Evaluating Liquefaction Resistance at High Overburden Stresses

by

Ross W. Boulanger[1]

ABSTRACT

This workshop paper summarizes a re-evaluation (Boulanger 2003a,b) of three factors that affect the estimation of liquefaction resistance for clean sands under high overburden stresses and sloping ground conditions (e.g., within a large earth dam): the relations used to correct penetration resistances to an equivalent overburden stress of one atmosphere (i.e., C_N), the adjustment factor for the effects of overburden stress on cyclic resistance ratio (i.e., K_σ), and the adjustment factor for the effects of static shear stresses on cyclic resistance ratio (i.e., K_α). These three effects have been investigated in a number of ways and a number of different relations exist for each of them. A relative state parameter index (ξ_R) is used to develop a consistent theoretical framework for interrelating the C_N, K_σ, and K_α factors. This re-evaluation shows that C_N and K_σ are inter-related through the sand properties and state in ways that have compensating effects on the predicted cyclic resistance. Modified relations for practice are provided that offer significant economy over some current methods when applied to very high σ_v' levels. Similarly, a simple method is provided for quantifying the effect of overburden stress on K_α relations.

KEYWORDS: Liquefaction, embankment dams, overburden stress, static shear stress.

1. INTRODUCTION

Two important factors in the semi-empirical evaluation of liquefaction resistance at high effective overburden stresses (σ_v') are the correction of CPT and SPT penetration resistances to an equivalent overburden stress of one atmosphere (i.e., C_N) and the adjustment factor for the effect of overburden stress on cyclic resistance ratio (i.e., K_σ). Overburden stress also affects the adjustment factor for the effect of static shear stress on cyclic resistance ratio (i.e., K_α), which can be important for earth embankment dams. The K_σ and K_α factors were introduced by Seed (1983) and are used as:

$$CRR = CRR_{\sigma=1,\alpha=0}\, K_\alpha\, K_\sigma \qquad (1)$$

where α is the static horizontal shear stress ratio ($\alpha=\tau_s/\sigma_v'$), τ_s is the static horizontal shear stress, σ_v' is the vertical effective consolidation stress, $CRR_{\sigma=1,\alpha=0}$ is the cyclic resistance ratio for $\sigma_v'/P_a=1$ and $\alpha=0$ as obtained through a semi-empirical relation for the earthquake magnitude and other conditions under consideration, and P_a is atmospheric pressure.

This workshop paper summarizes a re-evaluation of C_N, K_σ, and K_α relations for clean sands with specific attention to the high overburden stress levels pertinent to large dams. The details of this re-evaluation are provided in two forthcoming publications: K_α relations in Boulanger (2003a), and K_σ and C_N relations in Boulanger (2003b). Note that this workshop paper has been condensed from a preprint version that contained considerably more detail. C_N relations were re-evaluated based on: (1) a cone penetration theory that has been validated against calibration chamber tests data for σ_v'/P_a up to about 7, and (2) a nonlinear weighted regression analysis of previously published SPT calibration chamber test data for σ_v'/P_a up to about 5.5. The theoretical solutions for C_N as a function of relative density (D_R) fit the SPT and CPT data well, and were used to extend these relations to larger σ_v'/P_a values. The trends in the published C_N, K_σ and K_α relations were

[1] Department of Civil & Environmental Engineering, University of California, Davis, CA 95616 (USA).

shown to be consistent with critical state concepts. Subsequently, a consistent theoretical framework for inter-relating the C_N, K_σ, and K_α relations for clean sands was developed.

2. PENETRATION RESISTANCE

2.1. Solutions for CPT Tip Resistance from Penetration Theory

Salgado et al. (1997a) performed numerical analyses of q_c for about 400 calibration chamber tests on clean sands. Details of the theory and its implementation are presented in Salgado et al. (1997a). The numerical analyses were shown to provide excellent agreement with the experimental data, providing reasonable validation of the theory for predominantly silica sands of medium compressibility.

Solutions for q_c as a function of D_R, σ_v', and the lateral earth pressure coefficient at rest (K_o) were subsequently presented by Salgado et al. (1997b) for typical, upper bound, and lower bound sets of soil properties. Those solutions are closely approximated as (Boulanger 2003b):

$$\frac{q_c}{P_a} = C_0\, C_1 \left(\frac{\sigma_v'}{P_a}\right)^m \left(\frac{K_o}{0.45}\right)^{m-0.077} \quad \text{(2a)}$$

$$m = 0.7836 - 0.5208\, D_R \quad \text{(2b)}$$

$$C_0 = 25.7 + 39.7\, D_R + 212.3\, D_R^2 \quad \text{(2c)}$$

$$\begin{aligned} C_1 = {} & 1.0 \text{ typical soil property set} \\ & 0.64 \text{ lower bound property set} \\ & 1.55 \text{ upper bound property set} \end{aligned} \quad \text{(2d)}$$

with K_o=0.45 being considered as reasonable for most normally consolidated sands.

2.2 . Overburden Correction for CPT & SPT

The correction (or normalization) of q_c to an effective overburden stress of 1 P_a is performed as:

$$q_{C1} = C_N\, q_C \quad (3)$$

which from (2) becomes:

$$C_N = \left(\frac{P_a}{\sigma_v'}\right)^m \quad (4)$$

The C_N relation in (4), as plotted in Fig. 1, is independent of the material property set (as represented by C_1) and matches the widely used relation by Liao and Whitman (1985):

$$C_N = \left(\frac{P_a}{\sigma_v'}\right)^{0.5} \quad (5)$$

at a D_R of 54%.

The dependency of C_N on D_R is similar to that obtained for the SPT calibration chamber tests on four sands by Marcuson and Bieganousky (1977a,b) with σ_v'/P_a = 0.68, 2.72, and 5.44. A weighted nonlinear regression analysis of that data set showed that the C_N expression in (4) appears to provide a reasonable approximation for both the CPT and SPT data (Boulanger 2003b).

