Comparing CPT and $V_s$ Liquefaction Triggering Methods

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Abstract: Significant developments have taken place over the past 20 years to evaluate the liquefaction potential of soils using in situ tests. The cone penetration test (CPT) is now commonly used to evaluate liquefaction potential in soils. There have also been significant developments to evaluate liquefaction potential based on in situ shear wave velocity ($V_s$) measurements. Liquefaction methods based on shear wave velocity have the advantage that they are essentially independent of soil characteristics, such as fines content, but often lack the stratigraphic detail obtained using the CPT. Liquefaction methods based on the CPT have the advantage of continuous, repeatable measurements but require corrections based on soil characteristics that can be significant in soils with high fines content. Comparing the most recent $V_s$-based method with a CPT-based method provides an independent evaluation of the associated corrections applied to the CPT-based method. This paper compares the current $V_s$-based method with a specific CPT-based method from the literature to evaluate the associated CPT-based corrections. The paper also examines the advantage of using both CPT and $V_s$ measurements (e.g., using the seismic CPT) to evaluate liquefaction potential.

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Introduction

Significant developments have taken place over the past 20 years to evaluate the liquefaction potential of soils. The cone penetration test (CPT) is now commonly used to evaluate liquefaction potential in most liquefaction-prone geologic settings. Major developments of CPT-based liquefaction methods have occurred since the early 1980’s (e.g., Seed and Idriss 1981; Shibata and Teparaksa 1988; Suzuki et al. 1995; Robertson and Wride 1998; Moss et al. 2006; Idriss and Boulanger 2014). Liquefaction methods based on the CPT have the advantage of near-continuous, repeatable measurements that provide a detailed profile of the soil. However, CPT-based liquefaction methods require corrections based on soil characteristics that can be significant in sandy soils with high fines content. There have also been significant developments to evaluate liquefaction potential based on in situ shear wave velocity ($V_s$) measurements (e.g., Robertson et al. 1992; Andrus and Stokoe 2000; Kayen et al. 2013). Liquefaction methods based on shear wave velocity have the advantage that they are essentially independent of soil characteristics, such as fines content, but often lack the stratigraphic detail obtained using the CPT. Current CPT and $V_s$ methods to evaluate liquefaction potential are based on a large number of liquefaction case histories (e.g., Boulanger and Idriss 2014; Kayen et al. 2013) that are comprised of very young (Holocene-age) silica-based soils that have no bonding. Kayen (personal communication, 2014) suggested that by comparing the most current $V_s$-based method with a CPT-based method would provide an independent evaluation of the associated ‘fines’ corrections of the CPT-based method.

This paper compares the $V_s$-based liquefaction triggering method suggested by Kayen et al. (2013) with the CPT-based liquefaction triggering method by Robertson and Wride (1998) to evaluate the associated CPT-based corrections. The paper also examines the advantage of using both CPT and $V_s$ measurements (e.g., using the seismic CPT) to evaluate liquefaction potential.

CPT-Based Triggering Method

Robertson and Wride (1998) and updated by Zhang et al. (2002) suggested a normalized cone parameter with a variable stress exponent, n, defined as follows:

$$Q_m = [(q_t - \sigma_{vo})/p_a](\sigma_{vo}/\sigma_{vo}^0)^n$$  \hspace{1cm} (1)

where $q_t$ = measured cone resistance ($q_c$) corrected for water pressure ($q_t - \sigma_{vo}$)/$p_a$ = dimensionless net cone resistance; ($\sigma_{vo}/\sigma_{vo}^0)^n$ = stress normalization factor ($C_n$); $n$ = stress exponent; $p_a$ = atmospheric pressure in same units as $q_t$ and $\sigma_{vo}$; $\sigma_{vo}$ = in-situ total vertical stress; and $\sigma_{vo}^0$ = in-situ effective vertical stress.

Robertson and Wride (1998) and Zhang et al. (2002) used the term, $q_{c1n}$ that was subsequently updated by Robertson (2009) to the more generalized term $Q_m$ used here (where $Q_m$ = $q_{c1n}$).

