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CPT-Based Probabilistic Assessment of Seismic Soil Liquefaction Initiation

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16. Abstract

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Case histories of occurrence and non-occurrence of soil liquefaction were collected from seismic events that occurred over the past three decades. These were carefully processed to develop improved CPT-based correlations for prediction of the likelihood of "triggering" or initiation of soil liquefaction during earthquakes. Important advances over previous efforts include

(1) Collection of a larger suite of case histories,
(2) Development of an improved treatment of CPT thin-layer corrections,
(3) Improved treatment of normalization of CPT tip and sleeve resistances for effective overburden stress effects,
(4) Improved evaluation of the cyclic stress ratio (CSR) in back-analyses of field case histories,
(5) Assessment of uncertainties of all key parameters in back-analyses of field case histories,
(6) Evaluation and screening of case histories on the basis of overall uncertainty, and
(7) Use of higher-order (Bayesian) regression tools.

The resultant correlations provide improved estimates of liquefaction potential, as well as quantified estimates of uncertainty. The new correlations also provide insight regarding adjustment of CPT tip resistance for effects of "fines" content and soil character for purposes of CPT-based liquefaction hazard assessment.

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CONTENTS

LIST OF FIGURES

LIST OF TABLES

1 Introduction

Seismically induced soil liquefaction is a leading cause of damage and loss during earthquakes. This earthquake phenomenon is a function of liquefaction resistance of the soil in relation to the cyclic stress induced by ground shaking. Liquefaction that occurs in a built-up environment can be a significant human hazard. The objective of this research is to define, in the most accurate and unbiased manner possible, the likelihood of initiation, or "triggering," of seismically induced soil liquefaction.

Laboratory testing to assess the liquefiability of *in situ* soils is prone to sampling disturbance problems, and so fails to fully capture some of the more important variables such as prior seismic history, aging effects, and field stress conditions, to name a few. The correlation of seismic field performance with *in situ* index tests has shown good results in assessing the likelihood of initiation of liquefaction. The research reported herein presents correlations for assessing liquefaction susceptibility based on the cone penetration test (CPT).

In order to make the correlations as accurate and unbiased as possible, several important details relating to the interpretation of CPT data had to be worked out. This includes the problems of accurate interpretation of CPT measurements in thin interbedded strata, and appropriate normalization of both tip and sleeve resistance measurements for the effects of varying effective overburden stress.

A correlation is only as good as the quality of the data upon which it is based. One key objective was to assemble a database of the most highly scrutinized and consistently processed liquefaction and non-liquefaction field case histories available. To achieve this, strict protocols were established for processing and grading case history data according to the quality of information content. This database was then submitted for review to a panel of liquefaction experts.

Proper treatment of the resulting processed and screened data required a flexible and powerful statistical technique. Bayesian analysis provides just such a tool. This statistical technique can accommodate all forms of uncertainty associated with both the phenomena of liquefaction and our attempt to quantify this phenomenon. This technique also has the flexibility to fit any given mathematical form describing the physics of the failure mechanism. Reliability techniques are used to present the results in a probabilistic framework.

2 Previous Studies

This work was undertaken to fill important gaps that were left by previous, similar CPT-based studies. A number of CPT-based liquefaction-triggering correlations have been published, but only the most common and commonly used are discussed here.

The most frequently used correlation to date is that proposed by Robertson and Wride (1998) as presented in NCEER (1997) and Youd et al. (2001). This work provides the most usable and comprehensive CPT-based assessment of liquefaction triggering available. Some of the deficiencies of this work include lack of probabilistic assessment, inconsistent treatment and processing of the field case histories, unconservative assessment of the effects of "fines" on soil liquefiability, and overly simplified treatment of normalization of CPT tip resistance for effective overburden stress effects. The result is a methodology with an undefined level of uncertainty, and one that is unconservative in soils with a significant percentage of fines.

Other well-known studies, including Shibata and Teparaska (1988), Stark and Olson (1995), Suzuki et al. (1995), all employ a more limited database of field performance case histories than Robertson and Wride (1998). On the theoretical side, Mitchell and Tseng (1990) presented a correlation that was based on cavity expansion analyses, validated with laboratory cyclic simple shear and cyclic triaxial testing data. This work is valuable for bounding empirical results and providing a theoretical backbone but is based on a limited amount of data. Recent work by Juang et al. (2000, 2003) presents probabilistic results but uses a database with the same deficiencies as Robertson and Wride (1998).

3 Current Research Approach

Important advances over similar previous efforts include

1. Collection of a larger suite of case histories covering the last three decades of seismic events. Over 500 case histories were collected of which 188 case histories passed the screening process and were included in the final database.
2. Improved treatment of CPT thin-layer corrections.
3. Development of an improved treatment for the normalization of CPT tip and sleeve resistances for effective overburden stress effects based on comprehensive theoretical results and empirical evidence.
4. Improved evaluation of cyclic stress ratio (CSR) in back-analyses of field case histories. This includes the assessment of PGA via the best available method; strong motion recordings, site response, calibrated attenuation relationships, adjustment of estimated site PGA through general site response modeling, and general attenuation relationships.
5. Assessment of uncertainties of all key parameters in back-analyses of field case histories by quantifying the vital statistics for each parameter.
6. Evaluation and screening of case histories on the basis of overall uncertainty. The screening process provides a consistent framework for determining if a particular case history is sufficiently characterized to provide useful information as to the threshold of liquefaction.
7. Use of higher-order (Bayesian) regression tools and structural reliability methods for determining the best mathematical model for describing the relationship between CPT measurements and the manifestation of liquefaction as well as assessing the probability of liquefaction occurrence.

The resultant correlations provide improved estimates of liquefaction potential, quantified estimates of uncertainty, and a better understanding of the adjustment of CPT tip resistance for the effects of "fines" content and soil character for the purpose of CPT-based liquefaction hazard assessment.

4 Data Processing

In order to have an unbiased estimate of the occurrence or non-occurrence of liquefaction it is of preeminent importance to have the highest quality data. A probabilistic correlation requires powerful statistical techniques, but it is only as good as the quality of data to which the techniques are applied. To this end, data processing was of utmost importance in this study. A considerable amount of time was spent processing and reviewing the database to minimize epistemic uncertainty that can creep in due to human error, biased interpretation, and poor analysis techniques.

4.1 FIELD OBSERVATIONS

A liquefaction case history is based on a research engineer's observation of liquefaction or absence of liquefaction following a seismic event, and the index test measurements of the suspect critical layer. This basis is inherently fraught with uncertainty including lack of full coverage of affected area, misinterpretation of field evidence, poor index testing procedures, and difficult field conditions.

One of the primary discrepancies of a database of this type is that researchers tend to measure and report more liquefied than non-liquefied case histories. This can be attributed to the fact that testing in a liquefied area is much more appealing than testing in an area that hasn't experienced liquefaction. This unfortunately leads to a data bias; more liquefied case histories than non-liquefied case histories. To account for this data imbalance the procedure of bias weighting, as described later, is used.

Liquefaction field correlations are not truly based on the occurrence or non-occurrence of liquefaction but on observation of the manifestations of liquefaction at a particular location and the lack thereof at another. These manifestations can take the form of sand boils or sand blows, lateral spreading, building tilting or settlement, ground loss, and broken lifelines. Liquefaction can and does occur at depths where there is no surface evidence of the event, but this research does not explicitly address that particular situation.

The most content-rich sites are those labeled as marginal. Marginal liquefaction does not exist: a soil deposit either liquefies or does not liquefy. Marginal is a research engineer's interpretation that liquefaction was either incipient or occurred and resulted in minimal surface manifestations. These sites are included in the database and tend to have the most information content because they fall near the threshold of liquefaction/non-liquefaction.

All these vagaries are incorporated into the database and result in epistemic uncertainty. To minimize this uncertainty a panel of experts reviewed the database and came to a consensus on each site and the data it contained. This process of consensus resulted in a robust database that contains the best assessment of each variable to the highest standards of practice.

4.2 CHOICE OF LOGS

At any given site there can be multiple CPT and SPT logs. The proximity of the logs to the observed liquefaction/non-liquefaction is critical. The depositional environment and the properties that lead to liquefaction can vary significantly over small distances, so it is important to be as close to the observed location as possible. Logs that are considered to be representative of the conditions were chosen. When there are multiple logs, the values (such as tip and sleeve resistances) are averaged.

CPT logs that were measured using a mechanical cone or a sleeveless cone are not used in this database because of the lack of sleeve measurements. However, when a sleeveless cone trace has an adjacent SPT log that shows that the critical layer is composed of clean sand ($FC<5\%$), then the tip resistance is used in conjunction with a prescribed median "clean sand" friction ratio ($R_f \cong 0.35\%$). This allows the use of important early CPT case histories with a neutral friction ratio.

A few earthquake reconnaissance efforts have utilized a Chinese cone. The report by Earth Technology (1985) showed that there is very little difference between tip and sleeve readings using the Chinese cone compared with a cone following ASTM specifications (D3441 and D5778). Therefore the Chinese cone was treated no differently in this database.

4.3 CASE SELECTION

The objective in this study was to accumulate a group of statistically independent data points. Some previous correlations have used multiple liquefaction or non-liquefaction cases from a single site to generate more data for analysis. This method can be incorrect for two reasons. First, given a site with consistent stratigraphy and a uniform depositional environment, selecting two liquefied or two non-liquefied cases from the same critical layer results in cross-correlation of these two data points. This cross-correlation must be accounted for in any form of statistical analysis, and will result in much higher uncertainty or much reduced informational content for each data point. Second, if a particular layer within the site does liquefy, this modifies the incoming seismic energy for the layers above through seismic isolation, and below by blocking full reflection off the surface. This leads to a modified CSR for other layers at the site, which can be difficult to evaluate.

4.4 CRITICAL LAYER SELECTION

Selection of the critical layer is an important step in estimating the seismic strength of a particular soil deposit. The critical layer is the stratum of soil that constitutes the weakest link in the chain from a liquefaction perspective. Finding the weakest link requires observing the tip resistance and friction ratio in conjunction, with the addition of a SPT log for soil classification if one is available. For most depositional environments this can be a simple matter of looking for the smallest continuous stretch of tip resistance with low friction ratio that agrees with the SPT log in terms of a liquefiable material. This can be a difficult undertaking in fluvial depositional environments where the strata are thin, interbedded, and discontinuous both horizontally and vertically. A final criterion for identifying a critical layer is comparing the suspect layers to previous correlations. This aids determining which of multiple layers liquefied or did not liquefy in the more difficult sites.

An issue that is not commonly addressed in liquefaction correlations is that the *in situ* data are usually acquired post ground shaking. Particularly for the liquefied cases, the soil strength and properties have most likely been modified due to the process of liquefaction. Chameau et al. (1991) looked at sites that were affected by the Loma Prieta earthquake in which previous CPT data existed. Post-event CPT data were acquired and compared to the pre-event CPT data. Chameau et al. found that loose materials experienced the most alteration in tip resistance due to the ground shaking and subsequent liquefaction. This comes as no surprise. Recent work involving large-scale liquefaction blast tests have and are being performed in Japan where pre- and post-liquefaction CPT measurements are made. Hopefully this data will resolve the bias and allow for proper accounting of the changes that occur within a liquefied layer.

If it can be assumed that tip resistance has a positive correlation with relative density for clean sands (Schmertmann, 1978), then the greater the tip resistance the greater the relative density. In a critical-state framework, given a constant confining stress, the higher the relative density (lower the void ratio), the less capacity the soil has for contractive behavior. Liquefaction is premised on this contractive behavior of soils. Therefore, the closer a point lies to the limit-state or liquefaction boundary, the less contractive it is and the less pre- to post-liquefaction change in resistance it is likely to experience. On the non-liquefaction side of the limit-state or liquefaction boundary it is assumed that the resistance is unmodified by the ground shaking because no liquefaction has occurred. Another issue that arises is that if a CSR value is determined for a liquefied site using the post-liquefaction *in situ* measurements for site response analysis, the value may be slightly higher than pre-liquefaction conditions because of the stiffening that has occurred.

Given all these pre- and post-liquefaction considerations, it is conjectured that the limit-state function is unaffected by post-liquefaction densification because

1. near the limit state the liquefied soils are near the critical state (i.e., a small state parameter value) and therefore have not significantly densified due to liquefaction, and
2. non-liquefied soils will have no post-event densification and therefore are unaffected by the event and will maintain their position near the limit state.

The soils most affected by liquefaction, which will give vastly different post-event resistance measurements, are the loose or low tip resistance soils, and these have little impact on the limit-state function in a Bayesian-type analysis.

4.5 INDEX MEASUREMENTS

Once the critical layer has been selected it is a matter of determining the appropriate statistics of the measurements within the layer. Kulansingam, Boulanger, and Idriss (1999) studied various procedures for estimating an average tip resistance over a standardized distance of cone travel. They looked at different standardized distances and came to the conclusion that having a preset distance over which the resistance is averaged produced poor results.

The approach used in this study was to allow the depositional environment dictate. Using the procedures described above for identifying the critical layer, the maximum distance over which the soil deposit lies is often apparent. The top and bottom depths are taken as extrema. The averages and standard deviations are then calculated from a digitized form of the trace. Raw sleeve and tip measurements are used to calculate the friction ratio in order to eliminate aliasing that can occur in field calculations.

Induced pore pressure can have an effect on the tip and sleeve measurements. This effect is pronounced in soils that respond in an undrained manner to the strain imposed by the advancing cone (i.e., fine grained soils). For most soils susceptible to liquefaction, fully drained cone penetration is assumed (Lunne et al., 1997). Therefore, in general, no pore-pressure corrections are necessary for materials that are potentially liquefiable. This assumption of fully drained response was checked using pore-pressure measurements, when available, for each site.

4.6 MASKED LIQUEFACTION

In certain situations liquefaction occurs at depth but evidence may not reach the ground surface due to the monolithic or unified nature of overlying non-liquefiable strata. This masked liquefaction situation was researched and presented by Ishihara (1985) and reevaluated by Youd and Garris (1995). The results from that body of research are used to screen sites that are found to be liquefiable in terms of the index measurements that have overlying non-liquefiable material that fits the thickness criteria, that showed no surface manifestation of liquefaction, and that were reported as non-liquefied. For reference, at a site experiencing a low level of ground shaking (PGA < 0.2 g) with a 2-m-thick liquefiable layer, an overlying non-liquefiable layer of approximately m could eliminate all surface manifestation of liquefaction.

4.7 SCREENING FOR OTHER FAILURE MECHANISMS

Certain soil types are not susceptible to liquefaction but may deform via cyclic softening. These soils may exhibit surface manifestations that can appear quite similar to "classic" liquefaction cases, such as lateral spreading, and building tilting, punching, and settlement. However the failure mechanism is quite different from liquefaction. The soils that are susceptible to cyclic softening tend to have a high percentage of fines and these fines tend to fail in a plastic manner. Several cases of this nature were observed in the 2001 Kocaeli, Turkey, earthquake and the 2001 Chi-Chi, Taiwan, earthquake. Since the limit states and the overall correlations are based on "classic" liquefaction, it is not appropriate to include these cases in the analysis.

A criterion for screening these cases is based on research of fines content and plasticity in relation to liquefaction susceptibility (Andrews and Martin, 2000; Andrianopoulos et al., 2001; Guo and Prakash, 1999; Perlea, 2000; Polito, 2001; Sancio et al., 2003; Yamamuro and Lade, 1998, Youd and Gilstrap, 1999; to name a few). The criteria for soils not susceptible to liquefaction used in this study are shown graphically in Figure 4.1.

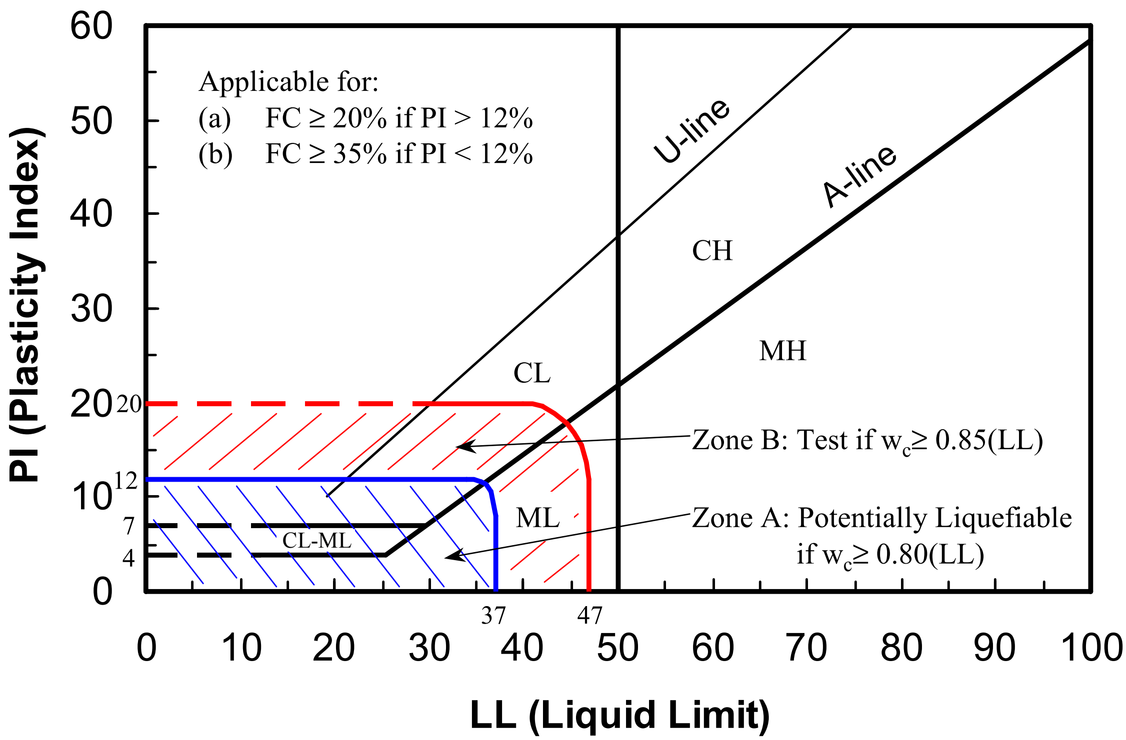

Figure 4.1 Screening criteria for failure mechanism other than liquefaction

4.8 NORMALIZATION

Effective overburden stress can have a significant influence on measured tip and sleeve resistances of the cone penetration test (CPT). Cohesive soils respond to confining stress primarily as a function of the overconsolidation ratio (OCR) and undrained strength (s_u). Cohesionless soils respond to confining stress primarily as a function of relative density (D_r) and the coefficient of lateral earth pressure (K_o), and, to a lesser degree, as a function of the angularity, compressibility, and crushing strength of the grains.

These effects due to overburden stress are nonlinear, showing a curve-linear decrease with linear increase in stress. To account for the effects of confining stress, the tip and sleeve resistance values are normalized to a reference stress value of one atmosphere (1 atm = 101.325 kPa = 1.033 kg/cm^2 = 14.696 psi = 1.058 tsf).

For accurate tip and sleeve resistances it is essential to normalize these index measurements appropriately. A comprehensive study was carried out to review all aspects of CPT normalization, and to solidify normalization procedures for the CPT using both empirical results and theoretical analyses. The end product was an improved normalization scheme for the CPT.