2.3 A Corresponding Relative State Parameter Index

A key feature of the CPT penetration theory by Salgado et al. (1997a) is that the peak friction and dilatancy angles are based on Bolton's (1986) relative dilatancy index (I_{RD}):

$$I_{RD} = D_R\left(Q - ln\frac{100\, p'}{P_a}\right) - R \quad (6)$$

where p′ = mean effective normal stress, Q = an empirical constant, and R= a fitting parameter that was taken as 1.0 for the test data that was evaluated. The value of Q determines the value of p′ at which dilatancy is suppressed (i.e., I_{RD} = 0). Bolton indicated that Q depends on the grain type, with Q≈10 for quartz and feldspar, 8 for limestone, 7 for anthracite, and 5.5 for chalk.

The I_{RD} index is necessarily empirical in nature, but has a functional form that is reasonable for describing the combined effects of relative density and confining stress on the dilatancy or volume change characteristics of clean sands.

Boulanger (2003a) used the I_{RD} relation in (6) to derive an equivalent state parameter (ξ). ξ is the difference between the current void ratio and the critical state void ratio for the current value of p′ (Been and Jefferies 1985). The definition of ξ is shown on Fig. 2, along with the critical state lines produced from the I_{RD} relation with Q=9, 10, and 11 (i.e., critical state corresponds to I_{RD} = 0). The curvature of the critical state line reflects the onset of significant particle crushing, and its position is controlled by the parameter Q. ξ can be further normalized by the difference in the maximum and minimum void ratios (e_{max} – e_{min}) to arrive at a relative state parameter (ξ_R) that provides better correlations to the shear behavior of sand (Konrad 1988). This leads to (Boulanger 2003a):

$$\xi_R = \frac{1}{Q - ln\left(\frac{100p'}{P_a}\right)} - D_R \qquad (7)$$

where this ξ_R is an empirical index that has a functional form consistent with critical state concepts, just like the empirical I_{RD} index from which it is derived. The advantage of ξ_R for this study is that it provides a consistent link to the penetration theory of Salgado et al. (1997a).

3. EVALUATING CRR–ξ_R RELATIONS FOR RECONSTITUTED SPECIMENS

The effect of D_R and σ_v' on the cyclic resistance ratio (CRR) of clean sand would intuitively be related to the sand's position relative to critical state or some other reference state. Relating cyclic behavior to a critical state framework in engineering practice is hampered by the difficulty in defining an appropriate critical state or other reference line without recourse to advanced sampling and testing of representative field samples. Consequently, the ξ_R index was evaluated as a practical substitute for representing the combined influence of D_R and σ_v' on CRR.

The CRR – ξ_R relation is illustrated by the laboratory tests on reconstituted clean Fraser Delta sand by Vaid and Sivathayalan (1996) and Vaid and Thomas (1995). The data includes simple shear and triaxial tests on specimens at D_R of 31 to 72% and consolidation stresses (σ_v'/P_a for simple shear, σ_{3C}'/P_a for the isotropically consolidated triaxial tests) of 0.5 to 4. The simple shear and triaxial test data collapse onto single CRR-ξ_R relations, with the quality of fit being better for Q=9 than for Q=10, as shown in Fig. 3. In either case, these data suggest that ξ_R can reasonably represent the combined effects of D_R and σ_v' on CRR.

A CRR-ξ_R relation has a unique corresponding K_σ relation that can be expressed as:

$$K_\sigma = \frac{CRR(\xi_R)}{CRR(\xi_{R1})} \qquad (8)$$

where ξ_{R1} = value of ξ_R for the same D_R at σ_v'/P_a = 1. The K_σ curves derived in this manner for reconstituted Fraser Delta sand are in excellent agreement with the curves derived directly from the experimental data, as expected. These data show that K_σ is dependent on D_R, with K_σ values at $\sigma_v'/P_a > 1$ becoming smaller with increasing D_R, and that K_σ is different for simple shear versus triaxial loading conditions. The results of the simple shear tests are shown on Fig. 4 along with other data and K_σ relations from Seed and Harder (1990), Harder and Boulanger (1997), and Hynes and Olsen (1998). The wide scatter in these data and relations may be due to many factors, including the differences between reconstituted specimens and field samples.

4. DERIVING CRR–ξ_R RELATIONS FOR IN SITU CONDITIONS

CRR-ξ_R relations may be significantly different for in situ conditions than for laboratory tests on freshly reconstituted specimens because CRR is strongly affected by factors such as fabric (deposition), age, stress-strain history, and cementation. Furthermore, laboratory test data are always subject to certain limitations, such as the unavoidable effects of boundary conditions.

CRR-ξ_R relations are consequently derived from the penetration test-based semi-empirical liquefaction relations, thereby retaining consistency with the field data and the eventual application of the findings to practice. For simplicity, it is assumed that the semi-empirical liquefaction correlations pertain exactly to σ_v'/P_a =1 with K_o=0.45. Two CRR–q_{c1} correlations for clean sands in M=7.5 earthquakes are shown in Fig. 5 (Robertson and Wride 1998; Idriss 1999, personal communication). These CRR-q_{c1} relations were mapped onto corresponding CRR-ξ_R relations by mapping q_{c1} to ξ_R. This mapping only requires the selection of a C_1 value for (2), after which each q_{c1} value corresponds to a unique D_R value, which in turn produces a unique ξ_R value (with Q=10 for consistency with the CPT analyses). For calibration to the semi-empirical liquefaction correlations, C_1 was based on the value of D_R that corresponds to a CRR of 0.6 (roughly the limit at which liquefaction might be triggered) when $\sigma_v'/P_a \approx 1$ (called $D_{R,lim}$ hereafter). CPT derivations are presented for C_1 = 1.0 and C_1 = 0.817, which correspond to $D_{R,lim}$ of 75% and 85% at q_{c1}/P_a =174, respectively. For SPT tests, a common correlation is:

$$\frac{(N_1)_{60}}{(D_R)^2} = C_d \qquad (9)$$

SPT derivations are presented for C_d = 53.3 and 41.5, which correspond to $D_{R,lim}$ of 75% and 85% at $(N_1)_{60}$ = 30, respectively.