Zhang et al. (2002) suggested that the stress exponent, n, could be estimated using the normalized Soil Behavior Type (SBTn) Index, $I_s$, used by Robertson and Wride (1998) and that $I_s$ should be defined using $Q_m$. Robertson (2009) suggested an updated method to evaluate the stress exponent, n, based on the following:

$$n = 0.381(I_s) + 0.05(\sigma_{vo}/p_a) - 0.15$$  \hspace{1cm} (2)

where $n \leq 1.0$; $I_s$ = Soil Behavior Type Index = [(3.47 - log Q_m)^2 + (log F_r + 1.22)^2]^{0.5}; $F_r$ = [($f_s$/$q_t - \sigma_{vo}$)]100%; and $f_s$ = CPT sleeve resistance.

Robertson (2009) suggested that the normalization using Eqs. (1) and (2) was based on a constant state parameter. Robertson and Wride (1998) proposed the following CPT-based liquefaction triggering relationship based on case histories, when 50 < $Q_{m,cs}$ < 160:

$$CRR^* = 93(Q_{m,cs}/1.000)^3 + 0.08$$  \hspace{1cm} (3)
CRR* = cyclic resistance ratio, adjusted to moment magnitude $M_w = 7.5$ and $\sigma_{in} = 100$ kPa; and $Q_{m,cs}$ = normalized clean sand equivalent cone resistance, where

$$Q_{m,cs} = K_c Q_m$$  \hspace{1cm} (4)

and

$$K_c = 5.581 I_c^3 - 0.403 I_c^2 - 21.63 I_c + 33.75 I_c - 17.88$$  \hspace{1cm} (5)

The liquefaction case history database is composed of predominately silica-based soils that are (1) very young (Holocene-age), (2) unbonded, (3) have similar geologic depositional environments, and (4) have limited stress history (i.e., essentially normally consolidated with similar in situ stress ratios of $K_s \sim 0.5$). Throughout this paper, the term “young unbonded soils” will be used to refer to soils that are young (Holocene-age) with essentially no bonding (e.g., no cementation).

Robertson and Wride (1998) developed the correction factor ($K_c$) by plotting CPT case history data on the normalized soil behavior type (SBTn) chart suggested by Robertson (1990). The resulting contours of normalized clean sand equivalent cone resistance values, $Q_{m,cs}$, suggested by Robertson and Wride (1998) are shown in Fig. 1. The contours of $Q_{m,cs}$ indicate that two soils on the same $Q_{m,cs}$ contour, but with different CPT measurements (i.e., $Q_m$ and $F_t$), would have the same response to cyclic loading. Robertson (2010a) showed that the contours of $Q_{m,cs}$ are also essentially contours of the state parameter ($\psi$).

The correction factor ($K_c$) to determine $Q_{m,cs}$ can be significant in soils with high fines content (FC). Robertson and Wride (1998) and Robertson (2009) suggested that the CPT-based soil behavior index, $I_c$, was a better indicator of in situ soil behavior than a physical characteristic such as fines content measured on disturbed samples. In soils with high fines content (FC > 35%; $I_c = 2.60$), the correction factor $K_c$ is almost 3.5. This represents a correction of up to 250% on a measured normalized cone resistance ($Q_m$) of 20 in a loose sandy soil with high fines content. A correction of similar magnitude is also applied using the more recent CPT-based methods (e.g., Boulanger and Idriss 2014) when FC = 35% and $Q_m = 20$. The primary cause of these large corrections is the increased large strain compressibility of sandy soils with high fines content, which can significantly reduce the measured cone resistance.

$V_s$-Based Trigger Method

Kayen et al. (2013) presented an updated shear wave velocity ($V_s$) liquefaction trigger relationship based on a global catalog of 422 case histories. The relationship is based on normalized shear wave velocity, $V_s$, which can be defined as

$$V_s = V_s (\rho_s/\sigma_{in})^{0.25} \text{ m/s}$$  \hspace{1cm} (6)

where $V_s = \text{measured shear wave velocity in m/s}$.

The Kayen et al. (2013) liquefaction case history database was composed of many of the same soils as the CPT database and hence are mostly young unbonded silica-based soil. Kayen et al. (2013) also showed that the liquefaction trigger relationship based on $V_s$ is insensitive to soil characteristics, such as fines content (FC). They showed that the boundary shift associated with a fines content adjustment from <5 to 35% has a maximum value of only 5 m/s. This amounts to a maximum correction of 5% when $V_s = 100$ m/s. The reason for the insensitive nature of the $V_s$ liquefaction relationship to fines content is due to the small strain measurement. This adjustment is consistent with previous studies (e.g., Andrus and Stokoe 2000). Kayen et al. (2013) correctly stated that this adjustment is minor in comparison with other aspects of the analysis. Most of the data from sites where liquefaction was observed was for $100 < V_s < 200$ m/s.