4.8.1 Previous Research

The bulk of research on CPT normalization was conducted by Olsen et al. (1988, 1994, 1995a, and 1995b). Olsen (1994) utilized a technique of defining the normalization for tip and sleeve resistances of various soil types from field and laboratory data. For a given "uniform" soil strata the resistance was measured at different confining stresses. The results were plotted as a function of confining stress in log-log space, resulting in a linear relationship. The stress normalization exponent for that particular soil state is then the slope of the linear fit in log-log space (with the symbol c for tip exponent and s for sleeve exponent). This procedure was carried out for soil types where reasonable data existed, which led to the Olsen and Mitchell, 1995, normalization exponent contours. These exponent contours can then be used in a forward analysis to normalize the tip and sleeve resistances as

$$q_{c,1} = C_q \cdot q_c \text{ and } f_{s,1} = C_f \cdot f_s \tag{4.1}$$

where $C_q = \left(\frac{P_a}{\sigma_v'}\right)^c$ and $C_f = \left(\frac{P_a}{\sigma_v'}\right)^s$

This work incorporated over two decades of field data and an extensive database of chamber test studies to deduce the tip normalization exponent for a number of different soil types. Olsen (1994) laid down the groundwork for cone normalization, and subsequent researchers (e.g., Robertson and Wride, 1998) deferred to this body of work when addressing normalization.

An inherent limitation to the empirical approach is that a layer must be uniform and stretch over a sufficient depth to be of use. Normalization data in granular materials are generally restricted to chamber test results because of the inherent variability in the field due to this type of depositional environment. In fine-grained soils, normalization data are generally restricted to field tests because of the difficulty of performing chamber studies on this type of soil. For soils that fall outside the requirements of uniformity and extent, it is difficult if not impossible to generate or retrieve normalization data for analysis.

4.8.2 Theoretical Foundation for Normalization

To expand on Olsen's work a new approach was taken. This approach was to look at a theoretical foundation for CPT normalization. A literature review of methods that theoretically predict CPT measurements from fundamental soil properties was carried out. Many methods have been proposed, including bearing capacity, cavity expansion, strain path, steady state, incremental finite element, and discrete element.

Based on the literature (Mayne, 1991; Keaveny, 1985, Keaveny and Mitchell, 1986; Yu and Houlsby, 1991; Salgado, 1993; Collins et al., 1994; Huang and Ma, 1994; Salgado et al., 1997; Yu and Mitchell, 1998; Yu, 2000) cavity expansion methods are the most advanced for theoretically predicting CPT tip resistance. Yu and Mitchell (1998), in particular, looked at all theoretical methods that were functionally comparative at the time and found cavity expansion to be the most developed, as well as providing the greatest accuracy in CPT predictions over all stress ranges. Bearing capacity methods are only valid for shallow or low confining stress regimes, and provide a linear approximation to a nonlinear problem. Other methods such as steady state, discrete element, strain path, and incremental finite element are promising methods but are in their infancy and have only been developed to predict CPT tip resistance for a specific soil type and stress condition. Steady state methods were used in this study as qualitative support for the quantitative cavity expansion results.

4.8.3 Cavity Expansion Analysis

Bishop et al. (1945) was the first to note the analogy between the expansion of a cavity and the penetration of a cone in an elastic medium. Subsequent researchers developed this further by incorporating higher-order stress-strain relationships to model sands and clays with increasing rigor and accuracy (Vesic, 1972; Ladanyi and Johnston, 1974; Baligh, 1976; Carter et al., 1986; Yu and Houlsby, 1991; Collins et al., 1992; Salgado et al., 1997).

Cavity expansion methods require two steps: (1) a theoretical (analytical or numerical) cavity limit pressure solution is calculated and (2) this limit pressure is then related to the cone tip resistance. This study utilized various cavity expansion solutions to determine normalization exponents. Because of the complexity of soil behavior and the different solutions required for different types of soil behavior, the discussion of theoretical methods is divided into four soil state categories: cohesive normally consolidated, cohesive overconsolidated, cohesionless contractive, and cohesionless dilatant. This report contains a brief description of the methods and models used: full details can be found in Moss (2003). The cavity expansion models employed were those of the following researchers:

- Yu and Houlsby (1991) derived an analytical solution for a total stress cylindrical cavity expansion model in normally consolidated cohesive clay. The soil is modeled as a linear elastic-perfectly-plastic material using a Mohr-Coulomb yield criterion. The closed-form solution for a standard 60◦ cone was used.
- Chang et al. (2001) and Cao et al. (2001) published companion papers that developed a closed-form modified Cam clay cavity expansion model that can be used to predict tip resistance for overconsolidated cohesive soils. These papers were bolstered by discussions from Ladanyi (2002) and Mayne et al. (2002).
- Ladanyi and Johnston (1974) derived an analytical solution for tip resistance in contractive sands using a spherical cavity approach and a linear elastic-plastic von Mises failure criterion. A numerical solution for the spherical cavity limit pressure is needed for this analytical solution, which was developed by Yu (2001) and implemented in the code CAVEXP.
- Salgado (1993) developed a nonlinear elastic-plastic cavity expansion model that accounts for dilatant behavior in cohesionless material. This model requires a finite element solution for the cavity limit pressure, which has been implemented in the code CONPOINT (Salgado et al., 1997 and 2001). Accounting for this soil state, Salgado's model first numerically calculates the cylindrical cavity limit pressure, then uses a stress rotation analysis to obtain the tip resistance.
- Boulanger (2003) used Salgado's model as a theoretical basis to calculate normalization exponents for dilatant cohesionless materials subjected to high confining stresses (σ_v'>4 atm) and cyclic loads.

The results from the cavity expansion analyses are presented in Figure 4.2, a plot of the calculated tip normalization exponents over $q_{c,1}$ and R_f ranges. The model results were generated for an effective stress range of 0.5 to 3.0 atm, with the exception of Boulanger's (2003) model that was derived for effective stress values higher than 4.0 atm.

Contours of variable tip normalization exponents were developed using the cavity expansion results as well as the existing field and calibration chamber test data from Olsen (1994). The resulting contours are shown in Figure 4.3 compared with the contours from Olsen and Mitchell, 1995. The theoretical results led to the adjustment of the previous normalization contours in key areas. In particular, for this liquefaction study, the region of contractive sands was modified to closer reflect the cavity expansion results. Figure 4.4 shows the proposed normalization contours.

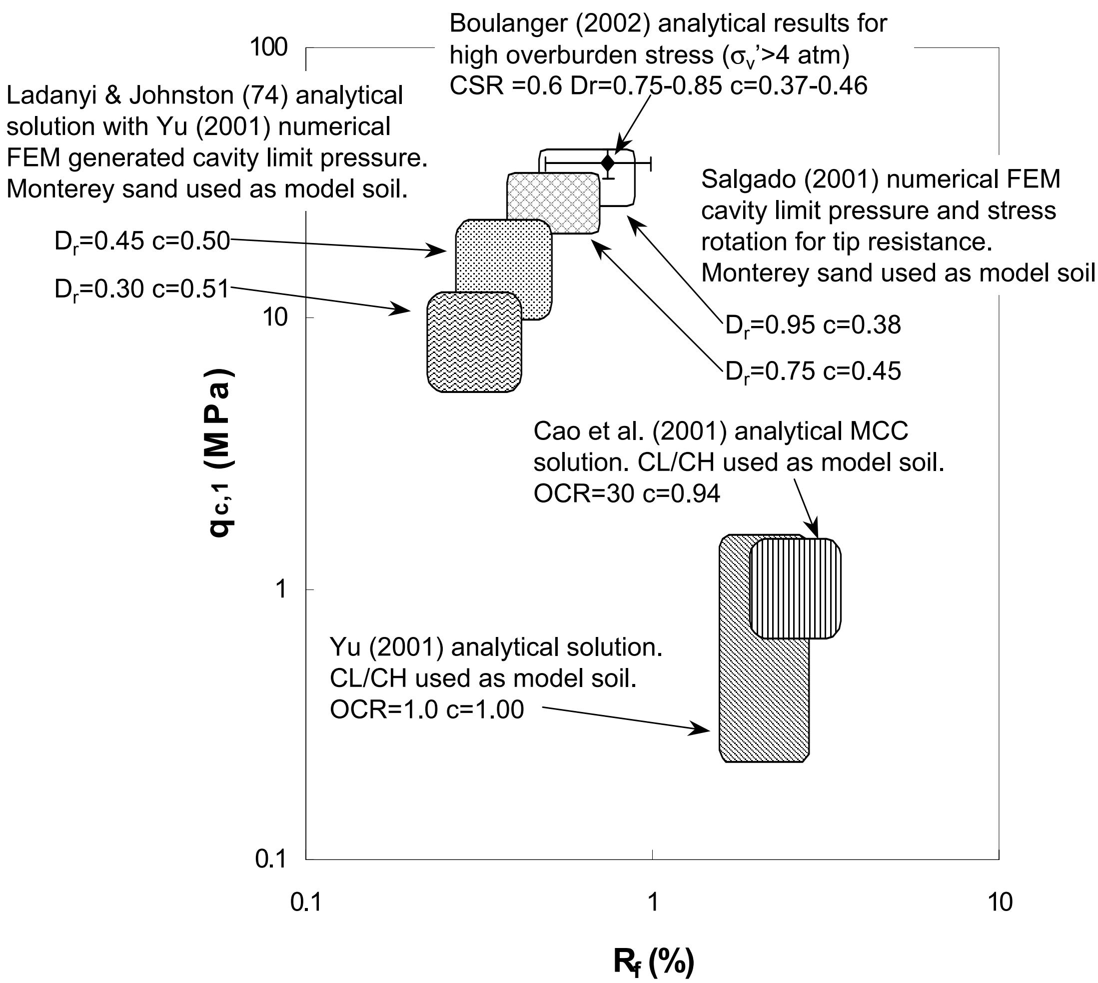

Figure 4.2 Tip normalization exponent results from cavity expansion analyses

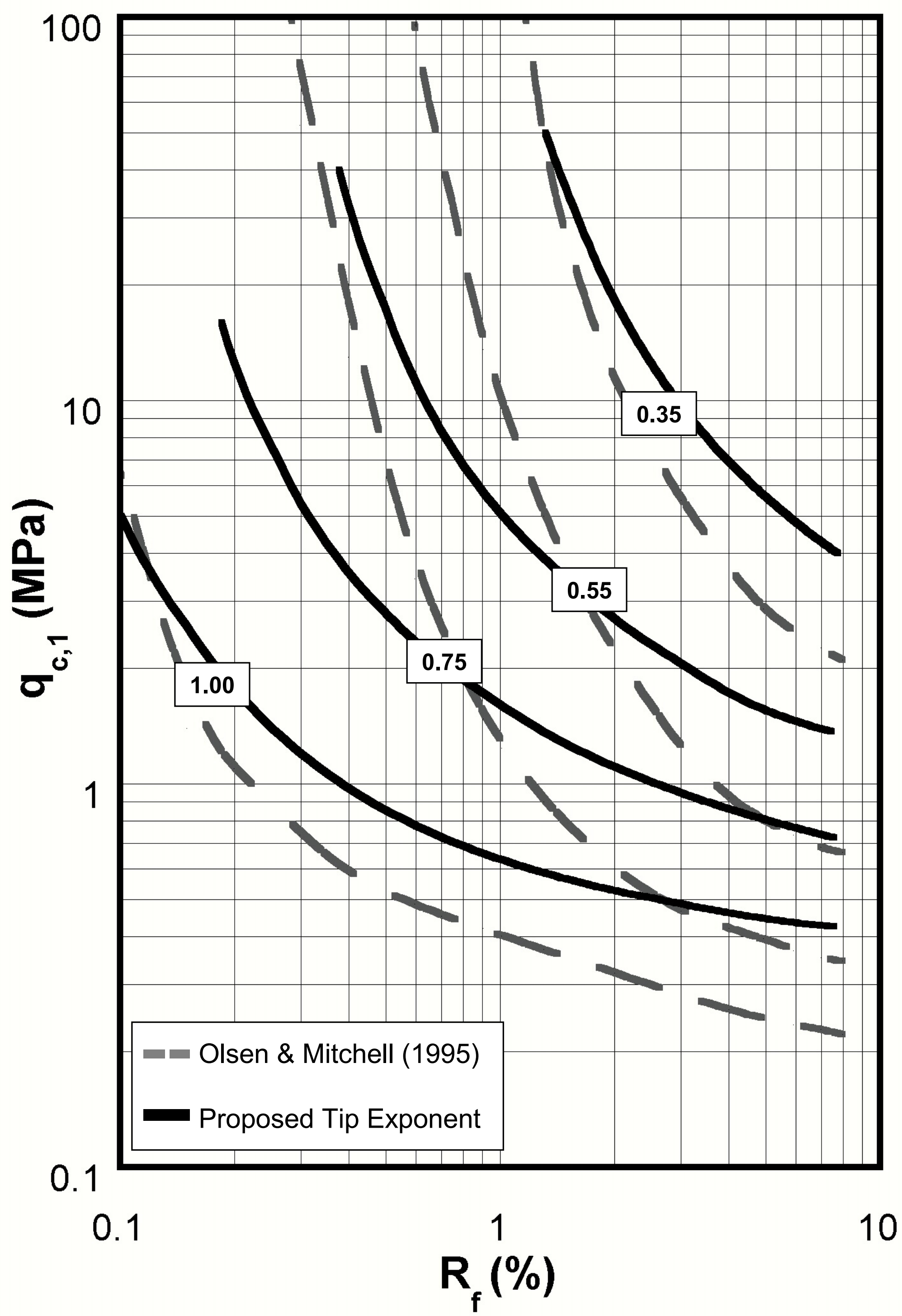

Figure 4.3 Comparison of proposed tip normalization exponent contours with Olsen and Mitchell (1995) tip normalization contours

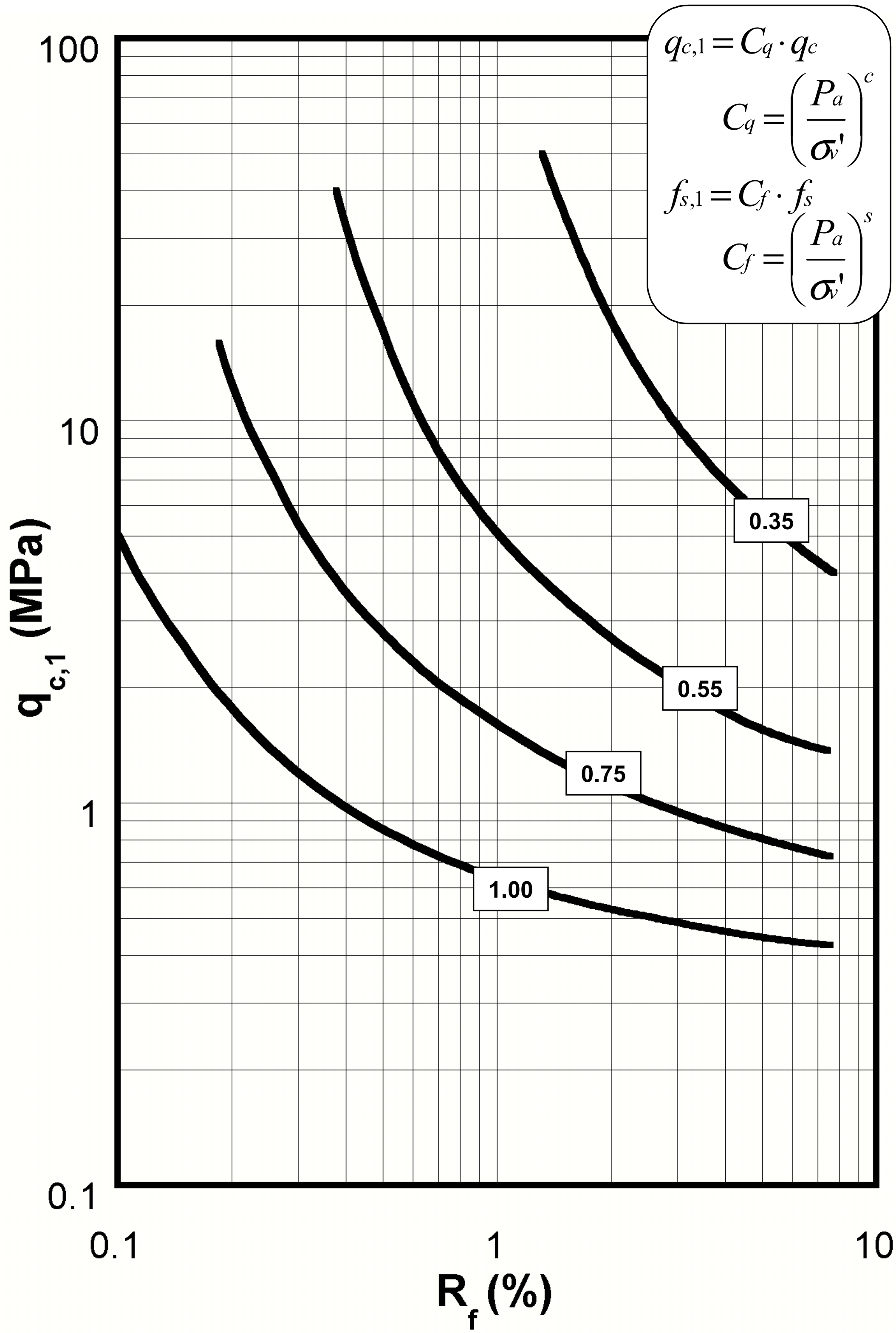

Figure 4.4 Proposed tip normalization exponent contours

4.8.4 Application of Normalization

To normalize the tip resistance appropriately, an iterative procedure is necessary. The iterative procedure involves the following steps:

1. An initial estimate of the normalization exponent is found using raw tip measurements, friction ratio, and Figure 4.4;
2. The tip is then normalized using Equation 4.1 (note: friction ratio will not change when tip and sleeve are normalized equivalently);
3. A revised estimate of the normalization exponent is found using the normalized tip resistance and Figure 4.4, which is compared to the initial normalization exponent estimate; and
4. The procedure is repeated until an acceptable convergence tolerance is achieved.

For most soils this process usually requires only two iterations to converge. It is recommended that the tip and sleeve be normalized equivalently. To aid in computation, an approximation of the normalization exponent curves can be represented as a single equation:

$$c = f_1 \cdot \left(\frac{R_f}{f_3} \right)^{f_2} \tag{4.2}$$

where $f_1 = x_1 \cdot q_c^{x_2}$

$f_2 = -(y_1 \cdot q_c^{y_2} + y_3)$

$f_3 = abs(\log(10 + qc))^{z_1}$

and $x_1 = 0.78, x_2 = -0.33, y_1 = -0.32, y_2 = -0.35, y_3 = 0.49, z_1 = 1.21$

This equation gives a good approximation of the tip normalization contours and can be used instead of Figure 4.4. [In Excel, the Solver Add-In in the Analysis Toolpack can be useful for this iterative procedure in spreadsheet calculations.]

4.9 THIN LAYER CORRECTION

The CPT measurement at a particular point in a highly stratified soil column represents the resistance at the tip with respect to the layers above and below the tip. This is analogous to the cone tip "sensing" ahead and behind the current location in the soil column. Depending on the thickness of the layer at the cone tip, the measured resistance value can be significantly different from the true resistance value of the stratum if it were a continuous thick layer.

Vreugdenhil et al. (1994) used a simplified elastic solution to analytically quantify the difference between the measured resistance values in the layered media versus a true resistance value for the layer if it were thick. The concept of an elastic solution appears contrary to the high strain that occurs when a cone punches through the soil. However, the elastic solution does not need to model the tip resistance *per se*, but the effect of a layer of soil at a distance, and the effect that this layer has on the measured resistance. At a distance, the effect of the cone on the soil can be assumed to be in the elastic range.