Three of the resulting CRR-ξ_R relations, covering a range of $D_{R,lim}$, are shown in Fig. 6. The differences in these relations are due to both the choice of $D_{R,lim}$ value and the shapes of the underlying semi-empirical liquefaction correlations. These CRR-ξ_R relations can be used to derive a corresponding consistent set of K_σ relations, as will be illustrated in the following section.

5. EXTENDING THE FIELD CRR–ξ_R RELATIONS TO HIGH OVERBURDEN STRESSES

The effect of overburden stress on a liquefaction analysis is illustrated by tracking its effects on both penetration resistance and CRR. The effect of σ_v' on q_c is accounted for using the relations in (2) and (3), such that the corresponding q_{c1} values are properly independent of σ_v'; Note that if C_N is truly dependent on D_R, then the use of a D_R-independent C_N relation [such as in (5)] would result in calculated q_{c1} values that are not actually independent of σ_v'. The effect of σ_v' on CRR is accounted for by assuming that the CRR-ξ_R relation is applicable for all values of σ_v' (as was previously shown to be an acceptable approximation in Fig. 3). Thus, for clean sand at a given D_R, a change in σ_v' causes a change in ξ_R [via (7)], which results in a change in CRR (Fig. 6). The combined effects of σ_v' are illustrated in Fig. 7 showing how the Idriss (1999) CRR-q_{c1} relation would be modified for σ_v'/P_a = 0.25, 4, and 10 for the case with C_1=1.0. Very similar results are obtained with C_1=0.817 because the C_1 parameter affects both the derivation of the CRR-ξ_R relation and its mapping back onto CRR-q_{c1} plot for different overburden stresses. Increasing σ_v' not only causes a decrease in CRR for a given q_{c1} (shifting the curves downwards), but also increases the limit at which triggering can develop (shifting the curves to the right). This increase in $q_{c1,lim}$ (where CRR exceeds 0.6) with σ_v' occurs because the limiting state at which triggering of liquefaction can occur is actually tied to a limiting value of ξ_R. With $\xi_{R,lim}$ being a constant, it follows that an increase in σ_v' causes an increase in both $D_{R,lim}$ and $q_{c1,lim}$.

The CRR-q_{c1}-σ_v' relations shown in Fig. 7 can also be expressed in terms of implied K_σ relations as:

$$K_\sigma = \frac{CRR}{CRR_1} \qquad (10)$$

where CRR_1 = the CRR at σ_v'/P_a =1 for the same q_{c1} value. Note that the identical K_σ relations can also be obtained directly from the CRR-ξ_R relation using (8). K_σ relations derived in this way are illustrated in Fig. 8 for C_1=1.0 and with the Robertson and Wride (1998) and Idriss (1999) baseline correlations. The resulting K_σ relations are dependent on D_R, with K_σ values at σ_v'/P_a >1 becoming smaller with increasing D_R. Note that K_σ values cannot be derived if CRR_1 is undefined, which happens for D_R > 71% in the first case and D_R > 75% in the second case. The relative positions of the K_σ curves are directly related to the slopes of the CRR-q_{c1} correlations and the corresponding CRR-ξ_R curves. These results simply illustrate the fact that CRR-q_{c1} [or CRR-$(N_1)_{60}$] and K_σ relations are not uncoupled, but rather there is a direct correspondence between them that depends on the material properties. In addition, the K_σ curves in Fig. 8 are in reasonable agreement with the available experimental data (e.g., Fig. 4), given the scatter that is undoubtedly related to numerous factors such as grain characteristics, fabric, age, cementation, and stress-strain history.

Lastly, it is worth noting that the ξ_R-based procedure for calculating CRR, as described in this section, does not explicitly involve use of a K_σ factor. The ξ_R-based procedure first requires an estimate of D_R from the measured penetration resistance, perhaps with the interim step of estimating the corrected q_{c1} or $(N_1)_{60}$ value. Either way, this is an iterative process because the penetration resistances, measured or corrected, are nonlinearly dependent on D_R. Once D_R has been obtained, ξ_R would be calculated using (7) and CRR would be calculated via the CRR-ξ_R relation that corresponds to the desired semi-empirical liquefaction correlation.

6. SIMPLIFIED IMPLEMENTATION & COMPARISON TO PRACTICE

6.1. Method A: Representing Current Practice

First, a reasonable adaptation of current practice (hereafter called Method A) is presented for subsequent comparisons. The measured penetration resistance will be corrected using the C_N relation by Liao and Whitman (1986) in (5) and the CRR_1 will be multiplied by the K_σ relation proposed by Harder and Boulanger (1997), which can be closely approximated as:

$$K_\sigma = 1 - C_\sigma \, ln\left(\frac{\sigma'_V}{P_a}\right) \qquad (11)$$

with C_σ = 0.185.

