LIQUEFACTION RESPONSE OF SOILS IN MID-AMERICA EVALUATED BY SEISMIC CONE TESTS

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ABSTRACT

The Mid-America earthquake region is recognized as containing significant seismic hazards from historically-large events that were centered near New Madrid, MO in 1811 and 1812 and Charleston, SC in 1886. Methods for evaluating ground hazards as a result of soil liquefaction and site amplification are needed in order to properly assess risks and consequences of the next seismic event in these areas. In-situ tests provide quick, economical, and practical means for these purposes. For this research effort, seismic piezocone penetration tests (SCPTu) have been performed at a number of sites in the heart of the Mid-America earthquake regions. Many of these sites have already been associated with liquefaction features such as sand dikes, sand boils, or subsidence, observed during geologic and paleoseismic studies. Data collected at these sites have been analyzed under current methodologies to assess the validity and internal consistency of empirical relations developed for Chinese, Japanese, and Californian interplate earthquakes when applied to historical Mid-American earthquakes. Validation of extrapolation of cyclic resistance curves to high cyclic stress ratio values will be considered. Lower bound estimates of moment magnitude (M_w) from parametric studies on SCPTu data indicate an earthquake event of magnitude greater than 7.0 would have been necessary to induce soil liquefaction at the sites studied.

INTRODUCTION

It is now recognized that several of the largest historical earthquake events in the United States occurred in the New Madrid, MO area during 1811 and 1812, and in Charleston, SC in 1886. Large events prior to these times are also acknowledged. The New Madrid

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series of 1811-1812 consisted of over 200 separate seismic events, which would have created an equivalent single event with a moment magnitude (M_w) of about 8.3 (Johnston & Schweig, 1996). The three largest individual events of the series were estimated to have moment magnitudes estimated at 7.9, 7.6, and 8.0 on December 16, 1811, January 23, 1812, and February 7, 1812 respectively (Johnston & Schweig, 1996). The Charleston, SC earthquake consisted of a single event on September 1, 1886, with a M_w estimated at 7.0 (Stover & Coffman, 1993).

Ongoing research on the magnitude, attenuation, and recurrence of earthquake events in Mid-America has led to the increased awareness of the potential for serious ground failures in the New Madrid, MO seismic zone and Charleston, SC earthquake region. Strong ground motions can lead to injury and death from damaged structures, primarily from the collapse of buildings and bridges. Site amplification and liquefaction-induced ground failures may increase the severity of earthquake effects. Large lateral and vertical movements will rupture pipelines and utilities, crippling lifeline facilities needed to provide aid and relief to the injured.

It will be desirable to evaluate the response of soils to earthquake shaking and potential for liquefaction in an expedient and cost effective manner in the Central and Eastern United States (CEUS). However, the evaluation of liquefaction response of soils is complicated in Mid-America due to the:

- deep vertical soil columns (600 m to 1400 m) of the Mississippi River Valley and Atlantic Coastal plain;
- infrequency of large events needed for calibration of models and analysis techniques (most recent sever event, $M_w > 6.5$, more than 100 years ago);
- uncertainty associated with the mechanisms and subsequent motions resulting from intraplate earthquakes (e.g., California earthquakes are interplate events).

SEISMIC PIEZOCONE TEST

To obtain parameters for engineering analysis and model studies, field test data are necessary. The seismic piezocone penetrometer is an electronic probe that rapidly provides four independent parameters to assess the subsurface profile with depth at an individual site. Figure 1 presents data from a seismic piezocone sounding in West Memphis, AR, including tip resistance corrected for pore pressure effects (q_t; Lunne et al., 1997), sleeve friction (f_s), penetration porewater pressure measurement measured behind the tip (u_2), and shear wave arrival time (t_s). The arrival time is incorporated into a pseudo interval analysis method (Campanella et al., 1986) for determination of shear wave velocity (V_s).

With regards to liquefaction evaluation, the individual recordings from seismic piezocone penetration tests (SCPTu) can be valuable in evaluating input parameters as illustrated by Figure 2. Specifically, the readings are processed to obtain:

- Direct measure of small strain shear stiffness ($G_{max} = \rho \cdot V_s^2$);
- Soil type and stratigraphy (q_t, $FR = f_s/q_t \cdot 100$, u_2);

- Depth of water table in sands (u_2);
- Liquefaction susceptibility from direct analysis (q_c and V_s);
- Estimations of properties for rational analysis (ϕ', D_r, OCR, K_o).