Robertson and Fear (1995) recommended corrections for a stiff thin layer based on an interpretation of Vreugdenhil et al. (1994). They modified the Vreugdenhil et al. results and suggested a correction curve for a tip resistance ratio of two ($q_{cB}/q_{cA}=2$). NCEER (Youd et al., 1997) workshop proceedings suggested a correction range for a tip resistance ratio of two ($q_{cB}/q_{cA}=2$) based on field data from Gonzalo Castro and Peter Robertson. There exists a discrepancy between the two recommendations. This current study attempts to reconcile these differences between the Robertson and Fear recommendations and the NCEER recommendations, and present consistent thin layer correction recommendations.

The elastic solution presented by Vreugdenhil et al. (1994) was compared with chamber tests studies of layered soil profiles by Kurup et al. (1994). In this verification the average relative tip resistance (q_c) values for the soil layers were used as a proxy for the elastic stiffness moduli (G). This is a reasonable assumption if the cone is pushed at a continuous rate through the different types of soil (constant strain rate) and the stiffness ratio (G_B/G_A) between two different soil types is not wildly disparate (i.e., the relative response to strain is similar in the two soils). Two different scenarios were considered, a thin layer of softer material and a thin layer of stiffer material.

Analytical results from the first model (embedded soft thin layer) showed that there was little alteration of the measured tip resistance. The soft thin layer appears to isolate the cone from the surrounding stiffer material. The entry and exit zones of altered resistance were on the order of 3 to 5 cone diameters for a 20% change in resistance, where the stiffness ratio between the thin layer and the surrounding material is high.

The results from the second model (embedded stiff thin layer), as shown in Figure 4.5, indicated that the alteration of measured resistance can be high, on the order of 100 to 200 cone diameters for a 20% change in resistance, with a high stiffness ratio. In this instance the difference in soil stiffness can have a large effect on the measured resistance at a great distance from the cone tip. This can lead to difficulties in determining the true resistance of the thin stiff layer, and in interpreting the depth at which the stratum originates and terminates.

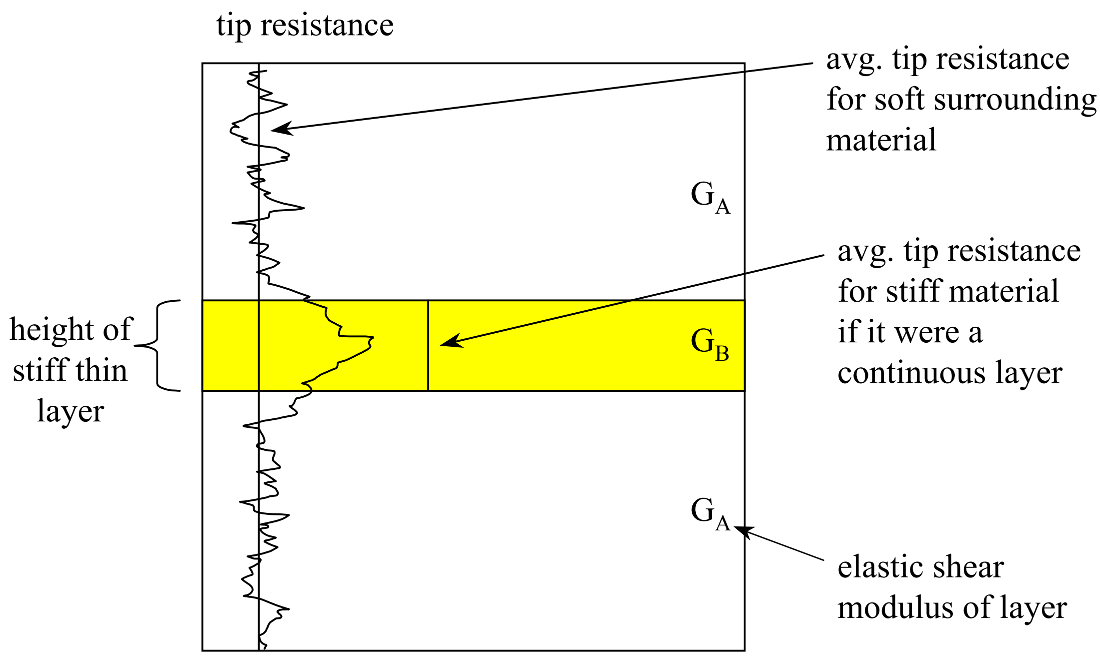

Figure 4.5 Conceptual model of stratigraphic sequence with stiff thin layer

In this current study we employed the original research by Vreugdenhil et al. to generate correction curves for tip resistance ratios of two, five, and ten (q_{cB}/q_{cA}=2, 5, and 10). Field data were used to corroborate the location and range of the correction curves. The field data were from sites with two relatively uniform layers in sequence where the mean tip resistances could be clearly defined at a certain distance away from the layer interface. The difference in stiffness between the two layers gives rise to an altered measured tip resistance; it appears as a warping of the tip resistance over a finite distance. This distance corresponds to a thin layer correction of 1.0; in other words, no correction is necessary in a thin layer scenario at this resistance ratio with a layer thickness of this value. The correction factors were then determined by decreasing the layer thickness to achieve factors of greater than 1.0. The empirical results agreed favorably with the theoretical results with regard to general trends, but the correction factors were found to be smaller at high stiffness ratios. There is high confidence in the resistance ratios of two and five. The data for the resistance ratio of ten are slightly suspect because of the difficulty of interpreting field data with this resistance ratio; it is difficult to discern when the cone is reading an altered resistance due to layer interference or when the cone is reading an artifact of the geologic depositional environment.

Data from 23 different sites were used to determine the case specific correction factors. These were then collected into "bins" for layer stiffness ratios of q_{cB}/q_{cA}=1.0 to 3.5, 3.6 to 7.5, and 7.6 to 15.0, and these were compared against correction factors corresponding to the theoretical curves calculated from the elastic solution.

Based on the elastic solution of Vreugdenhil et al. (1994), the NCEER (1997) recommendations, and field data, new thin layer correction curves are recommended as shown in Figure 4.6. Curves are suggested for tip resistance ratios of two and five, with the recommendations for a ratio of ten as the upper bound. The curves encompass correction factors up to a recommended limit of 1.8. These results are based on a standard cone of diameter 35.7 mm (cone tip area 10 cm^2). Note that only 4% of the cases in the liquefaction database required a thin layer correction. For database purposes the thin layer correction was limited to a maximum of 1.5 ($C_{thin} \leq 1.5$).

Equation 4.3 approximates the thin layer correction curves. This equation is valid for a stiffness ratio of less than or equal to 5 ($q_{cB}/q_{cA} \leq 5$). For higher stiffness ratios careful analysis and engineering judgment are required and it is recommended that the thin layer correction values be estimated by hand.

$$C_{thin} = A \cdot (\text{layer thickness})^{B} \tag{4.3}$$

where $A = 3.744 \cdot (q_{cB}/q_{cA})^{0.491}$

$$B = -0.050 \cdot \ln(q_{cB}/q_{cA}) - 0.204$$

$$q_{cB}/q_{cA} = \text{stiffness ratio}$$

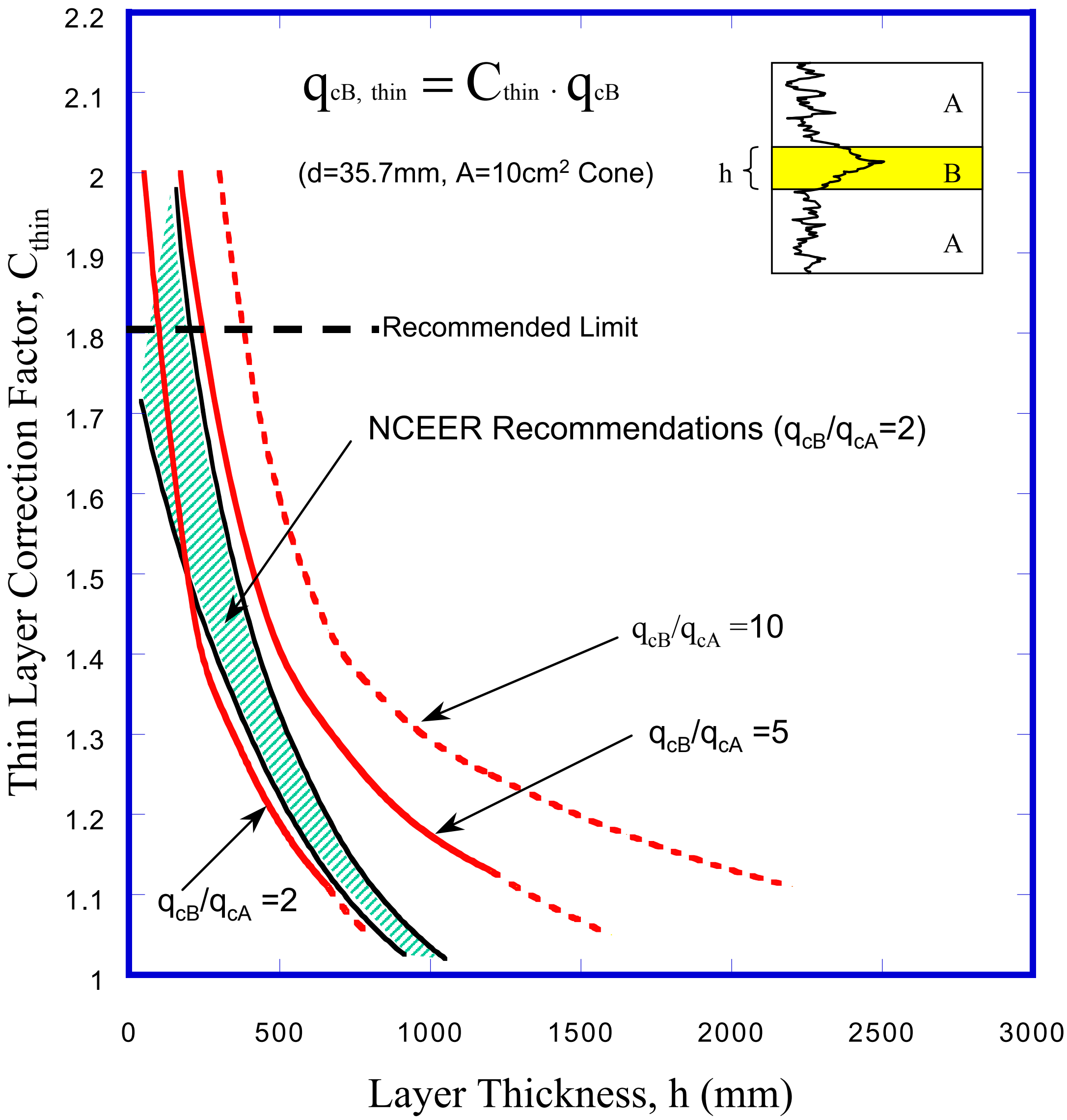

Figure 4.6 Proposed correction curves for stiff thin layer

4.10 CYCLIC STRESS RATIO

The dynamic stress that a critical layer experienced is determined using the simplified uniform cyclic stress ratio as defined by Seed and Idriss (1971):

$$CSR = 0.65 \cdot \frac{a_{\max}}{g} \cdot \frac{\sigma_v}{\sigma'_v} \cdot r_d \tag{4.4}$$

The CSR value calculated using Equation 4.4 is assumed to be the average or sample mean as in Equation 4.5. The variance of CSR is calculated via equation 4.6, where the coefficient of variation is equal to the standard deviation divided by the mean. Both Equation 4.5 and 4.6 are using first-order Taylor series expansions about the mean point, including only the first two terms.

$$\mu_{CSR} \cong 0.65 \cdot \frac{\mu_{a\max}}{g} \cdot \frac{\mu_{\sigma v}}{\mu_{\sigma' v}} \cdot \mu_{rd} \tag{4.5}$$

$$\delta_{CSR}{}^2 \cong \delta_{a\max}{}^2 + \delta_{rd}{}^2 + \delta_{\sigma v}{}^2 + \delta_{\sigma' v}{}^2 - 2 \cdot \rho_{\sigma v \sigma' v} \cdot \delta_{\sigma v} \cdot \delta_{\sigma' v} \tag{4.6}$$

Total and effective stress are correlated parameters; therefore the inclusion of the correlation coefficient term for these two variables is necessary.

4.11 PEAK GROUND ACCELERATION

The geometric mean of the peak ground acceleration is based on the best estimation of ground shaking possible. The methods of estimation are; strong motion recordings, site response, calibrated attenuation relationships, adjustment of estimated site PGA through general site response modeling, and general attenuation relationships. A calibrated attenuation relationship involves using all available recordings to tune general attenuation relationships for event-specific variations and azimuth specifics where recordings permit.

The coefficient of variation of the peak ground acceleration is fixed according to the method of ground shaking estimation:

- $\delta < 0.10$ for sites with strong motion stations less than 10 m from site,

- δ = 0.10 to 0.25 for sites with strong motion stations within 100 to 50 m from site or where site response analysis was performed using a nearby rock recording as input base motion,
- δ = 0.25 to 0.35 for sites with strong motion stations within 50 m to 100 m and/or estimates from calibrated attenuation relationships, and
- δ = 0.35 to 0.5 for others.

This is a subjective determination of the variance of the ground shaking but is based on typical uncertainty bands from general attenuation relationships that have coefficient of variations of between 0.3 and 0.5 (e.g., Abrahamson and Silva, 1997).

4.12 TOTAL AND EFFECTIVE STRESS

The total and effective vertical stresses are correlated variables and this correlation must be taken into account. The critical layer is selected using the procedures outlined above. From this the total extent of the critical layer is used to calculate the mean and variance of the critical layer. The variance is estimated using a 6 sigma approach, where the extrema of the layer are assumed to be three standard deviations away from the mean on either side. The total variance is then divided by six to give an estimate of the standard deviation.

A deterministic estimate is made of the mean unit weight of the soil above and below the water table. The variance is based on statistical studies of the measured variability of soil unit weight and is set at $\delta \cong 0.1$ (Kulhawy and Trautman, 1996). The mean water table elevation is taken as the reported field measurement (with consideration given for the depth of water table during the seismic event), with a fixed standard deviation of σ = 0.3 m., a reasonable estimate of water table fluctuations given relatively stable groundwater conditions. An estimate of the total and effective vertical stresses, their respective variances, and covariance can then be calculated using the expansion Equations 4.7–4.12:

$$\mu_{\sigma v} \cong \mu_{\gamma 1} \cdot \mu_{h_w} + \mu_{\gamma 2} \cdot (\mu_h - \mu_{hw}) \tag{4.7}$$

$$\mu_{\sigma v'} \cong \mu_{\gamma 1} \cdot \mu_{hw} + (\mu_{\gamma 2} - \gamma_w) \cdot (\mu_h - \mu_{hw}) \tag{4.8}$$

$$\sigma_{\sigma_v}^2 \cong \mu_{hw}^2 \cdot \sigma_{\gamma 1}^2 + (\mu_h - \mu_{hw})^2 \cdot \sigma_{\gamma 2}^2 + \mu_{\gamma 2}^2 \cdot \sigma_h^2 + (\mu_{\gamma 1} - \mu_{\gamma 2})^2 \cdot \sigma_{hw}^2 \tag{4.9}$$

$$\sigma_{\sigma_v'}^2 \cong \mu_{hw}^2 \cdot \sigma_{\gamma 1}^2 + (\mu_h - \mu_{hw})^2 \cdot \sigma_{\gamma 2}^2 + (\mu_{\gamma 2} - \gamma_w)^2 \cdot \sigma_h^2 + (\mu_{\gamma 1} + \gamma_w - \mu_{\gamma 2})^2 \cdot \sigma_{hw}^2 \tag{4.10}$$

$$Cov[\sigma_v, \sigma_v'] \cong (\mu_{hw}^2 \cdot \sigma_{\gamma 1}^2) + (\mu_{\gamma 1} - \mu_{\gamma 2}) \cdot (\mu_{\gamma 1} + \gamma_w - \mu_{\gamma 2}) \cdot \sigma_{hw}^2 + (\mu_h - \mu_{hw})^2 \cdot \sigma_{\gamma 2}^2 + \mu_{\gamma 2} \cdot (\mu_{\gamma 2} - \gamma_w) \cdot \sigma_h^2 \tag{4.11}$$

$$\rho_{\sigma_v \sigma_v'} = \frac{Cov[\sigma_v, \sigma_v']}{Var[\sigma_v] \cdot Var[\sigma_v']} \tag{4.12}$$

4.13 NONLINEAR SHEAR MASS PARTICIPATION FACTOR (R_D)

The nonlinear shear mass participation factor (r_d) accounts for nonlinear ground response in the soil column overlying the depth of interest. This factor, denoted as r_d, has been derived from ground response analyses. In recent work, 2,153 site response analyses were run using 50 sites and 42 ground motions covering a comprehensive suite of motions and soil profiles (Cetin and Seed, 2000; Seed et al., 2003a). This "brute force" approach allows for statistical analysis of the median response given the depth, peak ground acceleration, moment magnitude, and 30-m shear wave velocity of the site. The variance was estimated from the dispersion of these simulations. The median values can be calculated using Equations 4.13–4.14, and the variance from Equations 4.15–4.16:

For $d < 20$ meters,

$$r_d(d, M_w, a_{\max}) = \frac{\left[1 + \dfrac{-9.147 - 4.173 \cdot a_{\max} + 0.652 \cdot M_w}{10.567 + 0.089 \cdot e^{0.089 \cdot (-d \cdot 3.28 - 7.760 \cdot a_{\max} + 78.576)}}\right]}{\left[1 + \dfrac{-9.147 - 4.173 \cdot a_{\max} + 0.652 \cdot M_w}{10.567 + 0.089 \cdot e^{0.089 \cdot (-7.760 \cdot a_{\max} + 78.576)}}\right]} \tag{4.13}$$

and for $d \geq 20$ meters,

$$r_d(d, M_w, a_{\max}) = \frac{\left[1 + \dfrac{-9.147 - 4.173 \cdot a_{\max} + 0.652 \cdot M_w}{10.567 + 0.089 \cdot e^{0.089 \cdot (-d \cdot 3.28 - 7.760 \cdot a_{\max} + 78.567)}}\right]}{\left[1 + \dfrac{-9.147 - 4.173 \cdot a_{\max} + 0.652 \cdot M_w}{10.567 + 0.089 \cdot e^{0.089 \cdot (-7.760 \cdot a_{\max} + 78.567)}}\right]} - 0.0014 \cdot (d \cdot 3.28 - 65) \qquad (4.14)$$

where d is depth in meters at the midpoint of the critical layer, M_w is moment magnitude, and a_{max} is peak ground acceleration in units of gravity. The standard deviation for r_d is