CPT–$V_s$ Correlations

CPT cone resistance ($q_t$) is a large strain response that, in sandy soils, is controlled primarily by relative density, effective stress state, stress history, mineralogy, age, and bonding (e.g., cementation). Shear wave ($V_s$), is a small strain response that, in sandy soils, is controlled by the same factors as the cone resistance, but is more sensitive to factors such as age and cementation. Although there is no unique correlation between $q_t$ and $V_s$ (e.g., Rix and Stokoe 1991), it is possible to obtain a good correlation if the relationship and database is limited to soils of similar mineralogy, stress history, age, and cementation (e.g., Andrus et al. 2004).

Based on an extensive database obtained using the seismic CPT (SCPT), Robertson (2009) proposed a generalized relationship for predominately Holocene-age, unbonded silica-based soils linking $V_s$ to CPT normalized cone resistance, $Q_m$, given by

$$V_s = (\alpha_{vs} Q_m)^{0.5} \text{ m/s}$$  \hspace{1cm} (7)

where $\alpha_{vs} = 10^{0.55 (c-1.68)}$; and $V_s$ is in m/s.

The resulting contours of $V_s$, (Robertson 2009), are shown in Fig. 2. The previously mentioned correlation was based on a database of silica-based soils that had similar characteristics (e.g., depositional environment, age, unbonded, little or no stress history) as the soils in the liquefaction case history database.

Fig. 3 shows an example SCPT profile from a site in San Francisco that compares measured to estimated [based on Eq. (7)]:


Comparing Figs. 1 and 2 shows that the contours of $Q_{mc,cs}$ have a similar shape to the contours of $V_s$. This similarity in shape of the contours for $Q_{mc,cs}$ and $V_s$ suggests that the CPT corrections ($K_c$) used to form the contours of $Q_{mc,cs}$ are generally consistent with the $V_s$ liquefaction correlations suggested by Kayen et al. (2013). To evaluate this in more detail, it is possible to link $V_s$ directly with $Q_{mc,cs}$ by combining Eqs. (4) and (7) to get

$$V_s = \left(\frac{Q_{mc,cs} \alpha_v}{K_c}\right)^{0.5}\text{ m/s}$$  \hspace{1cm} (8a)

or

$$Q_{mc,cs} = \left(\frac{K_c}{\alpha_v}\right)(V_s)^2$$  \hspace{1cm} (8b)

Eqs. (8a) and (8b) are only valid for $50 < Q_{mc,cs} < 160$, since they are derived from the liquefaction case history database.

As soils become more compressible (e.g., increasing fines content) and $I_c$ increases, both $K_c$ and $\alpha_v$ also increase and the resulting ratio ($\alpha_v/K_c$) remains almost constant, with an average value of around 360 over the limited range of $50 < Q_{mc,cs} < 160$. Hence, the relationship between the CPT $Q_{mc,cs}$ and $V_s$ is almost constant regardless of fines content. The ratio ($\alpha_v/K_c$) represents the small strain stiffness to strength ratio, similar to $G_0/q_t$. It can be shown that

$$G_0/q_t = \frac{\gamma (V_s)^2}{Q_{mc,cs}} = \frac{(\rho/p_a)(\alpha_v/K_c)}{C_138}$$  \hspace{1cm} (9)

where $G_0 =$ small strain shear modulus, $q_t = (V_s)^2$; $\rho =$ soil mass density $= \gamma/g$; $\gamma =$ soil unit weight; and $g =$ acceleration due to gravity.

The observed average value of ($\alpha_v/K_c$) = 360 for young unbounded soils produces an average $G_0/q_t$ of about 7 that is only valid for $50 < Q_{mc,cs} < 160$. This value of $G_0/q_t$ is consistent with observations made by others (e.g., Rix and Stokoe 1991; Eslaamizaad and Robertson 1996; Schnaid et al. 2004; Schneider and Moss 2011).

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**Fig. 2.** Contours of normalized shear wave velocity, $V_s$, for predominately Holocene-age, unbounded soils (adapted from P. K. Robertson, “Interpretation of cone penetration tests—A unified approach”, Canadian Geotechnical Journal, Vol. 46, No. 11, pp. 151–158).