6.2 Method B: Simple Implementation of ξ_R-based Findings

The ξ_R-based approach requires an iterative estimation of D_R that, while not mechanistically difficult, does add another element of complexity to the liquefaction analysis. Consequently, a simplified approximation (hereafter called Method B) that uses D_R-independent C_N and K_σ relations is also introduced. The key observation that makes this simplification possible is that both C_N and K_σ are actually dependent on D_R, but in ways that produce opposing effects on the predicted CRR. For example, at σ_v'/P_a >1, an increase in D_R causes both an increase in C_N and a decrease in K_σ. A single D_R-independent C_N relation was therefore derived that tracked the limiting condition at which liquefaction can be triggered (i.e., $\xi_{R,lim}$). This relation can be reasonably approximated as:

$$C_N = \left(\frac{P_a}{\sigma'_v}\right)^{m_{lim}} \qquad (12)$$

where m_{lim} depends on the baseline liquefaction correlation and the parameters that relate penetration resistance to D_R (i.e., C_1 for CPT and C_d for SPT). Values of m_{lim} are summarized in Boulanger (2003b) for the two CPT liquefaction correlations (with C_1=0.817 and 1.0) and the one SPT liquefaction correlation (with C_d=41.5 and 53.3). Values for m_{lim} varied from 0.37 to 0.46 over the entire range of possibilities considered. Note that m and m_{lim} are not equal for equal values of D_R and $D_{R,lim}$, respectively, because m_{lim} tracks the effects of σ_v' on CRR as well as on penetration resistance.

The second part of Method B is the choice of a compatible D_R-independent K_σ relation. The K_σ relation was assumed to follow (11) because it reasonably approximates the curves in Fig. 8. A single C_σ value was chosen (for the given m_{lim}) that predicts CRR values that remain conservative relative to the ξ_R-based values, but to the minimum extent possible.

A comparison of Method A, Method B (m_{lim} =0.43, C_σ=0.16), and the ξ_R-based method for σ_v'/P_a = 1, 4, and 10 is shown in Fig. 9 for the Idriss (1999) correlation and C_1=1. Note that CRR is plotted against the measured (not corrected) q_c value. Method B essentially matches the ξ_R-based method in this case. Method A is substantially conservative (relative to the other methods) at high σ_v', particularly in setting the limit of q_c values for which triggering of liquefaction is considered possible. For example, at depths where σ_v'/P_a = 10, measured q_c values would have to exceed about $550P_a$ to be nonliquefiable by Method A, compared to about $470P_a$ by Method B.

Another comparison of Method A, Method B (m_{lim}=0.39, C_σ=0.21), and the ξ_R-based method is presented in Fig. 10, this time using the SPT-based correlation of Seed et al. (1985), as modified in Youd et al. (2001), with C_d=41.5. Method B produces CRR values that are conservative relative to the ξ_R-based method, but substantially less so than for Method A. At depths where σ_v'/P_a = 10, measured N_{60} values would have to exceed about 95 to be nonliquefiable by Method A, compared to about 74 by Method B.

The degree of conservatism in Method A relative to Method B (or the ξ_R-based method) depends on the $D_{R,lim}$ value as set by C_1 for the CPT and C_d for the SPT. A higher $D_{R,lim}$ results in a smaller m_{lim} value and hence a greater difference between Methods A and B.

7. RELATING K_α TO ξ_R

ξ_R was also evaluated as a reasonable index for expressing the combined effects of D_R and σ_v' on K_α for clean sands (Boulanger 2003a). Experimental data on clean sands are consistent with critical state concepts (e.g., Vaid and Chern 1985, Mohamad and Dobry 1986), but the difficulty in defining critical or quasi-steady state lines on sufficiently representative field samples has limited the use of critical state theories in practice. The alternative of estimating critical state parameters for any particular sand is seldom attempted, probably because of real and perceived difficulties in making such estimates.

The general trends in K_α data are well represented by the simple shear test results by Vaid and Finn (1979) and Boulanger et al. (1991), as summarized in Fig. 11. Data are shown for Ottawa sand at D_R of 50% and 68% under σ'_{vc} of $2P_a$ (Vaid and Finn 1979) and Sacramento River sand at D_R of 35% and 55% under σ'_{vc} of $2P_a$ (Boulanger et al. 1991). These results correspond to 10 cycles of loading and a failure criterion of 3% shear strain. The ξ_R value for each set of test data is shown beside the corresponding K_α curve in Fig. 11, based on Q=10 for these uniformly-graded quartzitic sands. There is a consistent progression in K_α values from less than 1.0 for the loosest sand (ξ_R = -0.16) to greater than 1.0 for the densest sand (ξ_R = -0.49), with the ξ_R values simply tracking the progression from D_R = 35% to 68% since all tests were at the same confining stress.

The effects of a large variation in confining stress are best represented by the cyclic triaxial tests on tailings sand by Vaid and Chern (1985). Specimens were prepared at D_R=70% and tested under radial consolidation stresses (σ'_{3c}) of $2P_a$

and $16P_a$; Most other available studies are limited to confining stresses of $4P_a$ or less. These results are also shown in Fig. 11, with the values of α and cyclic resistance re-calculated herein for the eventual failure plane (Seed et al. 1973). Vaid and Chern (1985) also presented consolidation curves and phase transformation states (or quasi-steady state lines) from monotonic loading tests on the tailings sand and Ottawa sand. These results show that the tailings sand (uniformly-graded, angular quartz and feldspar particles) was much more compressible and generally reached quasi-steady state conditions at much lower p′ values than did the Ottawa sand (uniformly-graded, rounded quartz particles) at similar D_R. Based on these data, it appears that an appropriate value of Q would be about 9 (or slightly less) for this tailings sand.

For the D_R = 70% tailings sand specimens at σ'_{3c}/P_a = 2, the value of ξ_R ranges from ξ_R = -0.43 at α = 0 to ξ_R = -0.41 at α = 0.34, based on Q=9. The slight variation in ξ_R with α reflects the slight change in p′ between these different triaxial testing conditions. The ξ_R value is only shown for α = 0.34 on Fig. 11). The resulting K_α relation is intermediate to the tests on Sacramento River sand at D_R = 55% and Ottawa sand at D_R = 68%, despite its greater D_R of 70%. The intermediate nature of its K_α relation is well represented by its ξ_R value (-0.41) being intermediate to the ξ_R values for these other tests (-0.36 to –0.49), as shown in Fig. 11.