For $d < 12.2$ m,

$$\sigma_{r_d}(d) = (d \cdot 3.28)^{0.864} \cdot 0.00814 \qquad (4.15)$$

and for $d \geq 12.2$ m

$$\sigma_{r_d}(\mathrm{d}) = 40^{0.864} \cdot 0.00814 \qquad (4.16)$$

4.14 MOMENT MAGNITUDE

Moment magnitude is a value that is usually reported by seismology laboratories following an event, and iterated on for a week or two until the final value is posted. Calculating the moment magnitude involves an inverse problem to determine the seismic moment. The uncertainty in these calculations comes from the non-unique inversion based on seismograms that are recorded at various teleseismic stations. The dimensions of the fault plane and the amount of slip associated with larger magnitude events tend to be easier to define than with smaller magnitude events. Also smaller events will have fewer recordings leading to a smaller sample size and more uncertainty. A simple equation (Eq. 4.17), based on the variance of a series of previous events (1989 Loma Prieta, 1994 Northridge,1999 Tehuacan, 1999 Kocaeli, 1999 Taiwan, 2001 Denali), was used to roughly estimate this epistemic uncertainty:

$$\sigma_{M_w} \cong 0.5 - 0.45 \cdot \log(M_w) \qquad (4.17)$$

4.15 DURATION WEIGHTING FACTOR (AKA MAGNITUDE SCALING FACTOR)

All results presented in this study are corrected for duration (or number of equivalent cycles) to an "equivalent uniform cyclic stress ratio" CSR*, representing the equivalent CSR for a duration typical of an "average" event of $M_W = 7.5$. This was done by means of a magnitude-correlated duration weighting factor (DWF_M) as

$$CSR^{*} = \frac{CSR}{DWF_{M_w}} \tag{4.18}$$

This duration weighting factor is somewhat controversial, and has previously been developed using a variety of different approaches (using cyclic laboratory testing and/or field case history data) by a number of investigators. Figure 4.7 summarizes some of these studies and shows (shaded zone) the recommendations of the NCEER Working Group (Youd et al., 2001). The study using SPT data (Cetin, 2000; Seed et al., 2003b), regressed the DWF_M from the database that included a number of events covering a wide spectrum of moment magnitudes. The current study using CPT data was lacking in a wide enough spectrum to discern accurately the DWF_M in a similar manner. Based on good agreement of the SPT work with previously published results, the recommended DWF_M from Cetin (2000) and Seed et al. (2003b) was used. The recommendation can be represented by the equation:

$$DWF_M = 17.84 \cdot M_w^{-1.43} \tag{4.19}$$

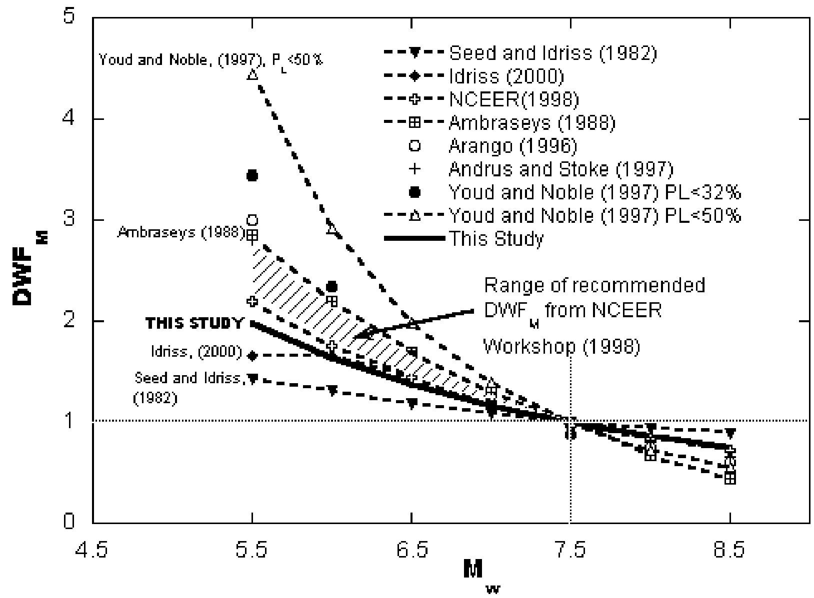

Figure 4.7 Comparison of different DWF_M studies (from Cetin, 2000)

4.16 DATA CLASS

After the case histories were selected and processed they were classified according to the quality of the informational content. Four classes are used to group the data, A–D, with D being substandard and therefore not included in the final database. The criteria for the data classes are as follows:

Class A

Original CPT trace with q_c and f_s/R_f, using a ASTM D3441 and D5778 spec. cone.

No thin layer correction required

$\delta_{CSR} \leq 0.20$

Class B

Original CPT trace with q_c and f_s/R_f, using a ASTM D3441 and D5778 spec. cone.

Thin layer correction.

$0.20 < \delta_{CSR} \leq 0.35$

Class C

Original CPT trace with q_c and f_s/R_f, but using a non-standard cone (e.g., Chinese cone or mechanical cone).

No sleeve data but FC ≤ 5% (i.e., "clean" sand).

$0.35 < \delta_{CSR} \leq 0.50$

Class D

Not satisfying the criteria for Classes A, B, or C.

4.17 REVIEW PROCESS

The final step in processing the data was an extensive review procedure. Each case in the database was reviewed a minimum of three times. A panel of qualified experts was assembled to do the review, this included in addition to the first author and Professors Raymond B. Seed; Jon Stewart, Les Youd, Kohji Tokimatsu, and Dr. Rob Kayen. Each case was reviewed by the first author, by Ray Seed, and at least one of the four independent reviewers. The objective was to remove as much human error and epistemic error from the database as possible.

A final note on the review process includes the review of the analytical and statistical procedures. The application of Bayesian analysis to SPT-based liquefaction-triggering correlations and the techniques used were reviewed extensively by the Pacific Earthquake Engineering Research Center (PEER), and by peer review of the following journals the *Journal of Geotechnical and Geoenvironmental Engineering* (Seed et al., 2003b) and the *Journal of Structural Safety* (Cetin et al., 2002). The CPT-based liquefaction-triggering correlation and the associated Bayesian analysis and methodology were also reviewed extensively at PEER's quarterly meetings by panelists Professors Les Youd, Geoff Martin, and I.M. Idriss.

It is the first author's belief that the power of the Bayesian framework in an engineering application is to incorporate all forms of information and that the review process is one of the more important and congenial steps in reducing epistemic uncertainty.

4.18 DATABASE

This CPT-based liquefaction field case history database consists of sites conforming to data classes A, B, and C, which were processed according to the techniques outlined in prior sections of this report. This database contains sites from 18 different earthquakes around the world that occurred from 1964–1999. This comprises the most extensive collection of field case history data for CPT-based liquefaction correlations to date.

More than 500 cases were studied, and 188 conforming to data classes A, B, and C were selected for use in the development of the new correlations. Cases of high uncertainty and cases with other significant potential deficiencies were deleted from further consideration. Table 4.1 presents the key variables for the 188 cases carried forward. Fuller descriptions of each case are presented in Moss (2003) and Moss et al. (2003c).

The data are arranged in chronological order with all pertinent variables included. The uncertainty of each parameter is included as a ±1 standard deviation. The mean water table measurements are shown; not shown is the uncertainty of the water tables which was assumed to be 0.3 m for all sites. Sites are described as liquefied or non-liquefied. The normalization exponent is shown in the column labeled c; this variable was treated deterministically and therefore no uncertainty is given.

Table 4.1 CPT-based liquefaction-triggering database

Earthquake	M_w	References:										
1964 Niigata, Japan	7.50±0.11	Farrar (1990), Ishihara & Koga (1981)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
Site D	Yes	B	2.7-6.0	1.12	47.94±10.56	32.44±4.16	0.16±0.03	0.95±0.05	0.15±0.05	0.45	6.24±1.73	1.14±0.65
Site E	Yes	B	1.8-4.8	0.67	68.00±12.82	44.46±4.94	0.16±0.03	0.92±0.07	0.15±0.04	0.47	4.56±1.13	1.22±0.60
Site F	No	B	1.7-2.2	1.70	31.95±2.13	29.50±2.38	0.16±0.03	0.97±0.04	0.11±0.02	0.38	9.39±8.97	1.40±1.81
Earthquake	M_w	References:										
1968 Inangahua, New Zealand	7.40±0.11	Ooi (1987), Dowrick & Sritharan (1968), Zhao et al. (1997)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
Three Channel Flat	Yes	C	0.5-2.5	0.10	29.00±6.60	15.27±3.37	0.40±0.10	0.97±0.03	0.48±0.19	0.53	2.84±0.96	1.39±0.70
Reedy's Farm	Yes	B	1.0-1.8	0.10	26.66±2.68	14.10±2.51	0.20±0.05	0.98±0.03	0.24±0.08	0.65	2.62±0.69	0.79±0.52
Earthquake	M_w	References:										
1975 Haicheng, China	7.30±0.11	EarthTech (1985), Arulanandan et al. (1986), Shengcong & Tatsuaoka (1984)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
Chemical Fiber Site	Yes	C	7.8-12.0	1.52	179.35±14.57	97.14±7.28	0.15±0.05	0.71±0.16	0.13±0.06	0.85	1.37±0.64	0.76±0.43
Const. Com. Building	Yes	C	5.5-7.5	1.52	116.45±6.81	67.60±4.94	0.15±0.05	0.83±0.11	0.14±0.05	0.92	0.77±0.14	1.37±0.27
Guest House	Yes	C	8.0-9.5	1.52	158.08±6.05	87.15±5.42	0.15±0.05	0.75±0.15	0.13±0.05	0.86	0.97±0.18	1.08±0.41
17th Middle School	Yes	C	4.5-11.0	1.52	136.46±19.79	75.34±8.40	0.15±0.05	0.79±0.13	0.14±0.06	0.87	0.92±0.29	1.02±0.44
Paper Mill	Yes	C	3.0-5.0	1.52	70.20±6.46	45.87±4.44	0.15±0.05	0.91±0.08	0.14±0.05	0.77	1.16±0.31	1.28±0.56
Earthquake	M_w	References:										
1976 Tangshan, China	8.00±0.09	[1] Arulanandan et al. (1982); [2] Zhou & Zhang (1979), Shibata & Teparaska (1988)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
Tientsin Y21 [1]	Yes	C	4.5-5.25	1.00	89.63±3.45	51.61±4.02	0.08±0.03	0.91±0.09	0.09±0.04	0.76	0.97±0.42	2.50±1.84
Tientsin Y24 [1]	Yes	C	3.5-4.5	0.20	75.40±4.09	38.12±3.34	0.09±0.04	0.93±0.08	0.11±0.05	0.70	3.64±0.632	0.72±0.15
Tientsin Y28 [1]	Yes	C	1.0-3.0	0.20	37.40±6.50	19.74±3.13	0.09±0.04	0.97±0.04	0.11±0.05	0.68	2.78±0.87	0.78±0.33
Tientsin Y29 [1]	Yes	C	2.8-3.8	1.00	59.70±3.66	37.14±2.80	0.08±0.03	0.95±0.06	0.09±0.04	0.74	1.93±0.22	0.91±0.59
T1 Tangshan District [2]	Yes	C	4.1-5.8	3.70	82.95±8.95	70.69±4.26	0.40±0.16	0.86±0.09	0.26±0.11	0.75	5.95±1.29	0.38±0.38
T2 Tangshan District [2]	Yes	C	2.3-4.3	1.30	58.80±4.77	39.18±2.93	0.40±0.16	0.92±0.06	0.36±0.15	0.78	3.79±1.56	0.38±0.38
T8 Tangshan District [2]	Yes	C	4.5-6.0	2.00	93.75±5.42	61.87±3.54	0.40±0.16	0.84±0.10	0.33±0.14	0.72	8.03±3.68	0.38±0.38
T10 Tangshan District [2]	Yes	C	6.5-9.8	1.45	150.50±11.37	84.77±5.92	0.40±0.16	0.73±0.14	0.34±0.15	0.75	5.90±1.01	0.38±0.38
T19 Tangshan District [2]	Yes	C	2.0-4.5	1.10	59.26±8.22	38.17±3.71	0.20±0.08	0.94±0.06	0.19±0.08	0.69	8.00±1.74	0.38±0.38
T22 Tangshan District [2]	Yes	C	7.0-8.0	0.80	141.98±5.45	76.25±4.90	0.20±0.08	0.80±0.13	0.19±0.08	0.70	8.83±2.21	0.38±0.38
T32 Tangshan District [2]	Yes	C	2.6-3.9	2.30	59.45±4.72	50.13±3.63	0.15±0.06	0.94±0.06	0.11±0.05	0.74	5.63±0.75	0.38±0.38
Tientsin F13 [1]	No	C	3.1-5.1	0.70	75.80±6.77	42.45±3.66	0.09±0.04	0.93±0.08	0.10±0.04	0.60	1.63±0.35	2.62±0.74
T21 Tangshan District [2]	No	C	3.1-4.0	3.10	59.93±3.66	55.51±3.03	0.20±0.08	0.93±0.07	0.13±0.05	0.72	15.52±1.21	0.38±0.38
T30 Tangshan District [2]	No	C	5.0-8.0	2.50	116.00±10.01	76.76±4.78	0.10±0.04	0.86±0.11	0.08±0.04	0.65	14.92±1.64	0.38±0.38
T36 Tangshan District [2]	No	C	5.7-9.0	2.30	132.75±11.07	83.21±5.33	0.15±0.06	0.82±0.13	0.13±0.06	0.72	7.61±1.10	0.38±0.38
Earthquake	M_w	References:										
1977 Vrancea, Romania	7.20±0.11	Ishihara & Perlea (1984)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
Site 2	No	C	6.5-9.0	1.00	144.25±8.75	78.03±5.47	0.10±0.04	0.79±0.13	0.13±0.06	0.55	3.45±1.82	0.38±0.38
Earthquake	M_w	References:										
1979 Imperial Valley, USA	6.50±0.13	Bennett et al. (1984), Bierschwale & Stokoe (1984)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
Radio Tower B1	Yes	A	3.0-5.5	2.01	74.72±8.20	52.75±4.53	0.18±0.02	0.89±0.08	0.16±0.03	0.52	4.38±2.21	0.96±0.58
McKim Ranch A	Yes	A	1.5-4.0	1.50	47.75±8.12	35.49±4.38	0.51±0.05	0.91±0.05	0.44±0.07	0.52	4.61±1.48	1.13±0.40
Kornbloom B	No	A	2.6-5.2	2.74	65.88±8.50	54.50±4.58	0.13±0.04	0.91±0.07	0.09±0.01	0.44	3.65±2.48	2.45±1.87
Wildlife B	No	B	3.7-6.7	0.90	98.70±10.22	56.52±4.90	0.17±0.05	0.86±0.09	0.13±0.04	0.40	6.45±3.83	1.50±1.00
Radio Tower B2	No	B	2.0-3.0	2.01	41.47±3.65	36.66±3.71	0.16±0.02	0.95±0.05	0.12±0.02	0.40	8.59±5.47	1.41±1.12

Table 4.1—*continued*

Earthquake	M_w	References:										
1980 Mexicali, Mexico	6.20±0.14	Diaz-Rodrigues (1983, 1984), Anderson et al. (1982)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
Delta Site 2	Yes	B	2.2-3.2	2.20	44.20±3.36	39.30±4.19	0.19±0.05	0.94±0.05	0.14	0.90	7.28±1.33	0.04±0.01
Delta Site 3	Yes	B	2.0-3.8	2.00	48.20±5.60	39.37±4.46	0.19±0.05	0.93±0.06	0.15	0.65	3.14±0.56	0.78±0.20
Delta Site 3p	Yes	B	2.2-3.8	2.20	49.60±5.04	41.75±4.40	0.19±0.05	0.93±0.06	0.14	0.58	3.19±0.96	0.93±0.31
Delta Site 4	Yes	B	2.0-2.6	2.00	37.40±2.29	34.46±4.08	0.19±0.05	0.95±0.05	0.13	0.53	5.28±0.46	0.81±0.10
Delta Site 1	No	B	4.8-5.3	2.30	86.30±2.54	59.32±4.33	0.19±0.05	0.86±0.09	0.16	0.43	4.68±0.01	1.96±1.12
Earthquake	M_w	References:										
1981 Westmorland, USA	5.90±0.15	Bennett etl al. (1984), Bierschwale & Stokoe (1984), Youd and Wieczorek (1984)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
Wildlife B	Yes	B	2.7-6.7	0.91	89.31±13.34	51.93±5.94	0.23±0.02	0.86±0.09	0.24±0.06	0.43	6.80±3.13	1.38±0.77
Kornbloom B	Yes	B	2.8-5.8	2.74	73.48±9.75	58.18±4.86	0.19±0.03	0.88±0.08	0.14±0.03	0.40	3.20±1.88	2.78±1.79
Radio Tower B1	Yes	A	2.0-5.5	2.00	72.50±7.71	50.43±4.92	0.17±0.02	0.89±0.08	0.14±0.02	0.52	4.61±1.99	0.88±0.42
McKim Ranch A	No	B	1.5-5.2	1.50	57.30±11.09	39.15±5.56	0.09±0.02	0.92±0.06	0.08±0.02	0.50	5.29±1.35	1.13±0.32
Radio Tower B2	No	A	2.0-3.0	2.01	40.98±3.33	36.17±4.17	0.16±0.02	0.94±0.05	0.12±0.02	0.40	9.52±4.57	1.36±0.73
Earthquake	M_w	References:										
1983 Nihonkai-Chubu, Japan	7.70±0.10	Farrar (1990)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
Akita A	Yes	C	0.8-6.5	0.78	64.16±18.49	37.48±6.60	0.17±0.05	0.93±0.07	0.18±0.08	0.40	5.44±3.38	2.01±2.66
Akita B	Yes	B	3.3-6.7	1.03	91.91±12.97	52.96±5.30	0.17±0.05	0.89±0.09	0.17±0.06	0.52	3.93±1.84	1.05±1.28
Akita C	No	B	2.0-4.0	2.40	49.80±6.59	43.91±3.31	0.17±0.05	0.94±0.06	0.12±0.04	0.48	4.04±0.96	1.77±0.91
Earthquake	M_w	References:										
1983 Borah Peak, USA	6.90±0.12	[1] Andrus, Stokoe, & Roesset (1991); [2] Andrus & Youd (1987)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
Pence Ranch [1]	Yes	B	1.5-4.0	1.55	49.75±8.26	37.98±3.92	0.30±0.06	0.93±0.05	0.24±0.07	0.43	7.54±2.24	1.38±0.76
Whiskey Springs Site 1 [2]	Yes	B	1.6-3.2	0.80	44.80±5.38	29.10±3.13	0.50±0.10	0.93±0.05	0.46±0.12	0.35	8.87±5.04	1.83±1.89
Whiskey Springs Site 2 [2]	Yes	B	2.4-4.3	2.40	59.33±6.44	50.01±3.57	0.50±0.10	0.89±0.06	0.34±0.09	0.32	6.60±3.03	3.90±3.11
Whiskey Springs Site 3 [2]	Yes	B	6.8-7.8	6.80	125.45±5.49	120.45±5.03	0.50±0.10	0.70±0.13	0.24±0.07	0.33	7.80±2.07	2.58±1.65
Earthquake	M_w	References:										
1987 Edgecumbe, New Zealand	6.60±0.13	Christensen 91995), Zhao et al. (1997)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
Robinson Farm E.	Yes	B	2.0-5.5	0.76	57.67±9.26	28.03±4.29	0.44±0.09	0.88±0.07	0.51±0.16	0.60	10.54±4.38	0.37±0.19
Robinson Farm W.	Yes	C	1.0-2.8	0.61	28.84±4.75	16.19±3.13	0.44±0.13	0.95±0.04	0.48±0.19	0.73	13.84±1.97	0.10±0.00
Gordon Farm1	Yes	B	1.2-2.4	0.47	41.38±7.89	19.50±3.82	0.43±0.09	0.92±0.05	0.55±0.19	0.53	8.05±2.68	0.65±0.25
Brady Farm1	Yes	C	6.4-8.0	1.65	117.70±5.77	58.35±4.97	0.40±0.12	0.70±0.13	0.37±0.13	0.52	3.09±1.07	0.97±0.37
Morris Farm1	Yes	B	7.0-8.5	1.63	118.50±5.62	58.46±4.98	0.42±0.08	0.69±0.13	0.38±0.11	0.58	10.39±1.17	0.37±0.06
Awaroa Farm	Yes	B	2.3-3.3	1.15	42.25±2.90	26.06±3.04	0.37±0.07	0.92±0.06	0.36±0.09	0.38	11.36±2.20	1.10±0.25
Keir Farm	Yes	B	6.5-9.5	2.54	121.46±8.66	67.90±5.23	0.31±0.06	0.71±0.14	0.26±0.08	0.43	8.61±1.24	0.31±0.06
James St. Loop	Yes	B	3.4-6.8	1.15	77.90±9.17	39.15±4.58	0.28±0.06	0.85±0.09	0.31±0.09	0.53	9.08±3.00	0.56±0.24
Landing Rd. Bridge	Yes	B	4.8-6.2	1.15	84.10±4.63	41.43±4.06	0.27±0.05	0.83±0.10	0.30±0.08	0.63	10.57±2.07	0.32±0.07
Whakatane Pony Club	Yes	B	3.6-4.6	2.35	61.20±3.21	44.03±3.33	0.27±0.05	0.89±0.08	0.22±0.05	0.88	8.60±1.59	0.10±0.03
Sewage Pumping Station	Yes	B	2.0-8.0	1.29	76.21±15.71	39.81±5.94	0.26±0.05	0.85±0.09	0.28±0.09	0.67	7.47±2.34	0.30±0.21
Edgecumbe Pipe Breaks	Yes	B	5.0-5.9	2.50	81.98±3.41	53.04±3.69	0.39±0.08	0.81±0.10	0.32±0.08	0.40	7.77±1.57	0.39±0.12
Gordon Farm2	No	B	1.7-1.9	0.90	27.00±1.01	18.17±2.77	0.37±0.07	0.95±0.04	0.34±0.09	0.50	21.57±3.25	0.50±0.26
Brady Farm4	No	B	3.4-5.0	1.53	63.57±4.59	37.38±3.53	0.40±0.12	0.86±0.08	0.38±0.13	0.56	13.24±2.09	0.41±0.13
Morris Farm3	No	B	5.2-6.6	2.10	89.35±4.57	52.07±3.99	0.41±0.12	0.78±0.11	0.36±0.12	0.65	12.23±2.08	0.31±0.12
Whakatane Hospital	No	B	4.4-5.0	4.40	68.45±3.23	65.51±3.90	0.26±0.05	0.87±0.09	0.15±0.04	0.50	17.05±2.25	0.49±0.09
Whakatane Board Mill	No	B	7.0-8.0	1.44	114.81±4.76	55.36±4.85	0.27±0.08	0.74±0.13	0.27±0.10	0.63	10.73±2.94	0.43±0.17