**Fig. 3.** Example SCPT profile in Holocene-age deposits (San Francisco, CA) comparing measured and CPT estimated $V_s$ profile.
for young, silica-based soils that have no bonding when $Q_{m,cs} < 70$; it follows that the same soils can be contractive and strain softening when $V_{s1} < 160$ m/s. This is consistent with the value of $V_{s1}$ suggested by Robertson et al. (1995) based on laboratory testing of clean fresh (i.e., very young) silica-based sands.

Andrus et al. (2004), suggested an alternate relationship between an equivalent clean sand ($V_{s1}$) and $Q_{m,cs}$ for Holocene-age unbonded sands using

$$ (V_{s1})_{cs} = 62.6(Q_{m,cs})^{0.231} \text{ m/s} \quad (10) $$

As discussed earlier, there is little difference observed between $V_{s1}$ and $(V_{s1})_{cs}$, hence Eq. (10) can also be used to estimate $V_{s1}$. The relationship suggested by Andrus et al. (2004) produces similar values (within 10%) to that given by Eq. (8) in the range of $50 < Q_{m,cs} < 160$ m/s but differs outside this range.

Fig. 4 presents a summary of SCPT data (Robertson 2009) obtained in Holocene-age uncemented deposits from California comparing $V_{s1}$ with $Q_{m,cs}$. The SCPT data was screened using the procedure suggested by Schneider and Moss (2011) to identify Holocene-age, uncemented deposits (details provided later). Fig. 4 illustrates that there is some uncertainty in the correlation and confirms that it is preferred to measure $V_s$ rather than estimate from the CPT. The author is not advocating using the average relationship, represented by Eq. (8), in performing liquefaction triggering evaluation, but rather using the correlation to explain and evaluate the influence of the “fines” correction on the CPT-based liquefaction triggering method.

Research (e.g., Andrus et al. 2007) has shown that any relationship between small-strain shear wave velocity and large-strain cone resistance is also a function of soil age and bonding. Since all the liquefaction case histories that are the basis of both the $V_{s1}$ and CPT trigger relationships are young, essentially normally consolidated, unbonded silica-based soils (e.g., Youd et al. 2001; Boulanger and Idriss 2014), the simplified relationship expressed by Eq. (8) has the same limitation. Youd et al. (2001) suggested that the soils that comprise the liquefaction database are mostly <3,000 years old, and Andrus et al. (2009) suggested an average age of only 23 years.

Comparing CPT-Based and $V_s$-Based Methods

Combining the CPT-based trigger relationship suggested by Robertson and Wride (1998), represented by Eq. (3), and the relationship between $Q_{m,cs}$ and $V_{s1}$ suggested by Robertson (2009), represented by Eq. (8b), produces an equivalent CPT-based $CRR^* - V_{s1}$ relationship, as follows:

$$ CRR^* = 93[(K_c/\alpha_{vs})(V_{s1})^2/1,000]^3 + 0.08 \quad (11) $$

Based on the suggested values by Robertson and Wride (1998) and Robertson (2009), the following range of values for $K_c$ and $\alpha_{vs}$ are obtained:

1. Clean sands (apparent fines content <5%), $I_c = 1.60, \alpha_{vs} = 363.08$ and $K_c = 1.066$; and
2. Silty sands (apparent fines content ~35%), $I_c = 2.60, \alpha_{vs} = 1,288.25$ and $K_c = 3.427$.

Using these values in Eq. (11), the $CRR^*$ values (based on the CPT–$V_s$ correlation) can be compared to the $V_{s1} - CRR^*$ curves proposed by Kayen et al. (2013), Ku et al. (2012), based on an expanded database of liquefaction case histories, showed that the Robertson and Wride (1998) deterministic CPT-based $CRR^*$ relationship has a probability of liquefaction $P_L$ of about 30%. Fig. 5 compares the $CRR^* - V_{s1}$ curves suggested by Kayen et al. (2013) for a $P_L = 30\%$ and the equivalent CPT-based curves derived from Robertson and Wride (1998) using Eq. (11). There is generally good agreement between the CPT-based curve for clean sand ($I_c = 1.6$) and the $V_{s1}$-based clean sand curve (FC < 5%).

The CPT-based curve at high apparent fines content ($I_c = 2.60$) produces $CRR^*$ values that are slightly lower than the Kayen et al. (2013) relationship, especially at high normalized shear wave velocity. This would indicate that the original corrections suggested by Robertson and Wride (1998), based on $I_c$, are somewhat conservative compared to the $V_s$ trigger method of Kayen et al. (2013) and could be adjusted slightly to obtain better agreement.