For the D_R = 70% tailings sand specimens at σ'_{3c}/P_a = 16, ξ_R ranges from ξ_R = -0.08 at α = 0 to ξ_R = 0.05 at α = 0.34. This latter ξ_R value is the only positive value listed on Fig. 11, and corresponds to the lowest K_α value. Thus, the ξ_R index correctly identified the D_R=70% tailings sand to be more contractive than the D_R=35% Sacramento River sand because of the differences in confining stress. It is also noteworthy that these triaxial tests produced K_α>1 at $\alpha \approx 0.1$, which is related to the importance of shear stress reversal on K_α relations (e.g., see Mohamad and Dobry 1986).

Vaid and Finn (1979) also studied the effect of confining stress on K_α in their simple shear tests on Ottawa sand. For tests on specimens at D_R=50% with α = 0.093, the value of K_α was shown to be essentially the same for σ'_{vc}/P_a of 2, 3, or 4. For this range of σ'_{vc}, the corresponding ξ_R values would only range from -0.28 to -0.31; The change in ξ_R is small because the critical state line is relatively flat over this stress range (Fig. 2). Thus, ξ_R provides a rational explanation for why these results showed no perceptible effect of σ'_{vc} on K_α, although it is noteworthy that the results might have been more sensitive to σ'_{vc} if a larger α value had been used.

8. SUMMARY AND CONCLUSIONS

The effect of overburden stress on liquefaction evaluations was re-evaluated (Boulanger 2003a,b) using a theoretical framework that provided consistency between the different components of the design process. The cone penetration theory of Salgado et al. (1997a), which had been validated against a large set of calibration chamber test data, was used to develop improved C_N relations and to relate the penetration resistance to a relative state parameter index (ξ_R). Published SPT data (Marcuson and Bieganousky 1977a,b) were reanalyzed and shown to be in good agreement with the C_N relations derived for the CPT. Experimental data by Vaid and Sivathayalan (1996) and Vaid and Thomas (1995) on reconstituted sand specimens were used to show that CRR could be approximated as a unique function of ξ_R, thereby capturing the combined effects of D_R and σ_v' on CRR. Relations between CRR and ξ_R for field conditions were subsequently derived from semi-empirical liquefaction correlations, and these CRR-ξ_R relations were used to calculate the effects of σ_v' on predicted CRR. The resulting ξ_R–based relations were compared against commonly used methods and shown to reduce the conservatism that results when certain methods are extrapolated to overburden stresses greater than they were calibrated for. In addition, it was shown that the C_N and K_σ relations are inter-related through the sand properties and state in ways that can have compensating effects on the predicted CRR. Subsequently, the appropriate choice of D_R-independent C_N and K_σ relations (eliminating the need to estimate D_R or ξ_R) can reasonably approximate the effects of σ_v' on predicted CRR. In addition, ξ_R was shown to provide a rational and practical index for describing the variation of K_α with both relative density and confining stress. Modifications to existing practice were recommended that are simple to implement and reduce the excessive conservatism imposed by some of the current approaches.

The application of semi-empirical liquefaction analysis methods to large depths is inherently complicated by the shortage of case histories for such conditions, and thus considerable engineering judgment must be exercised in the procedures being used to extrapolate our shallow-depth experiences. The present study attempts to improve that extrapolation by analyzing the various components of the design process with a consistent theoretical basis, but it nonetheless remains an extrapolation with concurrent unavoidable uncertainties.

9. ACKNOWLEDGMENTS

The author appreciates the many discussions and insights provided by I. M. Idriss throughout this study and the constructive comments of P. Byrne and R. Salgado.

10. REFERENCES

Been, K., and Jefferies, M. G. (1985). "A state parameter for sands." Geotechnique, 35(2), 99-112.

Bolton, M. D. (1986). "The strength and dilatancy of sands." Geotechnique, 36(1), 65-78.

Boulanger, R. W. (2003a). "Relating K_α to a relative state parameter index." J. Geotechnical and Geoenvironmental Engrg., ASCE, in press.

Boulanger, R. W. (2003b). "High overburden stress effects in liquefaction analyses." J. Geotechnical and Geoenvironmental Engrg., ASCE, in press.

Cubrinovski, M., and Ishihara, K. (1999). "Empirical correlation between SPT N-value and relative density for sandy soils." Soils and Foundations, JGS, 39(5), 61-71.

Harder, L. F., Jr., and Boulanger, R. W. (1997). "Application of K_σ and K_α correction factors." Proc. Workshop on Evaluation of Liquefaction Resistance of Soils, NCEER-97-0022, National Center for Earthquake Engineering Research, SUNY Buffalo, N.Y., pp. 167-190.

Hardin, B., and Black, W. (1968). "Shear modulus and damping in soils." J. Soil Mech. and Foundations Div., ASCE, 94(2), 353-369.

Hatanaka, M., Uchida, A., Oh-Oka, H. (1995). "Correlation between the liquefaction strengths of saturated sands obtained by in-situ freezing method and rotary-type triple tube method." Soils and Foundations, JSSMFE, 35(2), 67-75.

Hynes, M. E., and Olsen, R. (1998). "Influence of confining stress on liquefaction resistance." Proc., International Symposium on the Physics and Mechanics of Liquefaction, Balkema, Rotterdam, The Netherlands, 145-152.

Konrad, J.-M. (1988). "Interpretation of flat plate dilatometer tests in sands in terms of the state parameter." Geotechnique, 38(2), 263-277.

Liao, S. C., and Whitman, R. V. (1986). "Overburden correction factors for SPT in sand." J. Geotechnical Engineering, ASCE, 112(3), 373-377.

Marcusson, W. F., III, and Bieganousky, W. A. (1977a). "Laboratory standard penetration tests on fine sands." J. Geotechnical Engineering Division, ASCE, 103(GT6), 565-588.

Marcusson, W. F., III, and Bieganousky, W. A. (1977b). "SPT and relative density in coarse sands." Journal of the Geotechnical Engineering Division, ASCE, 103(GT11), 1295-1309.

