtained from a combination of theory, expert subjective input, supporting databases, site-specific data, and computation.

(h) Generation and use of earthquake records artificially generated from signal processing techniques widely used for probabilistic structural dynamics should be discouraged for liquefaction and follow-on nonlinear analyses such as permanent deformation analyses.

(i) The purpose of a probabilistic analysis is usually to provide an ordered framework for examining the seriousness of the risk of occurrence of a particular event in the context of the range of events possible. A number of examples in the literature demonstrate the application of probability of liquefaction in risk assessment and evaluation of a range of earthquake scenarios.

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APPENDIX A
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KEY WORDS: Soil liquefaction, mapping, probability, mathematical models, seismology

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11. ABSTRACT *(200 words or less)*

This document provides technical bases for formulating probabilistic approaches to liquefaction evaluation. The three basic elements of probabilistic liquefaction analysis are: (1) uncertainty in the earthquake load, (2) uncertainty in the available resistance, and (3) uncertainty in the method of analysis. The probabilistic approach is built from the steps in a deterministic liquefaction analysis; however, the input parameters, such as penetration resistance, site stratigraphy, acceleration, and magnitude, are treated as random variables and the accuracy of the method of analysis is factored in as a part of a capacity-demand model. Uncertainty in the earthquake load is generally treated with a probabilistic seismic hazard analysis, which introduces time as a parameter. The site stratigraphy and engineering properties are generally treated as one-, two-, or three-dimensional random fields. Uncertainty in the method of analysis is generally estimated with logit regression analysis of the field performance data base. It is assumed that the reader has a working knowledge of probability theory, stochastic processes, liquefaction evaluation, and probabilistic seismic hazard analysis calculations.

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