Table 4.1—*continued*

Earthquake	M_w	References:										
1987 Elmore Ranch, USA	6.20±0.14	Bennett et al. (1984), Bierschwale & Stokoe (1984)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
Wildlife B	No	B	3.7-6.7	0.90	98.70±10.22	56.52±4.90	0.17±0.05	0.85±0.09	0.16±0.05	0.40	6.45±3.83	1.50±1.00
Earthquake	M_w	References:										
1987 Superstition Hills, USA	6.60±0.13	Bennett et al. (1984), Bierschwale & Stokoe (1984)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
Wildlife B	Yes	B	3.7-6.7	0.90	98.70±10.22	56.52±4.90	0.21±0.05	0.85±0.09	0.20±0.06	0.40	6.45±3.83	1.50±1.00
Earthquake	M_w	References: 1] Mitchell et al. (1994), Kayen et al. (1998); [2] Boulanger et al. (1995), Woodward-Clyde (1990), Rutherford Chekene (1987, 1988);										
1989 Loma Prieta, USA	7.00±0.12	[3] DeAlba et al. (1994), Rollins et al. (1994); [4] Holzer et al. (1994); [5] Bennett & Tinsley (1995), Toprak et al. (1999)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
SFOBB-1 [1]	Yes	A	6.25-7.0	2.99	127.53±4.03	90.64±3.90	0.28±0.01	0.79±0.01	0.17±0.01	0.66	5.63±0.73	0.66±0.13
SFOBB-2 [1]	Yes	A	6.5-8.5	2.99	141.03±7.74	96.79±4.72	0.28±0.01	0.76±0.02	0.18±0.01	0.55	8.84±1.95	0.55±0.23
POO7-2 [1]	Yes	B	5.5-6.8	2.30	111.18±13.02	73.41±5.50	0.28±0.03	0.81±0.11	0.22±0.05	0.70	7.09±0.84	0.45±0.06
POO7-3 [1]	Yes	B	7.1-8.1	2.30	137.50±4.95	85.51±4.35	0.28±0.03	0.75±0.13	0.22±0.05	0.67	10.84±1.20	0.25±0.05
POR-2 [1]	Yes	B	5.3-6.7	2.40	114.15±7.95	74.42±4.17	0.16±0.03	0.82±0.11	0.13±0.03	0.74	2.66±0.76	0.63±0.20
POR-3 [1]	Yes	B	5.0-7.0	2.40	106.80±6.97	71.48±4.01	0.16±0.03	0.84±0.11	0.13±0.03	0.78	2.64±1.15	0.48±0.23
POR-4 [1]	Yes	B	6.0-7.0	2.40	116.30±4.48	76.08±3.81	0.16±0.03	0.82±0.03	0.13±0.03	0.80	2.88±0.59	0.43±0.10
Marine Lab C4 [2]	Yes	A	5.2-5.8	2.50	95.75±3.31	66.32±3.19	0.25±0.03	0.84±0.10	0.20±0.03	0.78	2.92±0.58	0.51±0.16
Marine Lab UC-7 [2]	Yes	B	7.6-9.8	2.00	148.55±10.20	86.75±5.68	0.25±0.03	0.73±0.14	0.20±0.05	0.55	4.90±1.53	1.20±0.57
Sandholdt Rd. UC-4 [2]	Yes	A	2.4-4.6	2.70	56.40±7.28	48.55±2.99	0.25±0.03	0.99±0.01	0.23±0.03	0.60	11.66±8.81	0.44±0.46
Moss Landing S.B. UC-14 [2]	Yes	A	2.4-4.0	2.40	52.40±5.60	44.55±3.86	0.25±0.03	0.95±0.01	0.21±0.03	0.65	7.91±1.15	0.55±0.10
Woodward Marine UC-11 [2]	Yes	B	2.5-3.4	2.50	46.65±3.60	43.22±3.88	0.25±0.03	0.99±0.01	0.20±0.04	0.64	9.40±1.71	0.48±0.10
Habor Office UC-12&13 [2]	Yes	B	2.9-4.7	1.90	66.50±6.14	47.86±4.24	0.25±0.08	0.91±0.07	0.20±0.07	0.56	8.98±5.23	0.58±0.36
T.I. Naval Station [3]	Yes	B	3.5-7.0	1.50	97.43±11.60	60.64±4.67	0.16±0.03	0.87±0.10	0.14±0.04	0.60	5.05±1.91	0.85±0.50
Farris Farm Site [4]	Yes	A	6.0-7.0	4.50	106.75±4.50	87.13±3.87	0.31±0.08	0.90±0.02	0.28±0.05	0.67	4.44±0.52	0.71±0.10
Miller Fam CMF 8 [5]	Yes	A	6.8-8.0	4.91	123.42±5.29	98.99±4.16	0.30±0.07	0.73±0.01	0.25±0.03	0.81	4.83±0.94	0.25±0.20
Miller Farm CMF 10 [5]	Yes	A	7.0-9.7	3.00	155.35±9.52	99.92±5.36	0.30±0.07	0.88±0.02	0.37±0.06	0.45	4.80±2.41	1.93±0.99
Miller Farm CMF 5 [5]	Yes	A	5.5-8.5	4.70	122.40±10.47	99.84±5.18	0.30±0.07	0.77±0.12	0.29±0.04	0.63	7.13±1.57	0.49±0.20
Miller Farm CMF 3 [5]	Yes	A	5.75-7.5	3.00	103.55±6.74	95.70±4.46	0.30±0.07	0.83±0.02	0.26±0.04	0.71	3.27±1.44	0.72±0.44
Model Airport 18 [5]	Yes	B	3.7-4.5	2.40	70.70±3.28	54.02±2.90	0.29±0.07	0.89±0.08	0.22±0.06	0.72	8.93±1.45	0.35±0.09
Model Airport 21 [5]	Yes	B	3.4-4.7	2.40	69.75±4.61	53.56±3.07	0.29±0.07	0.89±0.08	0.22±0.06	0.74	8.38±2.54	0.30±0.11
Farris 58 [5]	Yes	B	7.4-8.0	4.80	131.90±4.16	103.45±4.18	0.31±0.08	0.74±0.13	0.19±0.06	0.67	8.54±0.35	0.48±0.02
Farris 61 [5]	Yes	B	6.0-7.3	4.20	110.43±5.15	86.39±3.92	0.31±0.08	0.78±0.12	0.20±0.06	0.64	4.27±0.58	0.81±0.12
Granite Const. 123 [5]	Yes	B	7.2-7.8	5.00	127.50±4.15	102.98±4.17	0.31±0.08	0.75±0.13	0.18±0.06	0.73	4.36±0.28	0.50±0.16
Jefferson 121 [5]	Yes	B	6.5-7.75	3.40	126.88±5.16	90.33±4.14	0.18±0.05	0.79±0.12	0.12±0.04	0.71	6.10±0.87	0.45±0.08
Jefferson 141 [5]	Yes	B	3.1-4.5	2.10	66.95±4.82	50.27±3.20	0.18±0.05	0.91±0.07	0.13±0.04	0.70	3.02±0.75	0.83±0.26
Jefferson 148 [5]	Yes	B	7.0-7.9	3.00	137.78±4.57	94.12±4.22	0.18±0.04	0.78±0.13	0.12±0.04	0.72	7.20±1.81	0.38±0.11
Jefferson Ranch 32 [5]	Yes	B	2.3-3.1	1.80	45.90±2.98	37.07±2.55	0.17±0.04	0.95±0.05	0.13±0.03	0.79	5.22±0.77	0.31±0.05
Kett 74 [5]	Yes	B	2.3-3.1	1.50	48.15±3.01	36.38±2.55	0.32±0.08	0.93±0.05	0.26±0.07	0.46	8.08±0.88	1.20±0.31
Leonardini 39 [5]	Yes	B	2.3-4.7	1.90	60.80±7.82	45.10±3.58	0.17±0.04	0.92±0.07	0.14±0.04	0.87	6.07±1.88	0.16±0.05
Leonardini 51 [5]	Yes	B	3.1-3.7	1.80	59.20±2.61	43.50±2.63	0.17±0.04	0.93±0.07	0.14±0.04	0.81	2.39±0.32	0.48±0.08
Leonardini 53 [5]	Yes	B	2.7-3.6	2.10	55.13±3.41	44.82±2.73	0.17±0.04	0.93±0.06	0.13±0.03	0.78	6.65±0.82	0.28±0.11
Marinovich 65 [5]	Yes	B	6.8-9.4	5.60	150.90±12.42	121.47±6.07	0.28±0.07	0.95±0.09	0.21±0.06	0.65	6.33±0.48	0.67±0.10
Radovich 99 [5]	Yes	B	4.75-6.9	4.10	79.38±4.42	72.26±3.54	0.28±0.07	0.95±0.09	0.19±0.05	0.62	6.37±0.93	0.74±0.15
Sea Mist 31 [5]	Yes	B	2.8-3.7	0.80	60.33±3.45	36.29±2.80	0.17±0.04	0.95±0.09	0.18±0.05	0.76	2.67±0.79	0.53±0.19
Silliman 68 [5]	Yes	B	4.7-7.1	3.50	103.37±8.23	79.83±4.28	0.28±0.07	0.95±0.09	0.22±0.06	0.64	5.56±0.35	0.69±0.05
SP Bridge 48 [5]	Yes	B	6.0-7.5	5.30	114.38±6.04	100.15±4.38	0.30±0.08	0.95±0.09	0.21±0.06	0.61	3.95±0.73	0.95±0.19
Alameda Bay Farm Is. [1]	No	A	5.0-6.0	2.50	103.75±4.23	74.32±3.56	0.24±0.02	0.95±0.09	0.16±0.03	0.34	7.85±2.98	2.15±0.89
MBARI3 RC-6 [2]	No	A	3.0-4.5	2.60	64.03±5.31	52.74±3.05	0.25±0.03	0.91±0.07	0.18±0.03	0.74	21.48±1.39	0.21±0.06
MBARI3 RC-7 [2]	No	A	4.0-5.0	3.70	74.80±4.19	66.95±3.24	0.25±0.03	0.88±0.08	0.16±0.02	0.70	12.35±0.81	0.30±0.06
Sandholdt Rd. UC2 [2]	No	A	3.0-4.5	2.70	61.20±5.40	50.90±3.51	0.25±0.03	0.91±0.07	0.18±0.03	0.65	25.55±7.61	0.30±0.10

Table 4.1—*continued*

Loma Prieta continued…												
Site	Liquefied?	**Data Class**	**Crit. Depth Range (m)**	**Depth to GWT (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**a_{max} (g)**	**r_d**	CSR	**c**	**$q_{c,1}$ (MPa)**	**R_f (%)**
General Fish CPT-6 [2]	No	A	2.2-3.2	1.70	48.90±3.79	39.09±3.74	0.25±0.03	0.94±0.05	0.19±0.03	0.70	18.06±2.78	0.32±0.06
MBARI4 CPT-1 [2]	No	A	2.3-3.5	1.90	48.08±4.46	38.27±3.28	0.25±0.03	0.93±0.06	0.19±0.03	0.70	18.79±1.99	0.28±0.06
Sandholdt Rd. UC-6 [2]	No	A	6.2-7.0	2.70	123.90±3.87	85.64±4.26	0.25±0.03	0.80±0.12	0.19±0.03	0.70	20.99±0.68	0.30±0.05
Moss Landing S. B.18 [2]	No	A	2.4-3.4	2.40	48.40±4.08	43.50±3.32	0.25±0.03	0.93±0.06	0.17±0.03	0.72	18.94±1.38	0.27±0.05
Leonardini 37 [5]	No	B	2.9-6.1	2.50	78.00±10.38	58.38±4.39	0.17±0.04	0.89±0.08	0.13±0.04	0.74	5.81±1.34	0.35±0.09
Leonardini 52a [5]	No	B	3.8-4.5	2.70	72.83±3.14	58.60±2.94	0.17±0.04	0.90±0.08	0.12±0.03	0.60	3.82±1.07	1.17±0.67
Matella 111 [5]	No	B	1.7-5.1	1.70	60.18±11.15	43.50±4.29	0.15±0.04	0.93±0.07	0.12±0.04	0.71	5.16±0.98	0.47±0.10
McGowan Farm 136 [5]	No	B	2.4-3.1	2.40	46.36±2.99	42.92±2.74	0.26±0.07	0.94±0.05	0.18±0.05	0.57	6.00±0.58	1.07±0.12
Marinovich 67 [5]	No	B	6.2-7.0	6.20	113.40±4.87	109.48±4.57	0.28±0.07	0.95±0.09	0.18±0.05	0.55	14.21±1.03	0.70±0.06
Radovich 98 [5]	No	B	5.1-8.75	3.50	124.54±12.30	90.94±5.53	0.28±0.07	0.95±0.09	0.24±0.07	0.60	8.33±1.74	0.68±0.30
Salinas River Bridge 117 [5]	No	B	6.4-7.4	6.40	113.97±5.29	109.97±4.71	0.12±0.03	0.95±0.09	0.08±0.02	0.46	5.31±0.79	1.64±0.39
Tanimura 105 [5]	No	B	4.2-6.8	4.20-	92.29±8.88	79.54±4.35	0.15±0.04	0.95±0.09	0.11±0.03	0.75	4.56±0.41	0.41±0.05
Earthquake	**M_w**	**References:**										
1994 Northridge, USA	6.70±0.13	[1] Bennett et al. (1998), Holzer et al. (1999); [2] Abdel-Haq & Hryciw (1998)										
Site	Liquefied?	**Data Class**	**Crit. Depth Range (m)**	**Depth to GWT (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**a_{max} (g)**	**r_d**	CSR	**c**	**$q_{c,1}$ (MPa)**	**R_f (%)**
Balboa Blvd. Unit C [1]	Yes	A	8.3-9.8	7.19	162.74±6.91	144.99±5.59	0.69±0.06	0.54±0.15	0.36±0.04	0.33	6.43±3.63	2.58±1.62
Malden St. Unit D [1]	Yes	B	9.2-10.7	3.90	169.80±6.41	110.45±5.45	0.51±0.06	0.57±0.17	0.29±0.09	0.45	2.98±1.42	2.36±1.28
Potrero Canyon Unit C1 [1]	Yes	A	6.0-7.0	3.30	122.67±4.51	91.27±3.92	0.40±0.04	0.76±0.11	0.25±0.04	0.50	6.52±2.51	1.08±0.49
Wynne Ave. Unit C1 [1]	Yes	A	5.8-6.5	4.30	112,76±3.50	94.85±3.38	0.54±0.04	0.74±0.11	4.30±0.35	0.42	8.96±5.77	1.13±0.87
Rory Lane [2]	Yes	A	3.0-5.0	2.70	66.60±6.33	53.85±3.66	0.77±0.11	0.81±0.08	2.70±0.50	0.45	4.78±0.59	1.80±0.90
Earthquake	**M_w**	**References:**										
1995 Hyogoken-Nanbu, Japan	7.20±0.11	Suzuki et al. (2003)										
Site	Liquefied?	**Data Class**	**Crit. Depth Range (m)**	**Depth to GWT (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**a_{max} (g)**	**r_d**	CSR	**c**	**$q_{c,1}$ (MPa)**	**R_f (%)**
Dust Management Center	Yes	B	6.0-8.0	2.00	119.50±6.72	70.45±4.92	0.37±0.11	0.76±0.12	0.31±0.11	0.64	7.83±2.53	0.49±0.20
Imazu Elementary School	Yes	C	8.0-12.0	1.40	185.80±13.87	101.43±7.23	0.60±0.18	0.56±0.17	0.40±0.17	0.90	0.80±0.19	0.80±0.34
Koyo Junior High School	Yes	B	6.5-7.5	4.00	124.50±4.65	95.07±3.96	0.45±0.14	0.74±0.12	0.28±0.10	0.50	8.03±0.54	1.24±0.87
Kobe Customs Maya Office A	Yes	B	4.0-9.0	1.80	121.35±4.66	75.24±3.97	0.60±0.18	0.72±0.11	0.45±0.16	0.78	2.93±0.34	0.40±0.13
Kobe Customs Maya Office B	Yes	B	2.0-6.0	1.80	82.35±3.96	55.86±3.12	0.60±0.18	0.83±0.08	0.48±0.15	0.54	6.98±0.73	0.87±0.17
Kobe Port Const. Office	Yes	B	3.0-5.0	2.50	70.50±3.32	55.79±2.91	0.60±0.18	0.85±0.08	0.42±0.13	0.76	5.99±1.15	0.29±0.11
Koyo Pump Station	Yes	B	5.0-6.0	2.60	99.45±4.19	71.00±3.41	0.45±0.14	0.81±0.10	0.33±0.11	0.65	2.38±0.57	1.75±0.82
Kobe Wharf Public Co.	Yes	B	4.0-5.5	2.10	88.63±5.41	60.33±3.41	0.45±0.14	0.84±0.09	0.35±0.12	0.65	6.03±0.74	0.78±0.40
Koyo Elementary School	Yes	B	6.5-7.0	4.20	119.03±4.61	94.01±3.91	0.45±0.14	0.75±0.12	0.28±0.10	0.54	2.93±1.44	2.17±1.50
Mizukasa Park	Yes	C	6.9-7.9	2.00	138.30±5.00	85.33±4.36	0.65±0.20	0.66±0.13	0.45±0.16	0.75	1.63±0.60	0.99±0.48
Shiporex Kogyo Osaka Factory	Yes	B	4.0-7.0	1.50	93.95±6.39	54.71±4.44	0.40±0.12	0.82±0.10	0.37±0.12	0.74	3.93±2.18	0.41±0.24
Hamakoshienn Housing Area	Yes	B	2.5-5.0	2.00	67.13±8.35	49.96±3.85	0.50±0.15	0.88±0.07	0.38±0.13	0.59	7.00±1.51	0.65±0.22
Taito Kobe Factory	Yes	B	3.2-4.2	1.60	62.73±3.35	42.13±3.38	0.45±0.14	0.89±0.07	0.39±0.13	0.75	4.85±0.86	0.39±0.12
Tokuyama Concrete Factory	Yes	B	4.0-4.8	2.00	74.52±3.06	50.98±3.48	0.50±0.15	0.85±0.08	0.40±0.13	0.80	2.55±0.88	0.40±0.19
Nisseki Kobe Oil Tank A	Yes	B	4.8-6.1	2.40	99.08±4.98	69.15±3.53	0.60±0.18	0.78±0.10	0.43±0.14	0.72	5.30±1.31	0.61±0.36
Nisseki Kobe Oil Tank B	Yes	B	5.0-6.0	2.40	100.05±4.20	69.64±3.42	0.60±0.18	0.78±0.10	0.43±0.14	0.70	6.25±1.34	0.74±0.27
New Port No. 6 Pier	Yes	B	3.5-5.5	2.50	70.50±6.82	55.79±3.55	0.60±0.18	0.85±0.08	0.42±0.14	0.70	9.47±1.60	0.43±0.11
Minatojima Junior High	Yes	B	4.0-4.5	2.70	74.78±2.72	59.57±2.91	0.45±0.14	0.86±0.08	0.32±0.10	0.65	4.71±1.35	0.94±0.42
New Wharf Const. Offices	Yes	B	3.2-3.8	2.60	60.45±2.78	51.62±2.78	0.45±0.14	0.89±0.07	0.31±0.10	0.64	3.56±0.81	0.93±0.64
Fukuzumi Park	No	C	11.0-12.5	3.10	200.80±8.24	115.94±6.85	0.65±0.20	0.48±0.19	0.35±0.18	0.40	17.09±3.45	1.42±0.57
Honjyo Central Park	No	B	4.0-6.0	2.50	95.00±7.25	70.48±3.98	0.70±0.21	0.78±0.09	0.48±0.16	0.56	17.30±3.75	0.60±0.25
Kobe Art Institute	No	B	3.5-3.8	3.00	64.00±2.38	57.62±2.86	0.50±0.15	0.88±0.07	0.32±0.10	0.33	13.64±5.38	1.90±1.31
Yoshida Kogyo Factory	No	B	3.0-5.0	3.00	69.00±6.87	59.19±3.64	0.50±0.15	0.87±0.08	0.33±0.11	0.34	9.43±7.22	2.71±2.73
Shimonakajima Park	No	B	3.0-4.5	2.00	63.28±3.36	46.11±3.38	0.65±0.20	0.86±0.07	0.50±0.16	0.53	19.49±0.80	0.73±0.43
Sumiyoshi Elementary	No	B	2.4-3.2	1.90	46.92±2.68	38.09±3.15	0.60±0.18	0.91±0.06	0.43±0.14	0.54	17.35±4.20	0.66±0.31
Nagashi Park	No	B	1.1-1.8	1.00	26.00±2.60	21.59±2.32	0.65±0.20	0.95±0.03	0.49±0.16	0.51	14.51±4.31	1.05±0.49