Mohamad, R., and Dobry, R. (1986). "Undrained monotonic and cyclic triaxial strength of sand." Journal of Geotechnical Engineering, ASCE, 112(10): 941-958.

Naylor, D. J., Pande, G. N., Simpson, B., and Tabb, R. (1981). *Finite elements in geotechnical engineering*, Pineridge Press, Swansea, U.K.

Robertson, P. K., and Wride, C. E. (1998). "Evaluating cyclic liquefaction potential using the cone penetration test." Canadian Geotechnical Journal, 35(3), 442-459.

Salgado, R., Mitchell, J. K., and Jamiolkowski, M. (1997a). "Cavity expansion and penetration resistance in sands." J. Geotechnical and Geoenvir. Engrg., ASCE, 123(4), 344-354.

Salgado, R., Boulanger, R. W., and Mitchell, J. K. (1997b). "Lateral stress effects on CPT liquefaction resistance correlations." J. Geotechnical and Geoenvironmental Engineering, ASCE, 123(8), 726-735.

Seed, H. B. (1983). "Earthquake resistant design of earth dams." Proc., Symposium on Seismic Design of Embankments and Caverns, Philadelphia, PA, ASCE, pp. 41-64.

Seed, H. B., Lee, K. L., Idriss, I. M., and Makdisi, F. (1973). "Analysis of the slides in the San Fernando Dams during the earthquake of February 9, 1971." EERC 73-2, Earthquake Engineering Research Center, UC Berkeley.

Seed, H. B., Tokimatsu, K., Harder, L. F., and Chung, R. (1985). "Influence of SPT procedures in soil liquefaction resistance evaluations." J. Geotech. Engrg., ASCE, 111(12), 1425-1445.

Seed, R. B, and Harder, L. F., Jr. (1990). "SPT-based analysis of cyclic pore pressure generation and undrained residual strength." Proc., Seed Memorial Symposium, J. M. Duncan, ed., BiTech Publishers, Vancouver, B.C., pp. 351-376.

Skempton, A. W. (1986). "Standard penetration test procedures and the effects in sands of overburden pressure, relative density, particle size, ageing and overconsolidation." Geotechnique, 36(3), 425-447.

Vaid, Y. P., and Finn, W. D. L. (1979). "Static shear and liquefaction potential." J. Geotech. Div., ASCE, 105(GT10), 1233-1246.

Vaid, Y. P., and Chern, J. C. (1985). "Cyclic and monotonic undrained response of saturated sands." Advances in the Art of Testing Soils under Cyclic Conditions, ASCE, N.Y.

Vaid, Y. P., and Thomas, J. (1995). "Liquefaction and postliquefaction behavior of sand." J. Geotechnical Engineering, ASCE, 121(2), 163-173.

Vaid, Y. P., and Sivathayalan, S. (1996). "Static and cyclic liquefaction potential of Fraser Delta sand in simple shear and triaxial tests." Canadian Geotechnical Journal, 33, 281-289.

Yoshimi, Y., Tokimatsu, K., and Hosaka, Y. (1989). "Evaluation of liquefaction resistance of clean sands based on high quality undisturbed samples." Soils and Foundations, JSSMFE, 29(1), 93-104.

Youd, T. L., et al. (2001). "Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils." J. Geotechnical and Geoenvironmental Engineering, ASCE, 127(10), 817-833.

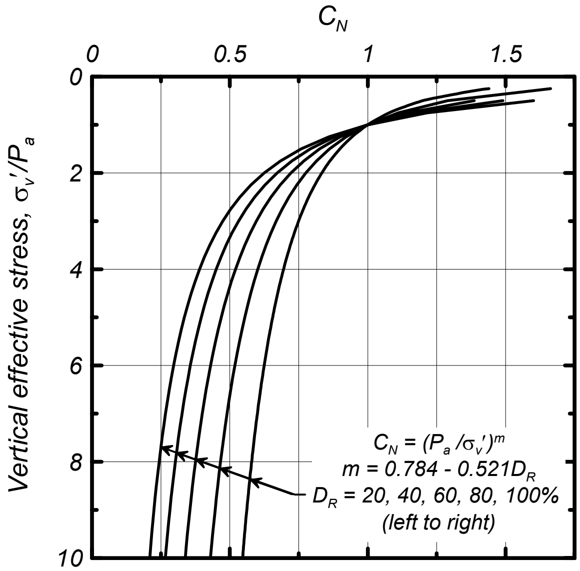

Figure 1. Overburden correction factor (C_N) for clean sands based on Salgado et al. (1997b).

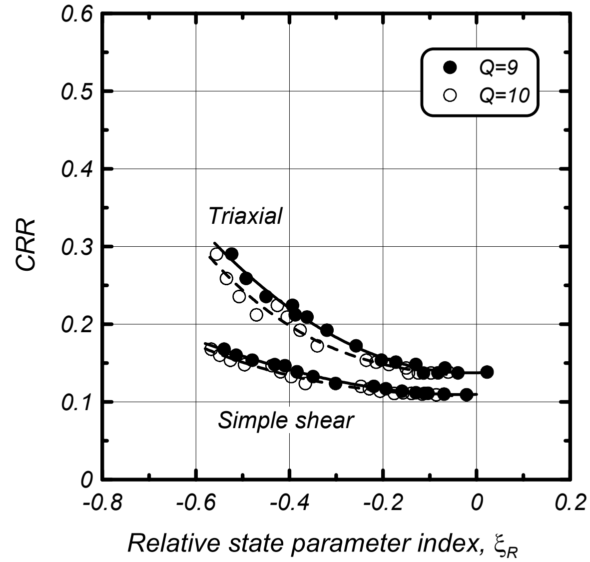

Figure 3. CRR versus ξ_R for reconstituted specimens of Fraser Delta sand: Test data by Vaid & Sivathayalan (1996) for D_R of 31, 40, 59, & 72% and σ_{vo}'/P_a of 0.5, 1, 2, & 4.