The additional dynamic soil properties of peak particle velocity (PPV or $\dot{u}$), and strain level (γ_s = PPV / V_s) can be determined from the shear wave velocity and geophone output (Figure 3).

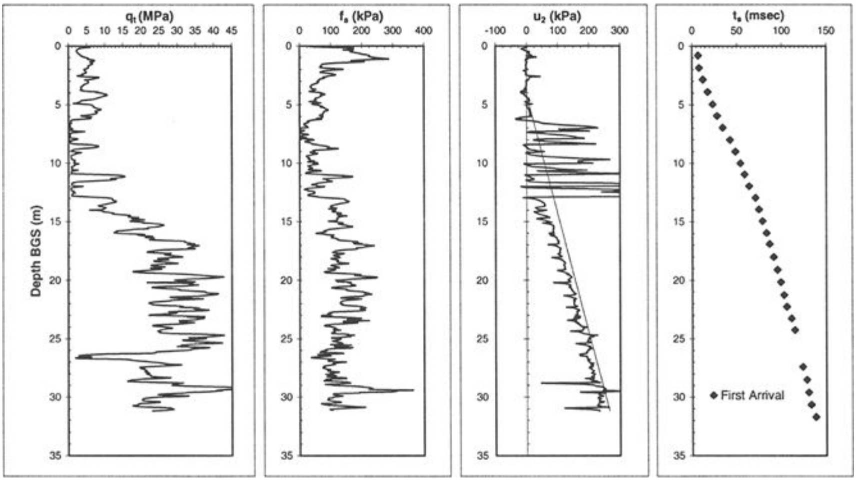

Figure 1. Raw Data from Seismic Piezocone in West Memphis, AR (MEMPH-K)

SEISMIC GROUND MOTIONS IN MID-AMERICA

Before an earthquake analysis can be performed, critical ground motion parameters must be selected. An assessment of ground motion hazards is difficult in the Mid-America earthquake region due to the lack of strong earthquakes in recent historical times (t ≈ 100+ years), and lack of recorded data from the limited events that have occurred. A stochastic ground motion model has been under development for the Central and Eastern United States (CEUS), and attenuation relationships for rock sites have been formed using this model (e.g., Toro et al., 1997). Synthetic ground motions based on a representative stiffness profile of the Mississippi River Valley deep soil column are under development for Mid-America (Herrmann & Akinci, 1999).

For this study, maximum horizontal acceleration (a_{max}) and Arias intensity (I_h) were determined using the Herrmann & Akinci (1999) ground motions program. The output