Table 4.1—*continued*

Earthquake	M_w	References:										
1999 Kocaeli, Turkey	7.40±0.11	[1] PEER (2000a), Cetin (2002); [2] PEER (2000), Sancio et al. (2002a, 2002b)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
Hotel Sapanca SH-4 [1]	Yes	B	1.2-2.0	0.50	28.10±5.07	17.31±2.31	0.37±0.09	0.96±0.03	0.41±0.12	0.70	3.25±1.41	0.45±0.29
Soccer Field SF-5 [1]	Yes	B	1.2-2.4	1.00	30.30±3.90	22.45±2.48	0.37±0.13	0.96±0.04	0.34±0.10	0.55	2.97±1.84	1.17±0.86
Police Station Site [1]	Yes	B	1.8-2.8	1.00	39.55±3.38	26.80±2.48	0.40±0.10	0.94±0.05	0.36±0.10	0.54	2.33±0.47	1.89±0.55
Yalova Harbor YH-3 [1]	Yes	B	3.0-4.5	1.00	63.60±14.93	39.40±3.12	0.37±0.13	0.90±0.07	0.39±0.11	0.57	8.10±0.66	0.43±0.07
Adapazari Site B [2]	Yes	B	3.3-4.3	3.30	60.40±3.86	55.50±3.10	0.40±0.10	0.89±0.07	0.25±0.07	0.65	5.77±2.62	0.77±0.42
Adapazari Site C2 [2]	Yes	B	3.3-4.8	0.44	73.61±5.26	38.19±3.41	0.40±0.10	0.88±0.08	0.44±0.13	0.64	3.22±1.87	1.03±0.76
Adapazari Site D [2]	Yes	B	1.8-2.5	1.50	35.28±2.56	28.90±2.39	0.40±0.10	0.95±0.04	0.30±0.08	0.75	3.54±1.82	0.58±0.40
Adapazari Site E [2]	Yes	B	1.5-3.0	0.50	40.13±4.85	22.96±2.75	0.40±0.10	0.94±0.05	0.43±0.13	0.73	5.95±2.76	0.41±0.27
Adapazari Site F [2]	Yes	B	6.8-8.0	0.50	42.90±4.01	67.71±5.01	0.40±0.10	0.94±0.05	0.38±0.12	0.53	4.13±1.44	0.91±0.39
Adapazari Site G [2]	Yes	B	1.5-2.7	0.45	37.50±3.96	21.31±2.58	0.40±0.10	0.95±0.04	0.43±0.13	0.84	5.03±1.28	0.32±0.17
Adapazari Site H [2]	Yes	B	2.0-3.0	1.72	41.09±3.43	33.44±2.56	0.40±0.10	0.94±0.05	0.30±0.18	0.68	5.55±2.03	0.58±0.31
Adapazari Site I [2]	Yes	B	3.0-3.5	0.71	58.00±2.46	33.08±2.69	0.40±0.10	0.91±0.06	0.42±0.11	0.72	3.85±1.04	0.56±0.32
Adapazari Site J [2]	Yes	B	2.5-3.5	0.60	44.45±6.36	30.16±2.75	0.40±0.10	0.94±0.05	0.43±0.12	0.65	3.77±1.41	0.80±0.46
Adapazari Site K [2]	Yes	B	2.0-3.0	0.80	43.85±3.43	27.17±2.55	0.40±0.10	0.94±0.05	0.39±0.11	0.62	4.19±1.64	0.91±0.49
Adapazari Site L [2]	Yes	B	2.0-2.8	1.72	38.78±2.75	32.35±2.46	0.40±0.10	0.94±0.05	0.29±0.08	0.75	2.61±1.24	0.57±0.36
Earthquake	M_w	References:										
1999 Chi-Chi, Taiwan	7.60±0.10	PEER (2000b), Stewart et al. (2002, 2003)										
Site	Liquefied?	Data Class	Crit. Depth Range (m)	Depth to GWT (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	a_{max} (g)	r_d	CSR	c	$q_{c,1}$ (MPa)	R_f (%)
Nantou Site C	Yes	B	2.0-4.5	1.00	58.75±8.12	36.68±3.65	0.38±0.08	0.92±0.06	0.36±0.10	0.56	4.46±2.07	1.11±0.62
WuFeng Site B	Yes	B	2.5-5.0	1.12	77.39±8.25	46.68±3.92	0.60±0.12	0.85±0.08	0.59±0.15	0.55	3.22±1.19	0.96±0.61
WuFeng Site C	Yes	B	2.5-5.5	1.20	72.40±9.74	44.93±4.19	0.60±0.12	0.86±0.08	0.59±0.16	0.65	3.16±0.73	1.84±1.33
WufFeng Site A	Yes	B	5.5-8.5	0.80	130.60±10.35	69.78±5.46	0.60±0.12	0.71±0.12	0.56±0.16	0.75	0.99±0.38	2.14±0.66
WuFeng Site C-10	Yes	B	2.5-7.0	1.00	87.25±14.49	50.46±5.65	0.60±0.12	0.82±0.09	0.60±0.18	0.58	2.52±1.36	2.18±2.16
Yuanlini C-19	Yes	B	4.0-5.8	0.57	121.79±6.92	63.62±4.71	0.25±0.05	0.82±0.11	0.25±0.07	0.67	2.78±0.54	1.08±0.29
Yuanlin C-2	Yes	B	2.5-4.0	0.56	60.07±5.14	33.68±3.11	0.25±0.05	0.93±0.06	0.27±0.07	0.75	4.95±1.55	0.49±0.28
Yuanlin C-22	Yes	B	2.8-4.2	1.13	63.11±4.83	39.86±3.01	0.25±0.05	0.92±0.07	0.24±0.06	0.70	5.17±0.70	0.46±0.17
Yuanlin C-24	Yes	B	5.2-7.8	1.20	114.20±7.19	65.15±4.39	0.25±0.05	0.83±0.11	0.24±0.06	0.75	5.33±1.24	0.60±0.26
Yuanlin C-25	Yes	B	9.5-12.0	3.52	193.69±9.49	122.76±6.11	0.25±0.05	0.67±0.18	0.17±0.06	0.61	6.83±0.97	0.80±0.19
Yuanlin C-32	Yes	B	4.5-7.5	0.74	111.78±10.13	60.18±5.03	0.25±0.05	0.84±0.11	0.25±0.07	0.70	4.83±1.49	0.62±0.27
Yuanlin C-4	Yes	B	3.0-6.0	0.66	83.52±9.86	45.85±4.47	0.25±0.05	0.89±0.08	0.26±0.07	0.55	4.60±1.09	1.30±1.34
Nantou Site C-8	Yes	B	5.0-9.0	1.00	130.00±13.28	71.14±6.03	0.38±0.08	0.77±0.12	0.35±0.10	0.55	3.31±0.34	2.08±0.40
Nantou Site C-7	Yes	B	2.5-4.5	1.00	63.50±6.63	38.98±3.38	0.38±0.08	0.91±0.07	0.37±0.09	0.76	2.31±0.87	0.57±0.43
Nantou site C-3 & C-16	Yes	C	12.0-16.0	1.00	263.00±15.19	135.47±9.53	0.38±0.08	0.55±0.20	0.26±0.11	0.74	1.21±0.23	1.96±1.13
Yuanlin C-3	No	C	10.0-13.0	1.79	218.88±11.65	123.62±7.44	0.25±0.05	0.65±0.19	0.19±0.07	0.75	6.74±0.83	0.30±0.14

Notes:
Listed are the means and variances of the parameters for each case history.
M_w=moment magnitude, Crit.=critical, GWT=ground water table, σ_{vo}=vertical total stress at midpoint of critical layer, σ_{vo}'=vertical effective stress at midpoint of critical layer, a_{max}=peak ground acceleration, r_d=nonlinear shear mass participation factor, CSR=uniform cyclic stress ratio, c=normalization exponent, $q_{c,1}$=normalized average cone tip resistance, R_f=friction ratio.
Multiple sets of references are called out by [#]. Case histories can be attributed to one or more of the references cited.
The variance of the Depth to GWT (ground water table) was set at 0.3 meters for all sites, and treated as normal distribution centered on the mean and truncated at the ground surface.

5 Correlations

5.1 PROBABILISTIC PRESENTATION OF RESULTS

Probabilistic triggering correlations were developed using a Bayesian updating procedure as described in detail in Moss (2003) and Moss et al. (2003a). The overall results are presented in Figure 5.1. This plot shows contours of equal probability as $q_{c,1}$ vs. CSR, for M_w=7.5 and σ_v'=1 atm. The median line is the limit state or threshold, equivalent to a 50% probability of liquefaction.

It has been recognized that a disparity between the number of liquefied vs. non-liquefied data points exists. This disparity can bias the resultant limit state. Cetin et al. (2002) explored this bias and presented a consistent method to account for what is called "choice-based sampling bias" as applied to the problem of liquefaction triggering. The same methodology was used in this study. Figure 5.2 shows the shift in the limit state when accounting for choice-based sampling bias.

Figure 5.3 shows the same contours, this time plotted as $q_{c,1,mod}$ vs. CSR, again for M_w=7.5 and σ_v'=1 atm . In this plot the data points have been adjusted for the effects that the "fines" have on the limit state, in other words this is a "clean-sand" representation of the results. The word "fines" is in quotes because for the CPT it is not a measure of the fines content of the soil, but rather the effect of increasing sleeve frictional resistance on soil liquefiability. The frictional resistance is assessed by a combination of the friction ratio (R_f) and the normalization constant (c). The parameter $q_{c,1,mod}$ is essentially analogous to a fines corrected SPT blow count ($N_{1,60,CS}$).

Comparisons of these probabilistic results with some of the more common CPT correlations are shown in Figures 5.4–5.5.

5.2 DETERMINISTIC PRESENTATION OF RESULTS

Shown in Figure 5.6 is a plot of constant friction ratio (R_f) contours, at P_L=15% for M_w=7.5 and σ_v'=1 atm. Data with $R_f \leq 0.5\%$ are shown as circles and dots, and $R_f > 0.5\%$ are shown as solid and hollow diamonds; this separates the database into "clean" and "dirty" soils. This figure is a simplified deterministic representation of the effect that an increasing friction ratio has on the limit state. (The parameters that participate in this are both the friction ratio (R_f) and the normalization exponent (c) in combination, but can be represented by a variable friction ratio at a mean normalization exponent.) An increase in the friction ratio (R_f) correlates systematically to a suppression of the liquefiability of a material. An optimum limit-state function was used to quantify the effect of this suppression of the liquefiability. This effect can be approximated by the equation:

$$q_{c,1,\mathrm{mod}} = q_{c,1} + \Delta q_c \tag{5.1}$$

where $\Delta q_c = x_1 \cdot \ln(CSR) + x_2$

and $x_1 = 0.38 \cdot (R_f) - 0.19$ and $x_2 = 1.46 \cdot (R_f) - 0.73$

The bounds of Δq_c are from R_f = 0.5 to 5.0, where Δq_c=0 when $R_f \leq 0.5$, Δq_c reaches its maximum at R_f = 5.0, and no data exist for $R_f > 5.0$. This correction was regressed from the liquefaction database and represents the change in liquefiability correlated to a change in friction ratio, as a function of CSR.

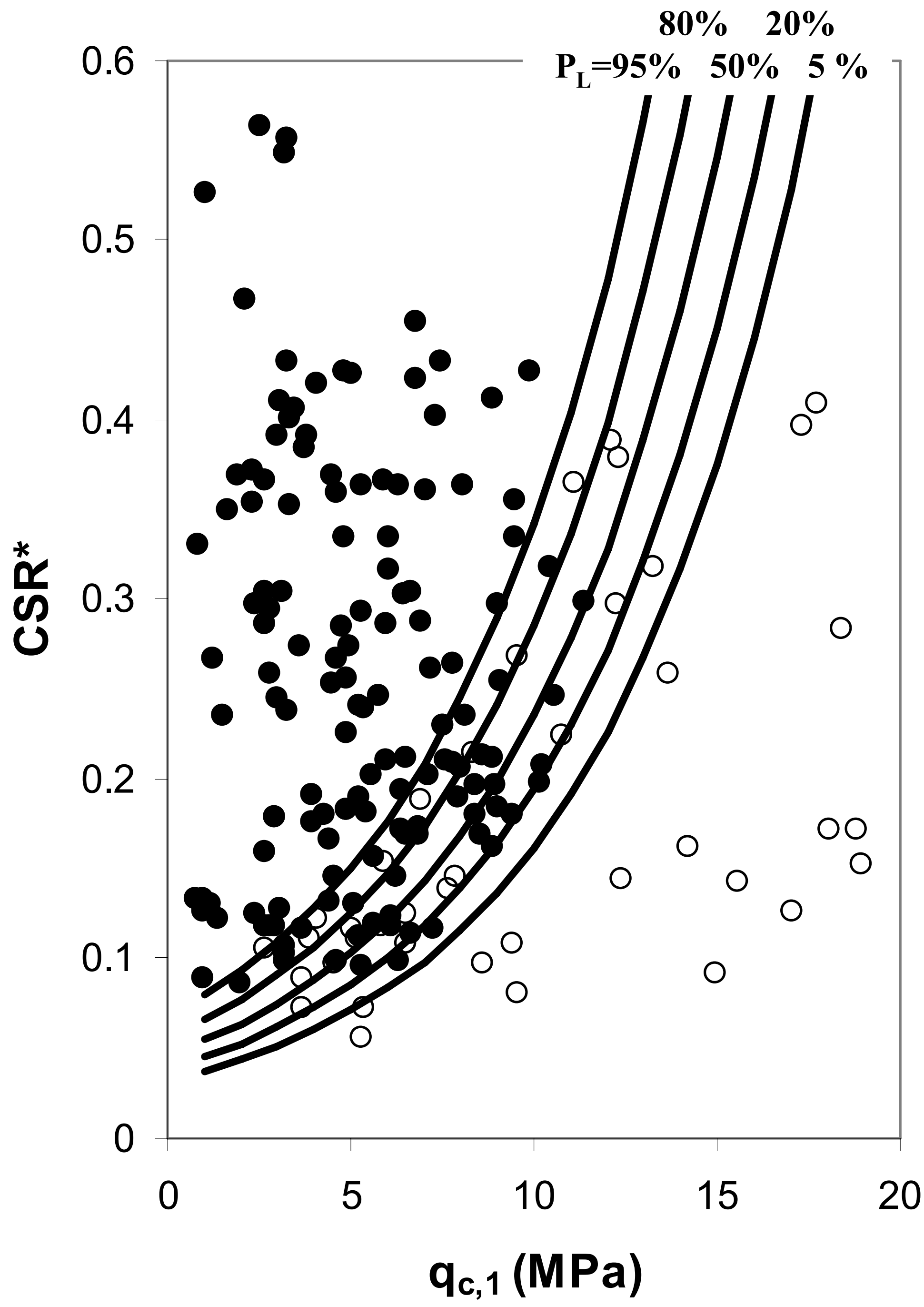

Figure 5.1 Probabilistic liquefaction-triggering curves shown for P_L=5, 20, 50, 80, and 95%. Dots indicate liquefied data points and circles non-liquefied.