To provide a better fit with the Kayen et al. (2013), trigger curves the $K_c - I_c$ relationship was modified slightly, as shown in Fig. 6 and the resulting improved agreement shown in Fig. 7. The
“modified” Robertson and Wride (1998) correction (shown in Fig. 6) was derived using trial and error to find an improved match between the CPT-based trigger curves and the \( V_{s1} \)-based curves. Fig. 7 illustrates that a slight modification in the CPT-based \( K_c \) correction can produce a very good match with the \( V_{s1} \)-based trigger curves by Kayen et al. (2013). Figs. 5 and 6 also show that using the original Robertson and Wride (1998) corrections produce slightly conservative lower estimates of CRR* in soils with \( I_c > 1.80 \).

The modified correction factor \( K_c-I_c \) relationship shown in Fig. 6 can be represented by

\[
K_c = 1.7793I_c^3 - 8.4301I_c^2 + 14.386I_c - 7.7282
\]

(12)

That is valid between 1.60 < \( I_c < 2.60 \) and \( K_c = 1.0 \) when \( I_c < 1.60 \).

Any relationship between CPT tip resistance and \( V_s \) has some uncertainty. This uncertainty is reduced when restricted to soils of similar geologic origin and age (e.g., very young Holocene-age, essentially normally consolidated, unbounded, silica-based soils), as used to develop the CPT – \( V_{s1} \) relationship by Robertson (2009) and the liquefaction trigger curves for the CPT and \( V_s \) (Robertson and Wride 1998; Kayen et al. 2013). The relationship to estimate \( V_{s1} \) suggested by Robertson (2009) has an average relative standard error of about 10% (Fig. 4). Fig. 8 illustrates the level of uncertainty in the CRR*–\( V_{s1} \) curves for \( P_L = 30\% \) for clean sand (FC = 5\% and \( I_c = 1.6 \)) based on the relative standard error of ±10\% from the CPT-based estimated \( V_{s1} \).

**Application of Combined CPT and \( V_s \) Measurements to Evaluate Liquefaction Triggering**

One of the advantages of the seismic CPT (SCPT) is that it provides a profile of CPT tip and sleeve resistance, as well as \( V_s \) at the same location in a very cost effective manner (Robertson et al. 1986). The 30-years experience with the SCPT has shown that the \( V_s \) measurements are generally accurate, reliable, and more cost effective than most invasive seismic methods (e.g., cross-hole testing). The added cost of the \( V_s \) measurements is small if CPT is performed at the site. Hence, the SCPT is becoming a popular in situ test (e.g., Mayne 2014) and the author recommends that SCPT be performed, where possible, to measure \( V_s \) along with the CPT measurements.

The above comparison between the \( V_{s1} \)-based method of Kayen et al. (2013) and the CPT-based method of Robertson and Wride
(1998) has shown that both methods will produce very similar results in terms of liquefaction triggering for most loose young, unbounded silica-based sands. The data base of liquefaction case histories are comprised of soils that are essentially normally consolidated with in situ stress ratio \((K_o)\) likely in the range \(0.4 < K_o < 0.7\). Hence, although both CPT measurements and \(V_s\) are influenced by horizontal effective stresses, the application of vertical effective stress in the normalization of both \(q_c\) and \(V_s\) can be effective when \(K_o\) is similar to the case history database (i.e., \(0.4 < K_o < 0.7\)). However, application of the methods for soils where \(K_o\) is significantly larger than around 0.5 can introduce uncertainty, unless a correction for \(K_o\) is applied (e.g., Maki et al. 2014). This can be an issue when applying these liquefaction assessment methods at sites where ground improvement may have increased \(K_o\).

An interesting problem occurs when the CPT-based method predicts triggering of liquefaction, for a given design earthquake loading, but the \(V_s\)-based method does not predict triggering of liquefaction in the same soil for the same design earthquake. Which method should be assumed correct?