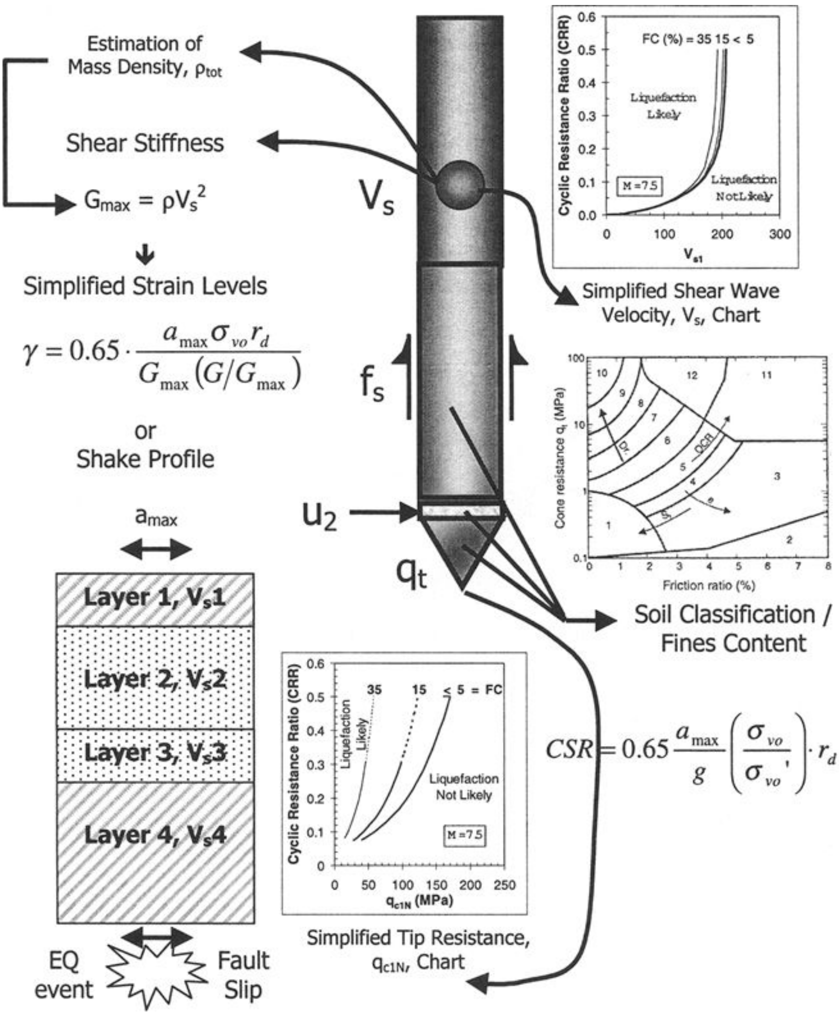

Figure 2. Seismic Piezocone Parameters used for Earthquake Analysis of Soil

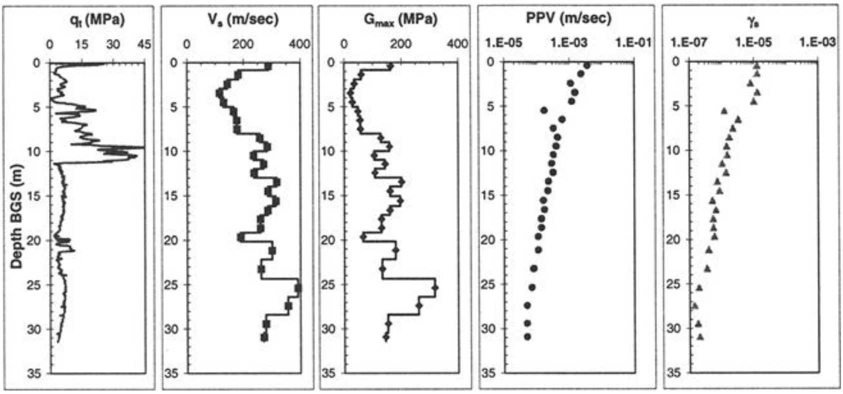

Figure 3. Dynamic Properties Determined from Seismic Piezocone Sounding at Shelby Farms, Shelby County, TN

of this program was primarily a function of the anticipated depth of the soil column, hypocentral distance to the site and moment magnitude. To determine hypocentral distance as a function of Joyner-Boore distance or epicentral distance the hypocentral depth is necessary. The hypocentral depth was assumed to be 9.3 km in the NMSZ and 10.9 km in Charleston, SC based on the work of Toro et al., (1997). Five ground motion models are available with the Herrmann & Akinci (1999) program, differing in spectral source, wave propagation model, and soil conditions. The model used in this study combined USGS 150-bar spectral scaling (Frankel et al., 1996), with Atkinson and Boore (1995) wave propagation, and a generic deep stiffness profile for the NMSZ (Herrmann et al., 1999).

LIQUEFACTION ANALYSIS OF SOILS IN MID-AMERICA

Since the effects of structure, aging, cementation, and strain history generally cannot be replicated in laboratory specimens of granular materials, the use of in-situ testing results and field performance data has become a popular means of assessing liquefaction susceptibility. In-situ test parameters at sites where surface manifestations of liquefaction were or were not evident have been compared to evaluate cyclic soil resistance. Databases consisting predominantly of sites from China, Japan, and California are available for the Standard Penetration Test (SPT; e.g., Seed et al., 1983), cone penetration test (CPT; e.g., Olson & Stark, 1998), flat dilatometer test (DMT; e.g., Reyna & Chameau, 1991), and shear wave velocity (V_s; Andrus et al., 1999). Analyses by these methods are considered as direct methods for liquefaction assessment of soils. It should be noted that these databases are applicable to Holocene deposits. The soils evaluated in this study consisted of Holocene deposits at sites west of the Mississippi River, with older Wisconsin deposits in the Memphis, TN area.

Evaluations based on two analysis procedures using SCPTu data from the Mid-America region are presented:

1. Cyclic stress based procedures for normalized CPT tip resistance (Robertson & Wride, 1998) and stress corrected shear wave velocity (Andrus et al., 1999);
2. Arias intensity methods for cone tip resistance adapted from the work of Kayen & Mitchell (1997).

A brief discussion of each analysis procedure will be presented, with more detail on each method given in Schneider & Mayne (1999) and the associated references presented above.