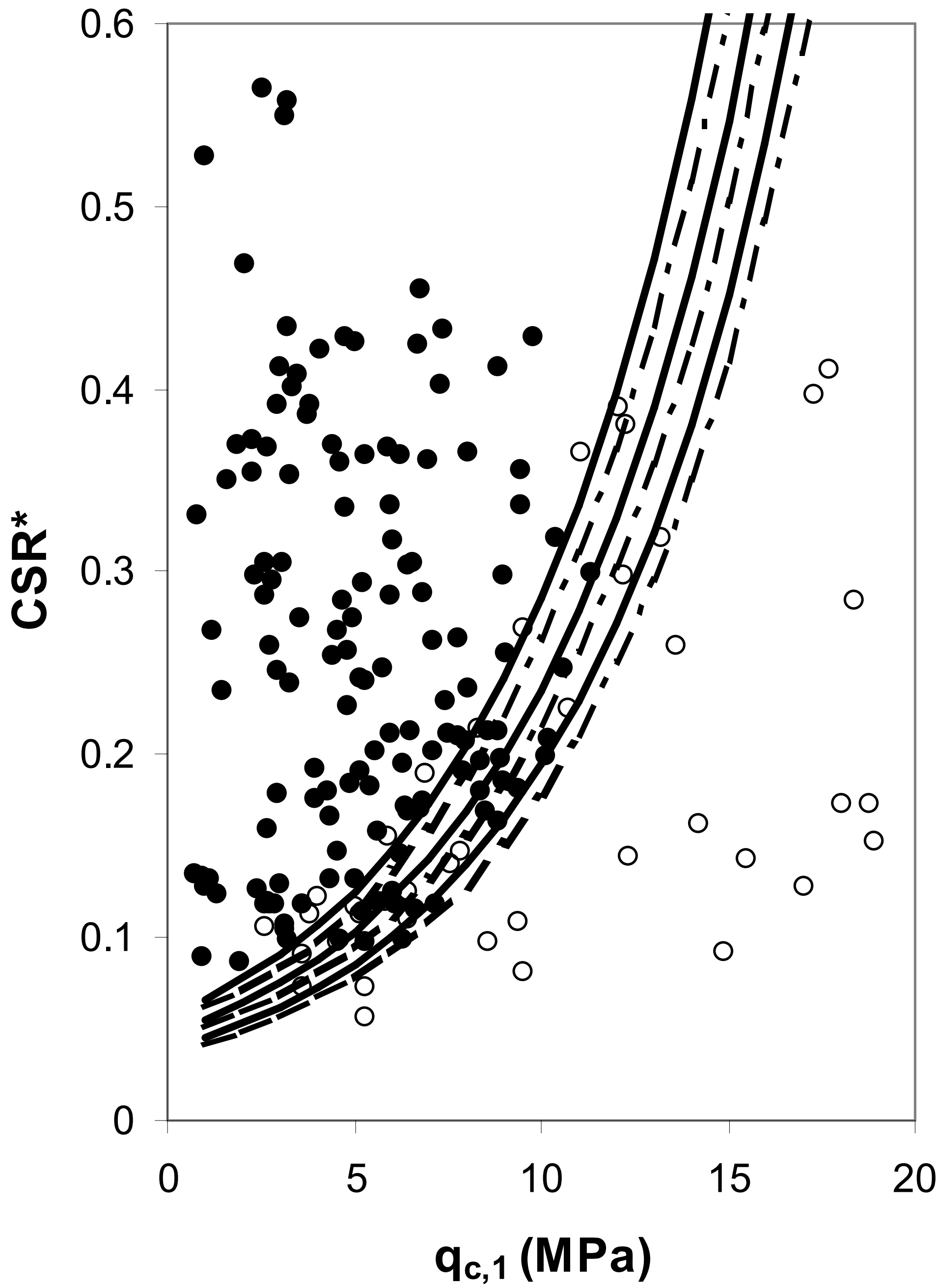

Figure 5.2 Plot showing correction for choice-based sampling bias. P_L=20, 50, and 80% contours are shown uncorrected (dashed) and corrected (solid).

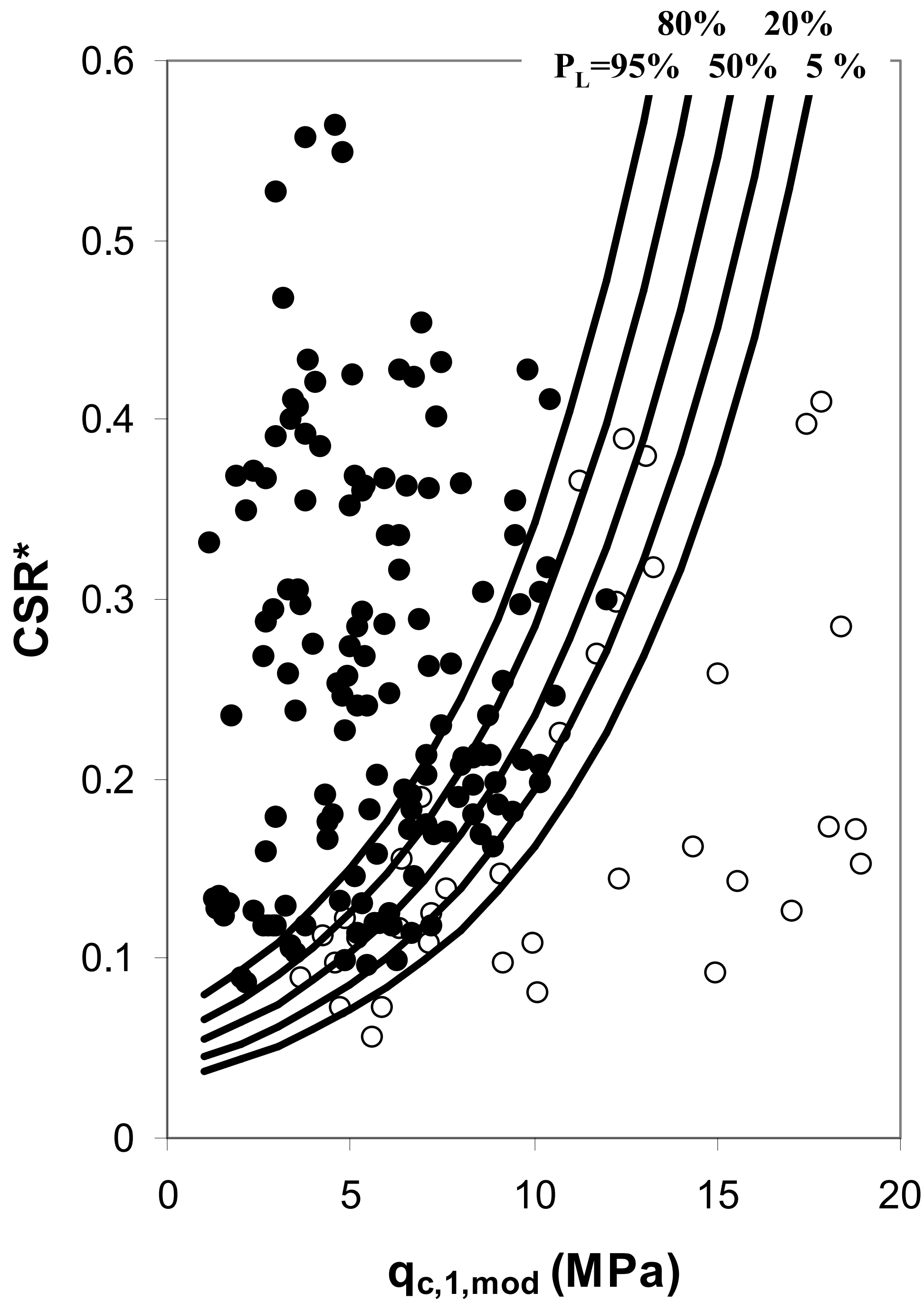

Figure 5.3 Triggering curves shown against data modified for friction ratio

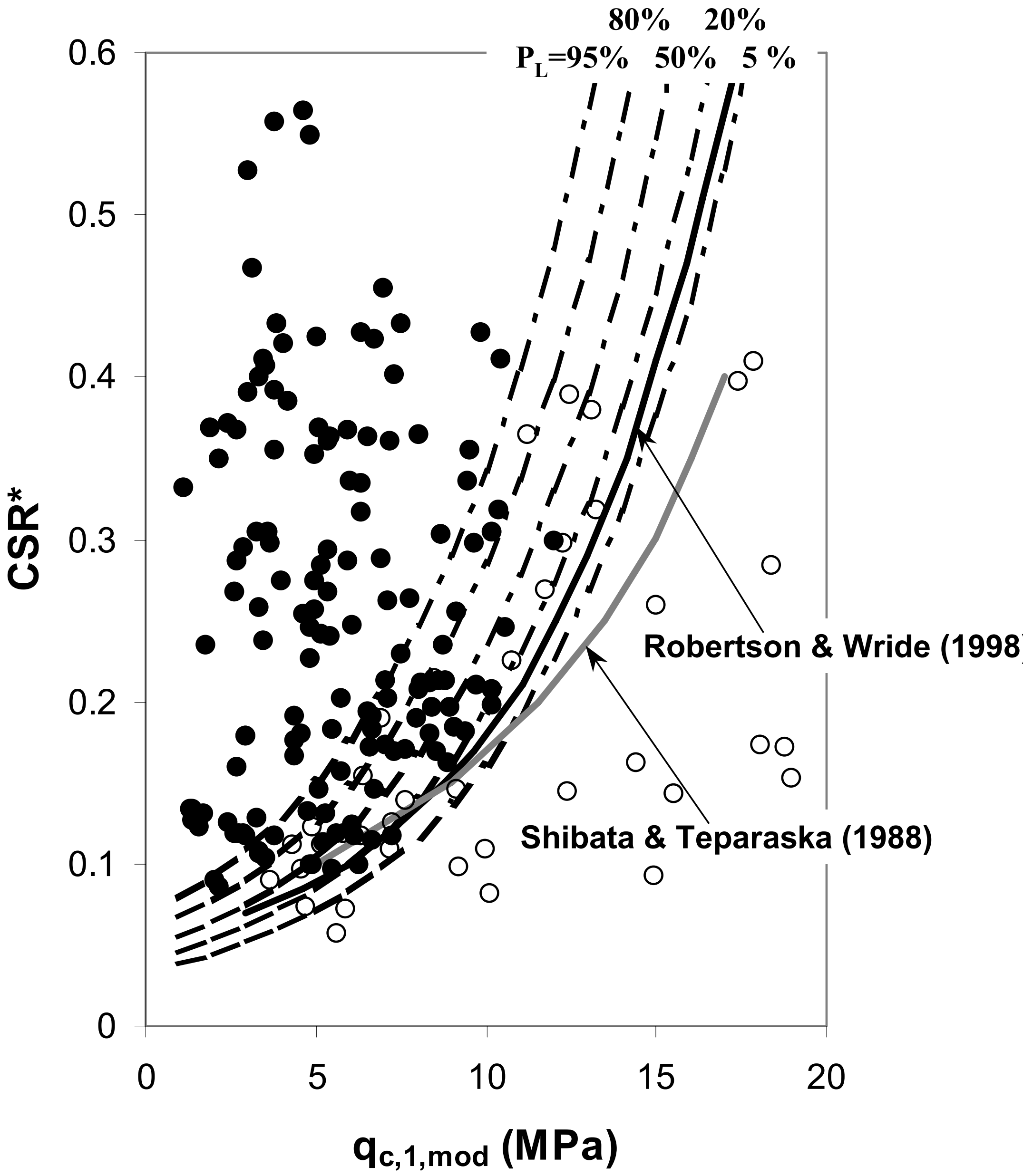

Figure 5.4 Comparison of triggering curves with previous deterministic studies

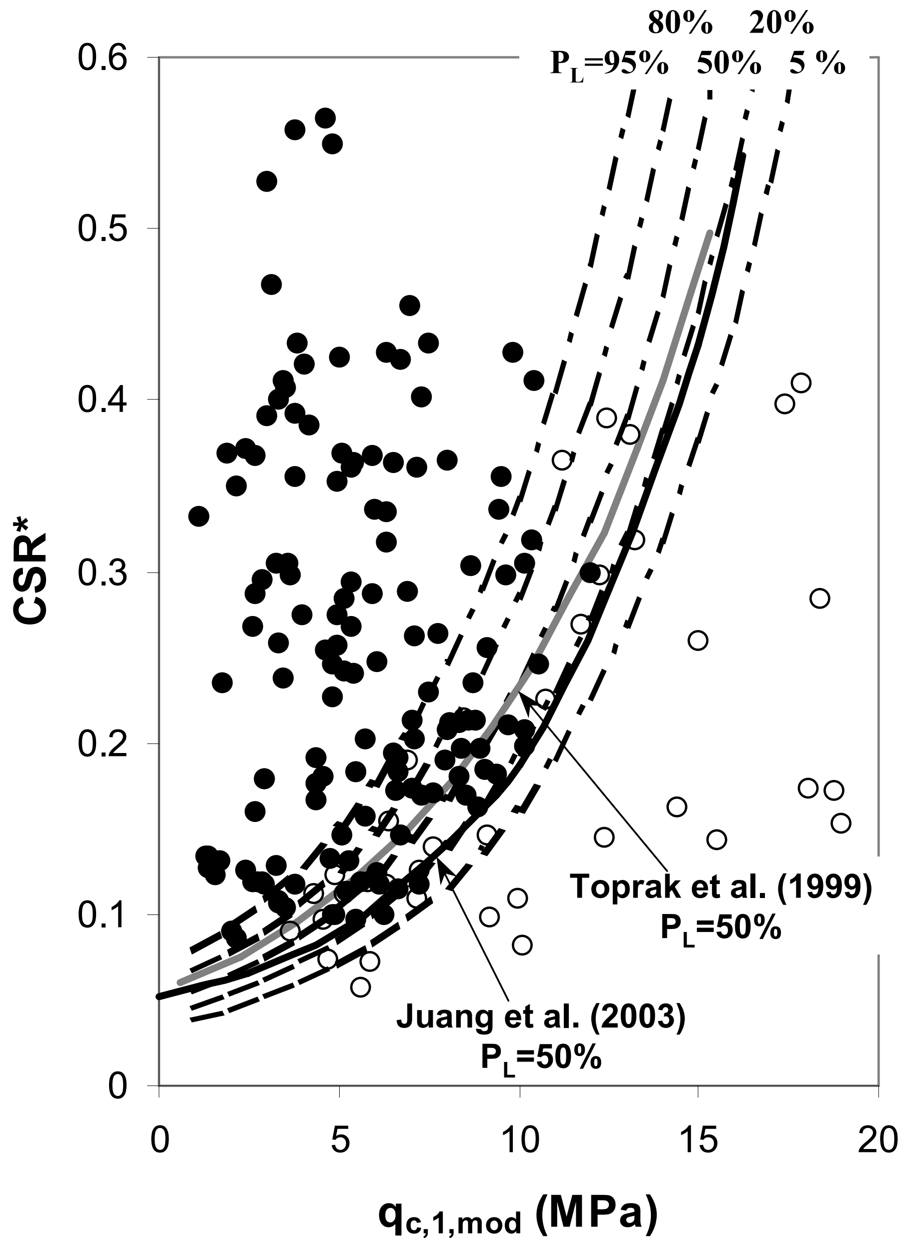

Figure 5.5 Comparison of triggering curves with previous probabilistic studies

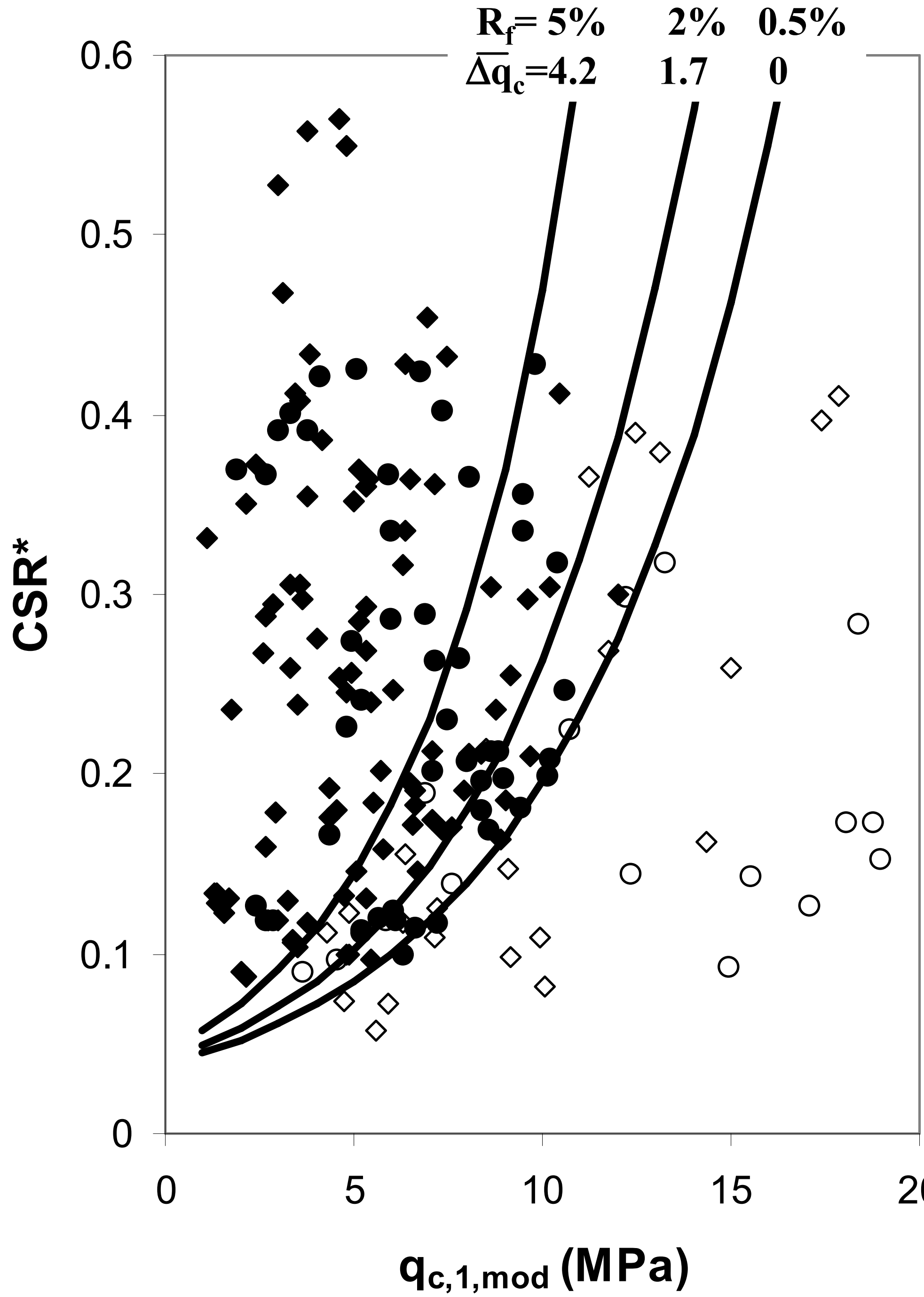

Figure 5.6 Constant friction ratio triggering curves all shown for P_L=15%. Round data points indicate "clean" sands and diamond data points indicate soils of higher fines content.

5.3 PROBABILITY AND DETERMINISM

The probability of liquefaction of P_L=15% was selected as the recommended deterministic boundary, based on prior thresholds of CPT and SPT-based analyses. Particularly for the CPT, the study by Juang et al. (2002) provided insight into where the deterministic threshold has been located by prior researchers, either on purpose or by default. The SPT work by Seed et al. (1985) targeted the limit state at a probability of ~10–15%. The previous CPT-based deterministic correlations are equivalent to a probability of ~10–35%. The threshold at a probability of 15% was selected as a reasonable location for both design safety and for consistency with previous work.

5.4 "FINES" ADJUSTMENT

There is a body of literature on the effects of fines content on soil liquefaction resistance (e.g., Andrews and Martin, 2000; Andrianopoulos et al., 2001; Guo and Prakash, 1999; Perlea, 2000; Polito, 2001; Sancio et al., 2003; Yamamuro and Lade, 1998, Youd and Gilstrap, 1999; to name a few). These studies include both laboratory tests (cyclic triaxial, cyclic simple shear, torsion, etc.) and theoretical analyses. Within the literature there is little consensus, and often one study completely contradicts another. Some of the more difficult laboratory issues include how to measure the void ratio (particularly when measuring minimum and maximum void ratios in "clean" sands, which is a difficult proposition in and of itself); how to create the sample in a consistent manner (pluviation, mixing, etc.); and what criteria should be used to define "failure" and/or liquefaction.