Kayen et al. (2013) correctly cautioned that the \(V_s\)-liquefaction correlations requires the cautionary understanding that some soils with unusual soil-specific void ratio–relative density characteristics or bonding may exhibit liquefaction behavior that differs from the generalized proposed relationships.” Essentially Kayen et al. (2013) cautioned that the \(V_s\)-liquefaction correlations may not apply to soils that have “unusual” characteristics. The term microstructure is often used to describe soils that have “unusual” characteristics (Leroueil and Hight 2003) compared to “ideal” soils that have no microstructure. There are several causes for the development of microstructure in soils, such as aging, cementation, cold welding, etc. Most of these factors give soil a strength and stiffness that cannot be accounted for by void ratio and stress history alone. Microstructure tends to reinforce the links between particles, and so that cannot be accounted for by void ratio and stress history alone. Schneider and Moss (2011) showed that \(K_o > 330\), with an average of 215. Hence, using SCPT data, where both \(q_c\) and \(V_s\) measurements are available in the same soil, it is possible to determine if a sandy soil falls within the range of \(110 < K_o < 330\), for young unbounded soil. If a soil has \(K_o > 330\) it can be considered to have “unusual” characteristics (i.e., microstructure) in terms of the application of the liquefaction triggering correlations.

Schneider and Moss (2011) showed that for soils with little or no microstructure (i.e., young Holocene-age, sandy soils with no bonding), \(110 < K_o < 330\), with an average of 215. Hence, using SCPT data, where both \(q_c\) and \(V_s\) measurements are available in the same soil, it is possible to determine if a sandy soil falls within the range of \(110 < K_o < 330\), for young unbounded soil. If a soil has \(K_o > 330\) it can be considered to have “unusual” characteristics (i.e., microstructure) in terms of the application of the liquefaction triggering correlations.

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liquefaction are limited by soil dilation, where correlations based on CPT \(Q_m\) may be more applicable.

Methods to estimate the maximum or limiting shear strain during cyclic loading have shown that at low values of \(Q_{m,cs}\), triggering liquefaction can quickly produce very large strains, whereas at larger values of \(Q_{m,cs}\), triggering liquefaction will produce a slower build-up of strains. Robertson and Wride (1998) showed that when \(Q_{m,cs} < 70\), shear strains quickly become very large (>20%) when liquefaction is triggered. This is consistent with the suggestion by Robertson (2010b) that soils are generally dilative when \(Q_{m,cs} > 70\). When \(Q_{m,cs}\) is less than about 70, the threshold strain has a more significant role. When \(Q_{m,cs} < 70\) and the threshold strain is exceeded, strains can accumulate rapidly leading to liquefaction. When \(Q_{m,cs} > 70\) and the threshold strain is exceeded, strains tend to accumulate more slowly and dilation tends to play an increasing role.

Andrus et al. (2009) and Hayati and Andrus (2009) suggested a method to account for soil aging on the resistance to cyclic loading based on CPT and \(V_s\) results using a measured to estimated velocity ratio (MEVR), where

\[
MEVR = \frac{V_{s,ME}}{V_{s,ME}} \quad (21)
\]

where \(V_{s,ME} = V_s\) measured in situ; and \(V_{s,ES} = V_s\) estimated for a very young unbound soil.

Andrus et al. (2009) suggested using Eq. (10) to calculate \(V_{s,ME}\), based on CPT measurements.

Hayati and Andrus (2009), based on laboratory and field cases, proposed a deposit resistance factor \((K_{DR})\) to correct for age using

\[
K_{DR} = 1.08\text{MEVR} - 0.08 \quad (22)
\]

The age corrected cyclic resistance ratio, \(CRR^{a}_G\) is then given by

\[
CRR^{a}_G = K_{DR}CRR^{c}_G \quad (23)
\]

where \(CRR^{c}_G\) is the CPT-based \(CRR^c\).

Eq. (23) indicates a uniform increase in \(CRR^c\) regardless of in situ density (i.e., \(Q_m\)).

Hayati and Andrus (2009) showed that the MEVR approach is based on a reference age (where \(K_{DR} = MEVR = 1.0\)) of about 23 years for \(V_{s,ME}\) and stated that this reference age “seems a reasonable average for the \(CRR^c\) curves because many liquefaction cases are associated with deposits that were 1–100 years old prior to the earthquake shaking.” Hayati and Andrus (2009) essentially identified that there is a “behavioral age” that could be less than the geologic age, where “behavioral age” is defined as the time since the last critical disturbance and that the measured \(V_s\) was a measure of the “behavioral age”.