Cyclic Stress Based Procedures

Simplified cyclic stress based procedures require the three input parameters of (1) cyclic stress ratio (CSR); (2) normalized in-situ test parameter for which a liquefaction case history databases exists; and (3) cyclic resistance ratio (CRR).

The cyclic stress ratio is a function of the anticipated earthquake and is expressed as:

$$CSR = \frac{\tau_{avg}}{\sigma_{vo}'} \approx 0.65 \cdot \left(\frac{a_{\max}}{g}\right) \cdot \left(\frac{\sigma_{vo}}{\sigma_{vo}'}\right) \cdot r_d \qquad (1)$$

where a_{max} is the maximum horizontal surface acceleration (in g's), σ_{vo} is the total vertical stress at the depth of concern, σ_{vo}' is the effective vertical stress at the depth of concern, and r_d is a stress reduction coefficient. The magnitude and depth dependent stress reduction coefficient (r_d) as presented by Idriss (1999) was used in this study.

Normalized cone tip resistance (q_{c1N}) as expressed in Equation 2 was utilized for this study to maintain consistency with the existing liquefaction databases and previously proposed CRR curves (Robertson & Wride, 1998).

$$q_{c1N} = q_c / (\sigma_{vo}')^n \qquad (2)$$

where n = 0.5 in clean sands, n = 0.75 for silty sands, and q_c and σ_{vo}' are in atmospheres. This CPT tip resistance normalization is not used for soils with fines content greater than 35 percent, but all critical layers this study contained less than 35 percent fines. Overburden stress corrected shear wave velocity (V_{s1}) is expressed as (Robertson et al, 1992):

$$V_{s1} = V_s / (\sigma_{vo}')^{0.25} \qquad (3)$$

where V_s is in m/s and σ_{vo}' is in atmospheres. The cyclic resistance ratio (CRR) is an empirical relationship between a stress normalized in-situ test parameter (e.g., q_{c1N}, V_{s1}) and a soils resistance to cyclic stresses imposed by an earthquake event representing a factor of safety of unity (FS=1). Since the magnitude of an earthquake will effect the cyclic resistance and is not incorporated into the CSR (Eq. 1), the cyclic resistance ratio is normalized to a moment magnitude (M_w) earthquake of 7.5 ($CRR_{7.5}$) using a magnitude scaling factor (MSF). The Idriss (1999) magnitude scaling factors expressed in the following equation were used in this study:

$$MSF = 31.9\,(M_w)^{-1.72} \tag{4}$$

The cyclic resistance ratio curves examined in this study were based on the recommendations of NCEER (1997). For normalized cone tip resistance (q_{c1N}; Eq. 2) the $CRR_{7.5}$ is expressed as (Robertson & Wride, 1998):

$$\text{if } 50 \leq (q_{c1N})_{cs} < 160 \qquad CRR_{7.5} = 93\cdot\left(\frac{(q_{c1N})_{cs}}{1000}\right)^3 + 0.08 \tag{5a}$$

$$\text{if } (q_{c1N})_{cs} < 50 \qquad CRR_{7.5} = 0.83\left(\frac{(q_{c1N})_{cs}}{1000}\right) + 0.05 \tag{5b}$$

where $(q_{c1N})_{cs}$ is a clean sand equivalent normalized cone tip resistance. Since soils in this study are considered relatively clean sands, no adjustment to q_{c1N} (Eq. 2) was necessary. For stress corrected shear wave velocity (V_{s1}; Eq. 3), the $CRR_{7.5}$ is represented as (Andrus et al., 1999):

$$CRR_{7.5} = a\cdot\left(\frac{V_{s1}}{100}\right)^2 + b\cdot\left(\frac{1}{V_{s1}^{\ *} - V_{s1}} - \frac{1}{V_{s1}^{\ *}}\right) \tag{6}$$

where $V_{s1}^{\ *}$ is the limiting upper value of V_{s1} for liquefaction occurrence, and *a* and *b* are curve fitting parameters equal to 0.022 and 2.8 respectively. The limiting value of shear wave velocity in relatively clean sands ($FC \leq 5$) of concern for this study has been estimated to be 215 m/s (Andrus et al., 1999).

It is anticipated that an event in Mid-America could result in cyclic stress ratios on the order of 1.0 or higher at close epicentral distances (Toro et al., 1997). Current field performance data are limited to CSR values typically below 0.4, with most data in the 0.1 to 0.2 range. Reconstituted specimens used in laboratory tests do not fully replicate soil fabric, which will lead to different interpretations of liquefaction resistance from laboratory test data than field observations. Advances in sampling of granular soils by freezing techniques allows in-situ soil fabric to remain relatively undisturbed prior to laboratory testing. The cyclic resistance of a deposit may be estimated using laboratory test results from frozen specimens, but an accurate estimate of K_o will be necessary for a reasonable assessment of field liquefaction resistance.