These studies are germane to this research but address only one aspect of the effects captured by the parameter Δq_c. Another aspect is how variable fines content affects the CPT tip and sleeve measurements (i.e., soil "classification"), and what effects this has on the cyclic resistance. An index test measurement includes the effects of all the competing physical phenomena that occur as the measurement is acquired. Physical responses may be working in a constructive or destructive manner to produce the final measurement. The end product is a combination of all these competing effects over time and space.

The cumulative result is that an increase in friction ratio correlates with an increase in liquefaction resistance. This is what has been observed in data trends and what has been quantified using statistical regression. A comparison of previous deterministic analyses on the effects of fines with this study is presented in Figure 5.7. The analysis by Suzuki et al. (1995) is based on a limited database and fit the threshold curves to the data by hand. Robertson and Wride (1998) (also presented in NCEER (1997) and Youd et al. (2001)) used a larger database and also fit the limiting curves by hand. Robertson and Wride (1998) appears to be highly unconservative, with increasing fines.

The nature of I_c, the parameter used by Robertson and Wride (1998) to quantify the effects of fines is based on soil "classification." That is to say I_c is based not on the physics of liquefaction but on soil "classification" which is a secondary correlation of tip (q_c) and sleeve measurements (f_s) to laboratory measured fines content (FC), and is controlled by different physics. The result is an exaggerated estimation of the effect of "fines" on liquefaction resistance. The Robertson and Wride (1998) approach has been found to be lacking in the small zone that is labeled "K_c = 1.0," and Robertson and Wride themselves recommend a null correction for fines in this zone. This area is a region where the I_c curves do not adequately capture the liquefaction behavior of a particular group of soils, and which exists because I_c is defined for soil character and not soil liquefiability. The Δq_c curves presented in this research capture the K_c=1.0 zone accurately because these curves are based on a soil's liquefiability. The Δq_c curves are almost wholly dependent on friction ratio when projected into the log-log space of R_f vs. $q_{c,1}$. In application, Δq_c is an additive function whereas I_c is a multiplicative function, and this difference leads to a dramatic (and unconservative value for I_c) difference in corrected tip resistance as the friction ratio increases.

Figures 5.8–5.9 show the Δq_c contours in relation to Robertson and Wride (1998) I_c contours and to the liquefaction database. As a soil becomes more plastic it is no longer capable of failing in a "classic" liquefaction manner. The limit of confidence in the model is shown as the lower bound on this figure.

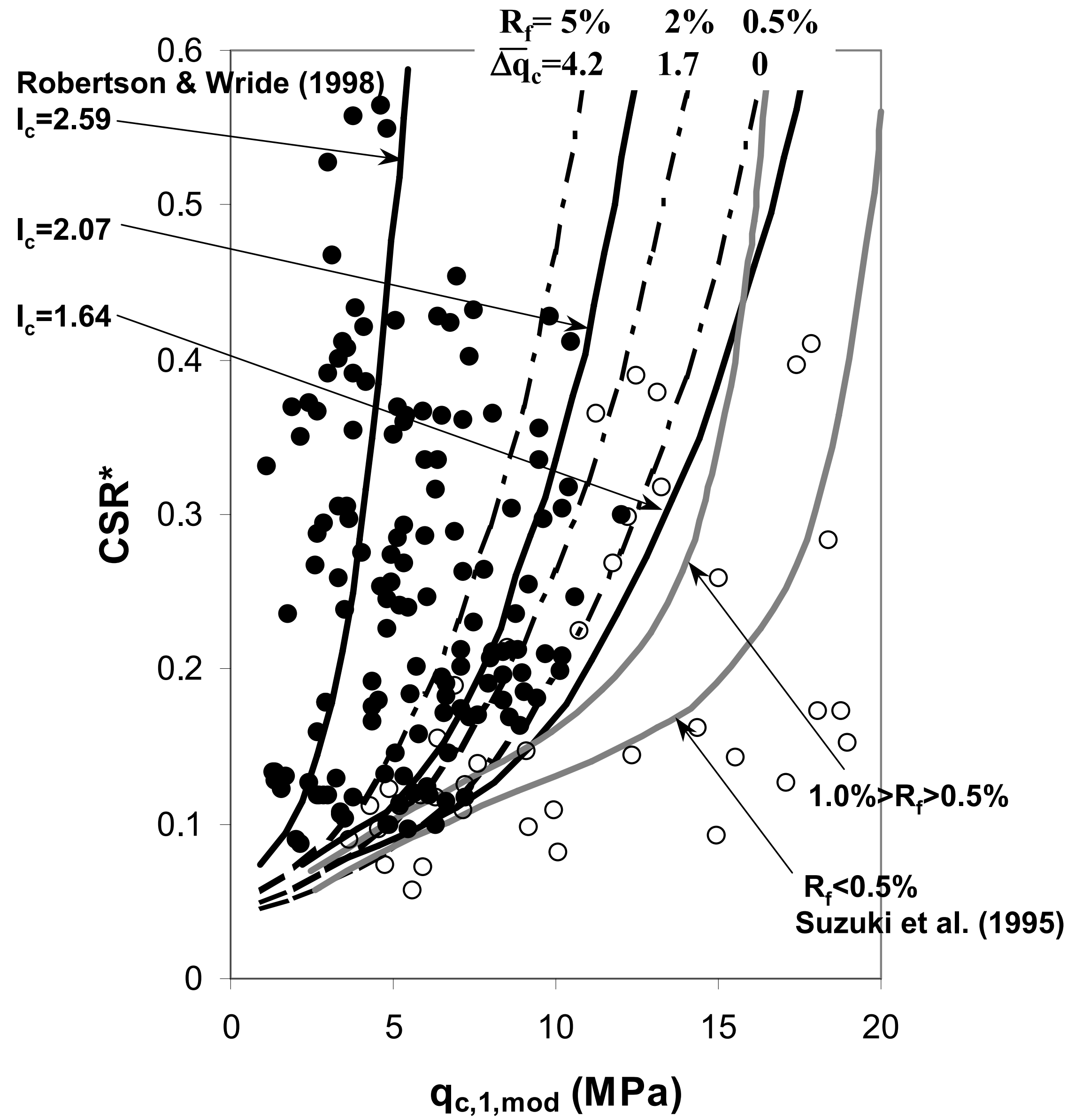

Figure 5.7 Comparison of constant friction ratio triggering curves with previous studies that included effects of "fines" on liquefiability

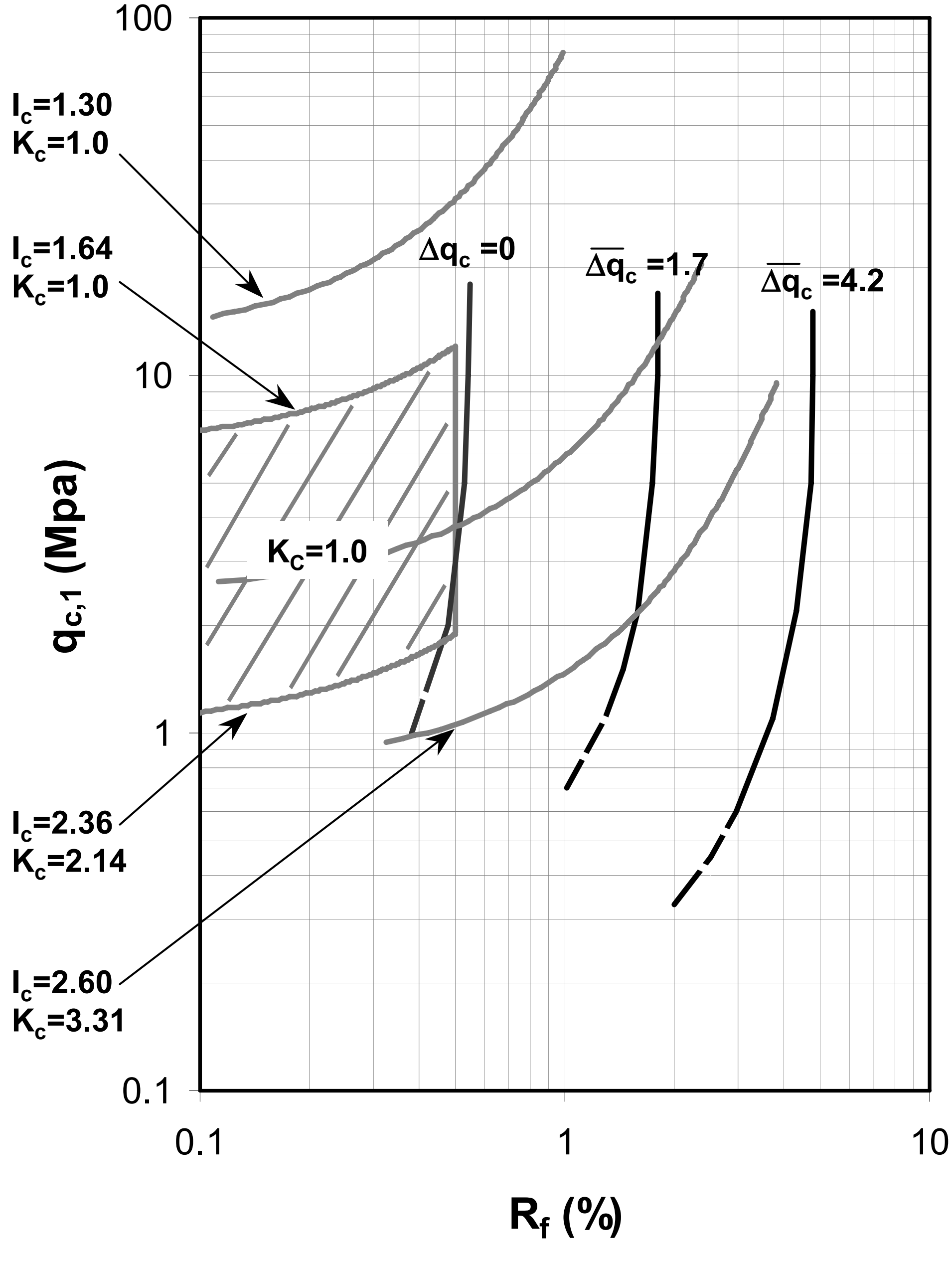

Figure 5.8 Comparison of Δq_c and I_c curves

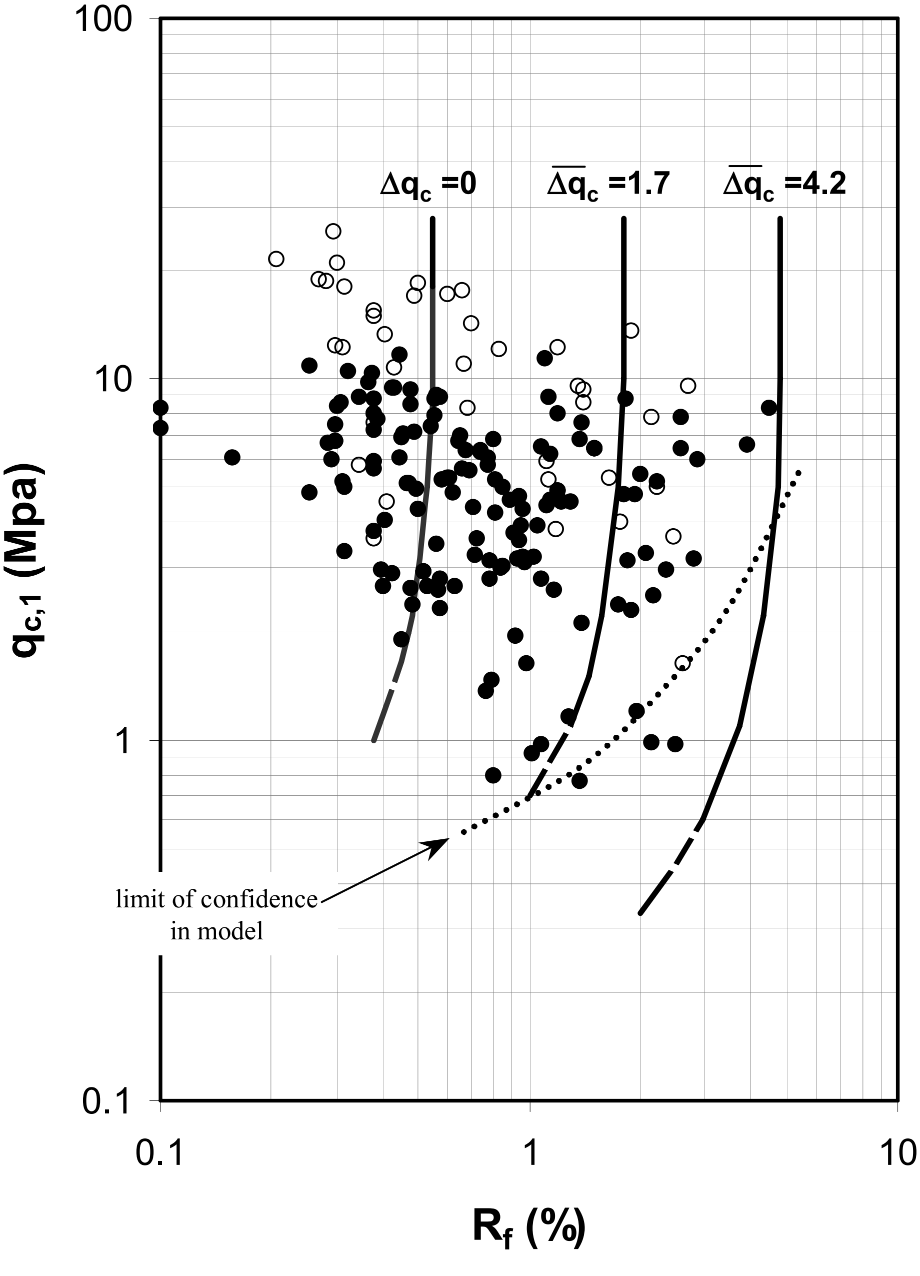

Figure 5.9 Curves of Δq_c shown against liquefaction database

5.5 FINAL CORRELATION

The resultant correlations can be represented both probabilistically and deterministically as discussed earlier. Usable probabilistic results are shown in Figure 5.3. The equal probability contours can be generated using the equation:

$$P_L = \Phi\left(\frac{\hat{g}}{\sigma_\varepsilon}\right) \tag{5.2}$$

where

P_L = the probability of liquefaction in percent

Φ =standard cumulative normal distribution

$$\hat{g} = q_{c,1}{}^{1.02} + q_{c,1}(\theta_1 R_f) + (\theta_2 R_f) + c(1+\theta_3 R_f) - \theta_4 \ln(CSR) - \theta_5 \ln(M_w) - \theta_6 \ln(\sigma_v') - \theta_7$$

σ_ε = standard deviation of model error term

For the given dataset the model parameters and model error term were estimated, using Bayesian updating methods, and the values are given in the following table.

Table 5.1 Model parameter estimates

	θ_1	θ_2	θ_3	θ_4	θ_5	θ_6	θ_7	σ_ε
Mean	0.110	0.001	0.850	7.177	0.848	0.002	20.923	1.632
Standard Deviation	0.058	0.005	0.086	0.842	0.492	0.007	1.870	0.386
Correlation Matrix								
θ_1	1	-0.255	0.425	0.471	-0.360	0.064	0.464	0.399
θ_2		1	-0.093	-0.205	-0.040	-0.096	-0.254	-0.269
θ_3			1	-.0267	-0.477	0.205	0.296	0.034
θ_4				1	0.357	0.015	0.579	0.493
θ_5					1	-0.020	-0.354	0.462
θ_6						1	0.219	-0.323
θ_7							1	0.371
σ_ε								1

For exact parameter estimation (assuming mean values), this then results in the concise equation:

$$P_L = \Phi\left(- \frac{\left(\begin{array}{l} q_{c,1}{}^{1.045} + q_{c,1}(0.110 \cdot R_f) + (0.001 \cdot R_f) + c(1 + 0.850 \cdot R_f) - 7.177 \cdot \\ \ln(CSR) - 0.848 \cdot \ln(M_w) - 0.002 \cdot \ln(\sigma_v') - 20.923 \end{array} \right)}{1.632} \right) \tag{5.3}$$

The cyclic resistance ratio for a given probability of liquefaction can be calculated from

$$CSR = \exp\left(\frac{\left(\begin{array}{l} q_{c,1}{}^{1.045} + q_{c,1}(0.110 \cdot R_f) + (0.001 \cdot R_f) + c(1 + 0.850 \cdot R_f) \\ - 0.848 \cdot \ln(M_w) - 0.002 \cdot \ln(\sigma_v') - 20.923 + 1.632 \cdot \Phi^{-1}(P_L) \end{array} \right)}{7.177} \right) \tag{5.4}$$

6 Summary and Conclusions

6.1 SUMMARY

The goals of this research were to acquire the most comprehensive CPT-based field performance case history database to date, process this data consistently and to high standards, and then use the results to develop accurate and reliable predictive relationships for assessment of the likelihood of “triggering” or initiation of seismically induced soil liquefaction. Thin layer corrections required for interpretation of CPT for some cases were quantified using a refined thin layer correction which was developed based on an elastic solution, on field data, and on previous recommendations. Improved methods for normalization of tip and sleeve resistance measurements for effects of varying effective overburden stresses were defined using prior empirical work, new theoretical analyses, laboratory calibration chamber test data and field data.

A correlation is only as good as the quality of the data upon which it is based. One goal was to produce a database of the most highly scrutinized and consistently processed liquefaction/non-liquefaction sites available. To achieve this, strict protocols were established for processing and grading data according to the quality of information content. Data that did not meet a minimum level of quality were discarded. The database was then reviewed by a panel of leading experts in the area of soil liquefaction engineering, and consensus views of key parameters for each case were determined.

Proper treatment of the data required a flexible statistical technique. A Bayesian-type analysis was chosen because this statistical technique can accommodate all forms of uncertainty associated with both the phenomenon of seismic “triggering” of soil liquefaction and our attempts to quantify this phenomenon. Reliability methods were utilized to present the results in a formal probabilistic framework.

6.2 CONCLUSIONS

This work resulted in a new CPT-based soil liquefaction "triggering" correlation that provides improved ability to assess the likelihood of initiation of soil liquefaction during earthquakes. Key elements that led to significant overall improvements relative to prior efforts included the following:

- A significantly larger number (more than 500) of CPT-based field performance case histories were assembled and analyzed.
- The quality and quantity of the field data, the careful and consistent processing of this data under the supervision and review of an expert panel, and the screening of the processed data based on information content and reliability of each case, resulted in a processed case history database with minimal uncertainty.
- The methods used to quantify CPT data for liquefaction purposes were scrutinized by the authors and the review panel. The canonization of these methods should result in more consistency throughout the field of liquefaction engineering in both acquiring and processing future data.
- The new and improved procedures for normalization of CPT tip and sleeve resistances for the effects of varying effective overburden stress represent an improvement over previous empirical work, and will likely have value beyond the narrow application of liquefaction hazard assessment.

Using higher-order statistical methods to characterize and deal with the various forms of uncertainty resulted in a much-improved basis for estimation of the likelihood of triggering of liquefaction during earthquakes. Moreover, the results are presented in a formal probabilistic framework, facilitating the assessment of risk and uncertainty in performance-based engineering, as well as in a more simplified "deterministic" framework based on a selected and defined level of risk.

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