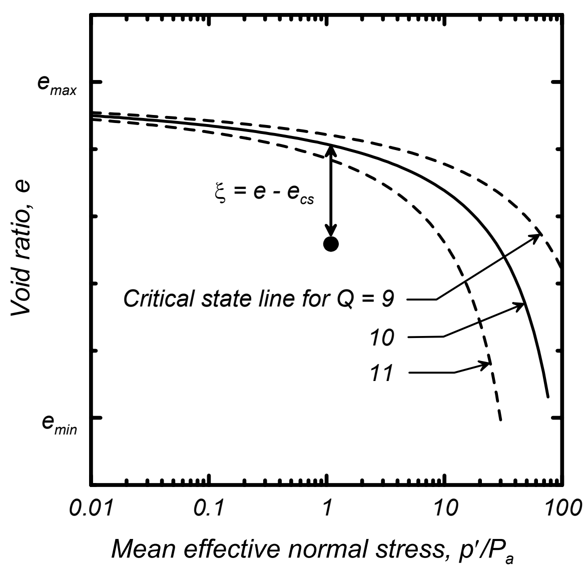

Figure 2. Critical state lines from Bolton's (1986) I_{RD} relation and the definition of state parameter.

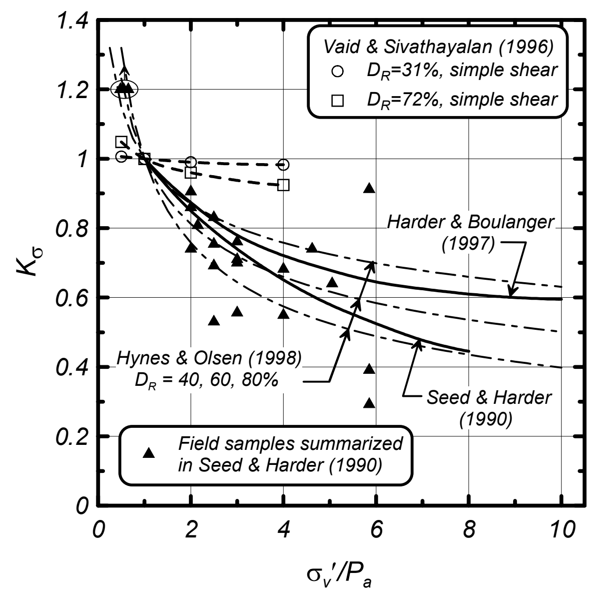

Figure 4. Comparison of K_σ relations with data from reconstituted Fraser Delta sand specimens (Vaid and Sivathayalan 1996) and various field samples (Seed and Harder 1990).

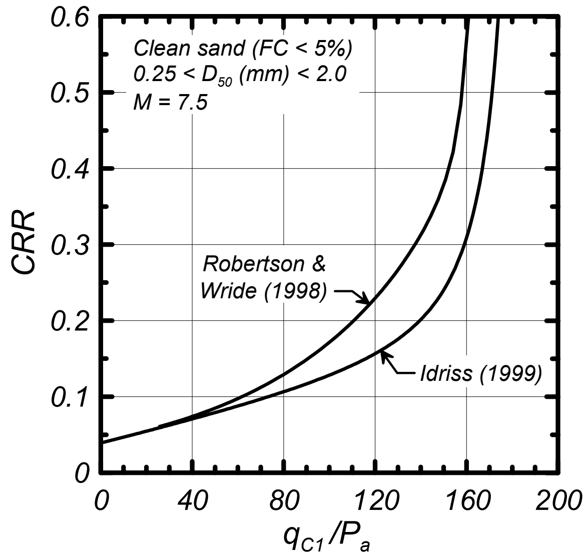

Figure 5. Semi-empirical CPT-based liquefaction Correlations.

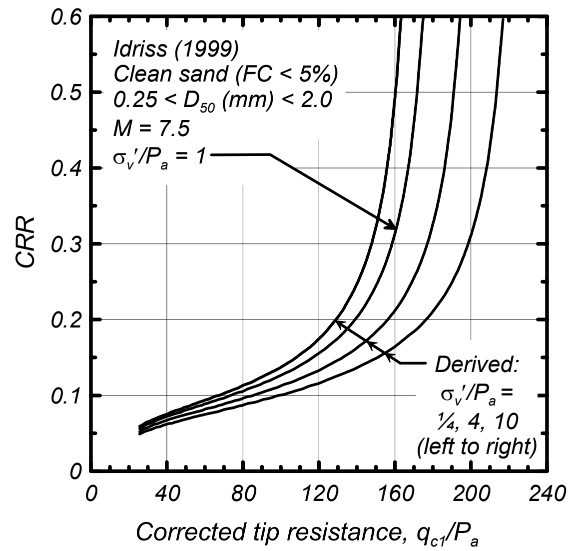

Figure 7. CRR versus q_{c1}/P_a for clean sand with different overburden stresses, using Idriss (1999) correlation for $\sigma_v'/P_a = 1$.

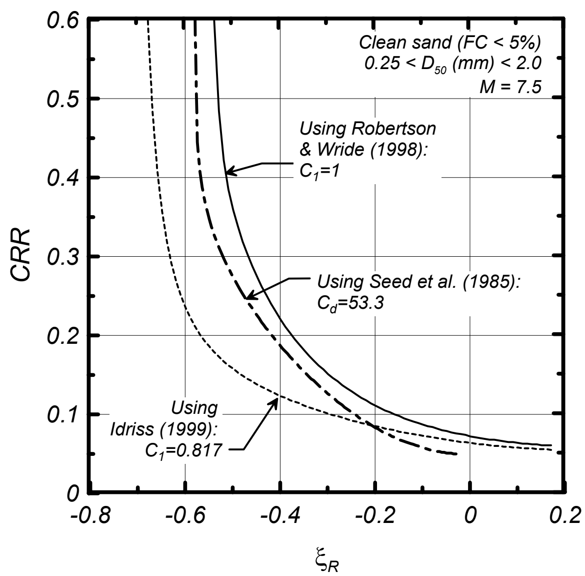

Figure 6. Field CRR-ξ_R relations derived from semi-empirical liquefaction correlations.