Field performance based CRR curves can be validated by comparison of laboratory based cyclic resistance from frozen specimens to in-situ test parameters taken adjacent to the sample location. A study by Suzuki et al (1995) presents field shear wave velocity and cone tip resistance data as compared to laboratory cyclic resistance from frozen specimens. Figures 4a and 4b display the data compared to q_{c1} and V_{s1} respectively. The Robertson & Wride (1998) curves match the average value of the data presented in Suzuki et al. (1995) study, but a number of points are misclassified. The uncertainty inherent when using simplified curves should be modeled using a conservative estimate of liquefaction resistance with respect to the field data. The engineer can then judge the factor of safety they are comfortable with based on experience and/or probabilistic methods.

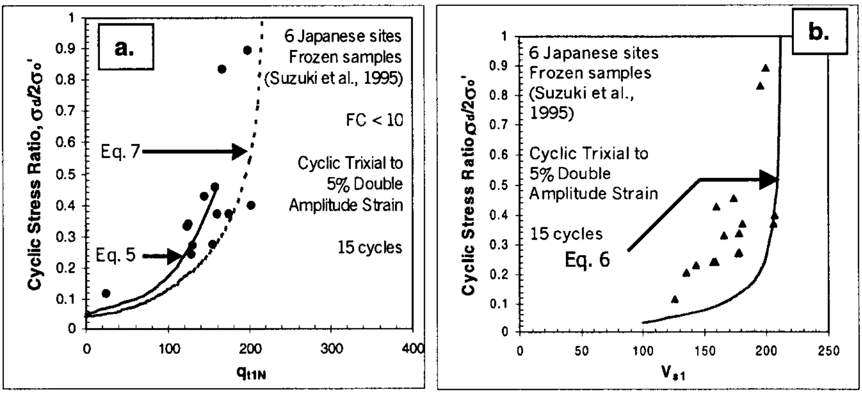

Figure 4. Comparison of CRR curves and Laboratory Frozen Specimens for (a) CPT Tip Resistance (b) Shear Wave Velocity

The format of the Andrus et al. (1999) CRR curve leads to an asymptotic value of shear wave velocity at high values of CSR. A similar form will be adapted for the CRR determined from CPT q_{c1N} data:

$$CRR = a \cdot \left(\frac{q_{c1N}}{350} \right) + \frac{b}{\left(q_{c1N}^{*} - q_{c1N} \right)} \tag{7}$$

where a and b are curve fitting parameters equal to 0.7 and 9.33 respectively. The limiting value of normalized cone tip resistance in clean sands has been estimated at 230 using cyclic triaxial test data presented in Suzuki et al. (1995). The $-b / V_{s1}^{*}$ term from Equation 6 forces the V_s CRR curve through zero. Since it is accepted that the CRR does not pass through the origin (NCEER, 1997), the corresponding CPT term is left out of Equation 7. To validate this curve for field performance data, Figure 5 compares Equation 7 and Equation 5 using the Olson & Stark (1998) CPT liquefaction case history database. Equation 7 is more conservative than currently-recommended methods, but encompasses all of the sites in Figure 5 where liquefaction was evident.