A similar approach can be applied to the Schneider and Moss (2011) empirical parameter \(K_G\) using a similar measured to estimated \(K_G\) ratio defined by

\[
MEK_G = K_{G,ME}/K_{G,ES} \quad (24)
\]

where \(K_{G,ME} = K_G\) based on measured values of \(V_s\) and \(q_t\); and \(K_{G,ES} = K_G\) estimated for very young, unbound soil.

Based on the definition of \(K_G\) [Eq. (13)], the MEK ratio is insensitive to changes in CPT \(q_t\) and \(Q_{m,cs}\) due to aging. Since \(q_t\) has been shown to be relatively insensitive to aging and/or light bonding, it is reasonable to assume that

\[
MEK_G = (\text{MEVR})^2 \quad (25)
\]

An equivalent average estimated \(K_{GE} \sim 200\) for a very young (age ∼23 years), unbound soil can be derived from case histories (Table 1) that is slightly lower than the mean of \(K_G = 215\) suggested by Schneider and Moss (2011) for Holocene-age, siliceous unbound sandy soils. Hence, a similar approach can be applied using MEK\(_G\) instead of MEVR and apply Eqs. (22) and (23) to estimate \(CRR^{a}_G\).

To illustrate how these approaches compare, example CPT and \(V_s\) values from published case histories are shown in Table 1. The examples in Table 1 were selected to illustrate possible differences between Holocene-age, uncremented sands; Pleistocene-age and Tertiary-age, uncremented sands; and aged, cemented sands. The soils at the Moss Landing State Beach (Moss landing, CA) site are Holocene-age, uncremented, silica-based sands and are typical of many sites in the current liquefaction database. The State Beach site was described in detail by Boulanger et al. (1997) and was included as an example in Boulanger and Idriss (2014). Liquefaction was observed along the access road to the Moss Landing State Beach during the 1989 magnitude 6.9, Loma Prieta earthquake (Boulanger et al. 1997) where the estimated peak ground acceleration at the site was 0.28 g. UC 15 was located at the Entrance Kiosk where significant liquefaction and large deformations were observed. UC 16 was located nearby on the Beach Path where minor liquefaction was observed and deformations were smaller than at UC 15. UC 18 was located closer to the beach where the sand was denser and no liquefaction was observed. Table 1 shows that the \(CRR^c\) values determined using the CPT-based method of Robertson and Wride (1998) are very similar to the values determined using the \(V_s\)-based method of Kayen et al. (2013). The \(K_G\) values at the State Beach site are consistent with the values suggested by Schneider and Moss (2011) for Holocene-age, uncremented sands (i.e., \(K_G < 330\)) and the MEVR values are close to 1.0, as suggested by Andrus et al (2009). The Moss Landing examples shown in Table 1 illustrate that the CPT-based and \(V_s\)-based liquefaction triggering methods generally provide similar results in most Holocene-age, uncremented, silica-based sands.

Andrus et al. (2009) presented case history data from sand sites in South Carolina that had experienced the 1886 magnitude 7.3, Charleston earthquake, where the estimated average cyclic stress ratio (CSR) at the sites was about 0.25. The sites were of either Pleistocene age or Tertiary age and were estimated to be uncremented. The sand at the James Island site was estimated to have a geologic age of about 100,000 years and did not experience liquefaction in the 1886 earthquake. The TEN-08 site experienced liquefaction during the 1886 earthquake and was estimated to have a behavioral age of only 110 years at the time of the SCPT (since it had liquefied in 1886). The Aiken SRS-5 site did not experience liquefaction and was estimated to have a geologic age of about 35 million years. Table 1 shows that the \(K_G\) values for the aged sands in South Carolina that did not experience liquefaction exceed 330 consistent with the suggestion by Schneider and Moss (2011). For the aged sands at James Island and Aiken, the \(V_s\)-based methods by Kayen et al (2013) and the age adjusted CPT-based method by Andrus et al. (2009) correctly predict that these sands would not liquefy during the 1886 earthquake, whereas the CPT-based method underestimated the \(CRR^c\). The estimated CSR at the threshold strain at the TEN-08 site that had experienced liquefaction in 1886 were measured about 110 years after the liquefaction and may not reflected the state of the soil prior to the earthquake.