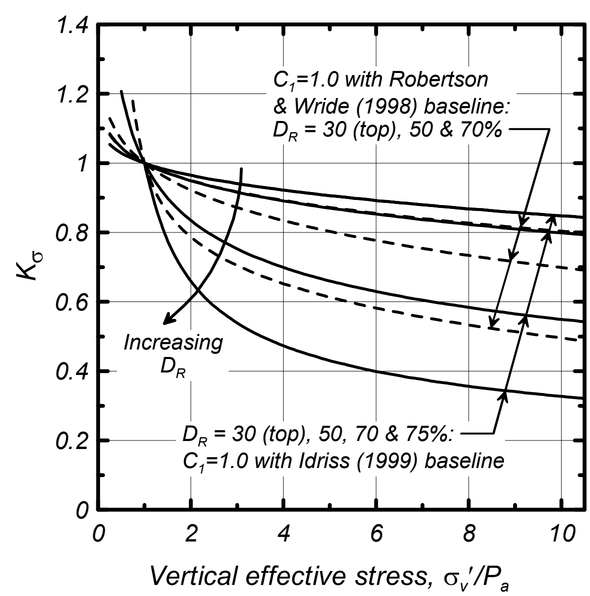

Figure 8. K_σ curves from CRR-ξ_R relations derived from liquefaction correlations for field conditions.

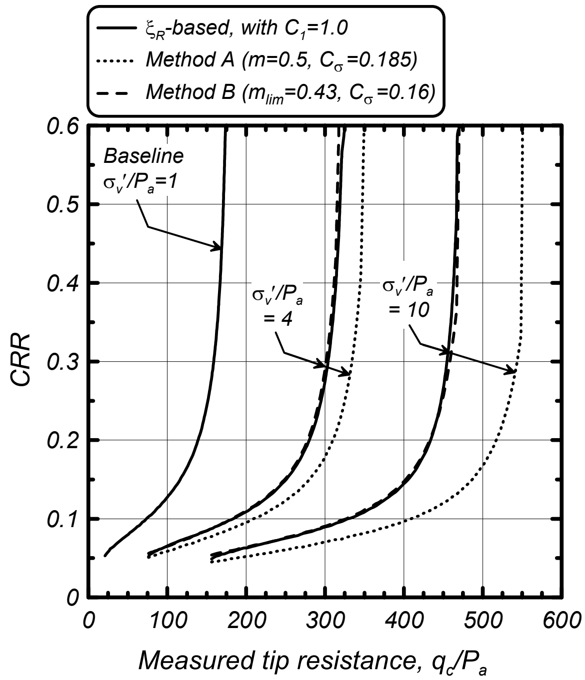

Figure 9. Comparison of ξ_R-based method, its simplified approximation (Method B), and a current method in practice (Method A) using Idriss' (1999) correlation as baseline and C_1=1.0.

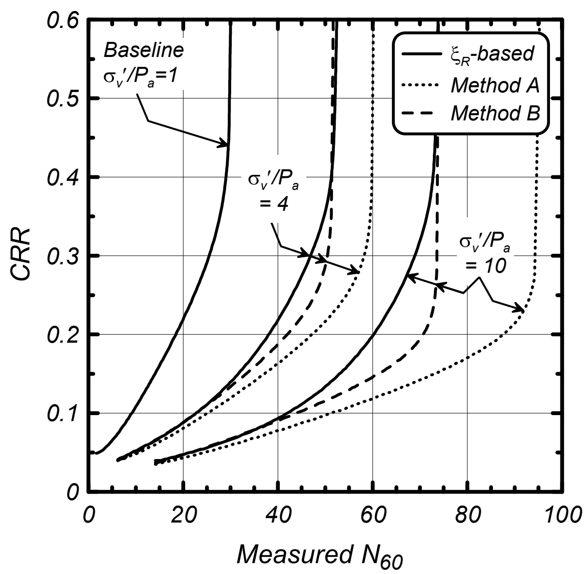

Figure 10. Comparison of ξ_R-based method, its simplified approximation (Method B with m_{lim}=0.39, C_σ=0.21), and a current method in practice (Method A with m=0.5, C_σ=0.185) using Seed et al. (1985) SPT correlation as the baseline and C_d=41.5.

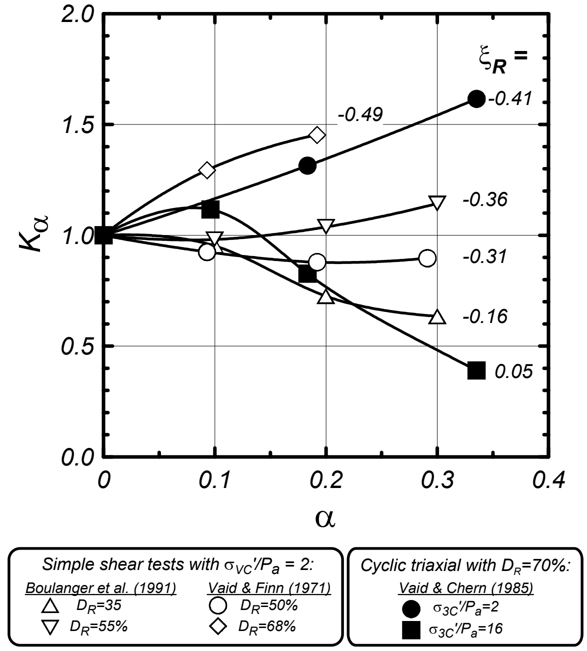

Figure 11. Effect of ξ_R on K_α.