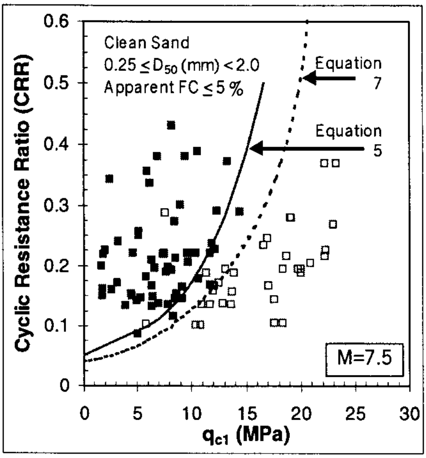

Figure 5. Comparison of CRR curves with CPT Field Performance data

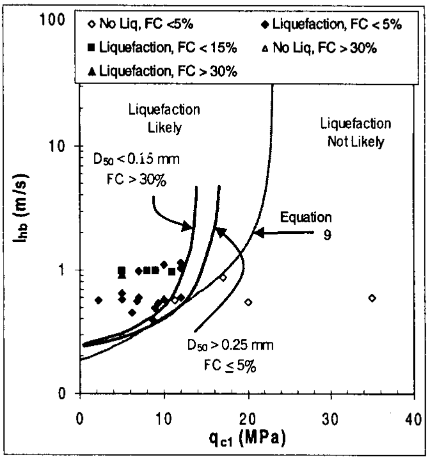

Figure 6. Field Data Compared to Arias Intensity based Resistance Curves

Arias Intensity Based Procedures

A developing method for liquefaction analysis based on Arias intensity of earthquake records has the advantage that it does not require magnitude scaling factors. Although this method seems promising, lack of strong motion data in the Mid-America Earthquake region area leads to increased reliance on synthetic ground motion models. Arias intensity represents the cumulative energy per unit weight in a given direction that is absorbed by a set of single degree of freedom oscillators (Arias, 1970). Horizontal Arias intensity (I_h) can be calculated as the sum of Arias Intensity in the x- (I_{xx}) and y- (I_{yy}) directions as (Kayen & Mitchell, 1997):

$$I_h = I_{xx} + I_{yy} = \frac{\pi}{2g}\int_{0}^{t_o} a_x^{\,2}(t)dt + \frac{\pi}{2g}\int_{0}^{t_o} a_y^{\,2}(t)dt \tag{8}$$

where g is the acceleration due to gravity, $a_x(t)$ is a horizontal acceleration time history, and $a_y(t)$ is the horizontal acceleration time history in the direction perpendicular to $a_x(t)$.

Similar to the CSR from the Seed & Idriss (1971) simplified procedure, the Arias intensity will typically decrease with depth. Depending upon the depth where the time history was recorded and the depth of the liquefied layer, it may be necessary to apply a depth correction factor, r_b. The depth correction factors used in this study were as presented in Kayen & Mitchell (1997).

Simplified liquefaction resistance curves have been generated comparing Arias Intensity (I_{hb}) to penetration resistance [$(N_1)_{60}$ and q_{c1}] for field case histories where strong ground motion data have been readily available. These curves are based on limited field data from California (n=28), and thus Arias Intensity Resistance to liquefaction curves ($I_{hb}R$) are considered approximate. Considering Arias intensity field performance data for the CPT, cyclic stress-based field data for the CPT, and stress-based laboratory tests on frozen specimens compared to CPT tip resistance, the CPT-based liquefaction resistance curve should approach a vertical asymptote. To maintain consistency in analysis, this asymptote should be equal to that presented for cyclic stress-based procedures in clean sands: $q_{c1n}{}^* = 230$. Alteration of the curve fitting coefficients to account for differences between Arias intensity and cyclic stress based analyses, yields the following equation:

$$I_{hb}R = a \cdot \left(\frac{q_{c1N}}{350}\right)^2 + \frac{b}{\left(q_{c1N}{}^* - q_{c1N}\right)} \tag{9}$$

where a = 1.1 and b = 42.7. Figure 6 displays field performance data, the simplified curves from (Kayen & Mitchell, 1997), and the simplified curve presented in Equation 9, thus relating Arias intensity and normalized cone penetration tip resistance to liquefaction resistance. To maintain consistency with data presented in Kayen & Mitchell (1997), q_{c1N} is converted to the units of MPa for Figure 6. Both sets of curves match well with the limited field data, but Equation 9 approaches a more internally consistent asymptote at $q_{c1N}{}^* = 230$.

Selection of Critical Layers

To accommodate evaluation under a number of earthquake magnitude scenarios and liquefaction susceptibility frameworks, critical layers were selected for analysis using a procedure independent of earthquake magnitude and acceleration. A method was developed which combined selection of uniform layers for soil classification purposes (Olsen, 1994), and development of loosened and densified layers as a result of soil liquefaction (Youd, 1984).

In a study of historical California earthquakes, Youd (1984) discusses how expelled porewater from a liquefied deposit can be trapped beneath low permeability layers. This creates a loose layer below the low permeability cap due to the migration of porewater into that layer, and a densified layer below the loose layer due to migration of porewater out of that layer.

To assess uniform layers for classification purposes, Olsen (1994) used the rate of increasing tip resistance compared to effective overburden stress in layer selection techniques. Analysis of the data involved plotting CPT tip resistance and sleeve friction measurements compared to effective overburden stress on a log-log plot. Layers of constant soil type and consistency increase with effective confinement on a slope of *1/c*, where *c* is the stress exponent for normalization. Very dense, overconsolidated soils