Table 1 also includes SCPT data from several sites in Western Australia that are lightly cemented and of late Pleistocene age. These sites have not experienced any major earthquake events and therefore provide no direct liquefaction resistance evidence.
The $K_G$ values are significantly greater than 330 and would suggest some microstructure. Although the sands are of late Pleistocene age, the $K_G$ values are higher than those for the older Tertiary sands from Savannah River, which would support the possibility of cementation. The estimated CRR$^*$ values derived from either $V_s$ or MEVR suggest that these soils would not experience cyclic liquefaction (CRR$^* > 0.7$). However, the CRR$^*$ values derived from the CPT suggest much lower cyclic resistance. In some cases, the estimated CSR to reach the threshold strain (CSR$_{th}$) is greater than the estimated CRR$^*$ from the CPT (e.g., Shenton Park and Perth sands). An uncertainty that exists for sandy soils with light cementation is that if the cyclic loading exceeds the threshold strain (i.e., CSR$^* >$ CSR$_{th}$) there is a risk that the bonding (cementation) could be either damaged or destroyed and the soil may behave more like an uncemented soil at larger strains.

These examples illustrate the importance to identify soils that have microstructure (such as aging and/or cementation) and may exhibit “unusual” characteristics that may make traditional cyclic liquefaction trigger methods (either SPT, CPT, or $V_s$) unreliable. Knowledge of either geologic age or the time since past liquefaction events (i.e., behavioral age) can assist in the liquefaction analysis. A combination of CPT ($q_t$) and $V_s$ measurements in the same soil (e.g., SCPT) provides an opportunity to directly evaluate the potential for microstructure. When combined with knowledge of either geologic age or behavioral age, the SCPT can assist in separating the affects of either age or cementation. If the soils are aged and uncemented, the existing $V_s$-based liquefaction methods suggested by either Andrus et al. (2009) and Kayen et al. (2013) appear to provide better estimates than penetration (either SPT or CPT) liquefaction methods. If the soils have light cementation, the approach suggested by Schneider and Moss (2011) can assist in estimating if the design earthquake loading (CSR$^*$) would exceed the CSR to reach the threshold strain (CSR$_{th}$). If the design earthquake could exceed the threshold strain, there is a risk that the benefits of cementation may be lost and the larger strain CPT-based CRR$^*$ maybe more appropriate.

It would appear that until further research on threshold strain for lightly bonded soils is available, it may be prudent to assume that any benefits from bonding could be destroyed when CSR$^*$ > CSR$_{th}$. For high-risk projects, careful undisturbed sampling combined with laboratory testing may be appropriate to evaluate the influence of possible microstructure. Shear wave velocity measurements can be made both in situ and on samples in the laboratory to evaluate sample disturbance.

It is likely that the benefits from aging could be different than the benefits from bonding (e.g., cementation). Aging and bonding will tend to increase the small strain stiffness but aging may have little influence on the threshold strain, whereas bonding may also increase the threshold strain depending on the nature and degree of bonding. Light bonding may increase the small strain stiffness but have little influence on the larger strain behavior. Clearly there is a need for further research in this area.

The examples shown in Table 1 also confirm the cautionary note provided by Kayen et al. (2013) regarding application of the $V_{ij}$ relationships for soils with “unusual” characteristics. A similar cautionary note should also be applied to existing penetration-based (i.e., CPT and SPT) liquefaction triggering methods. The examples in Table 1 also illustrate how the SCPT can be very helpful in identifying soils with “unusual” characteristics (i.e., microstructure). The SCPT has the advantage that the $V_s$ measurements are obtained at the same location as the CPT measurements in a cost effective way.

### Table 1. Example Sites with CPT and $V_s$

<table>
<thead>
<tr>
<th>Location</th>
<th>CPT Characteristics</th>
<th>SCPT Characteristics</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moss Landing, UC15 (liq)</td>
<td>47 5 128 74 155 5.9</td>
<td>42 147 0.92</td>
<td>Boulanger and Idriss (2014)</td>
</tr>
<tr>
<td>USA, Holocene, uncremented</td>
<td>50 8.6 126 123 189 3.3</td>
<td>643 193 0.24</td>
<td>Idriss (2014)</td>
</tr>
<tr>
<td>Charleston, SC, aged, uncremented</td>
<td>63 15.2 214 192 240 5.4</td>
<td>1087 279 0.14</td>
<td>Andrus et al. (2009)</td>
</tr>
<tr>
<td>TEN-08 (liq)</td>
<td>50 4.4 156 86 188 10</td>
<td>619 281 0.14</td>
<td>Andrus et al. (2009)</td>
</tr>
<tr>
<td>Ledge Point</td>
<td>206 11.1 390 77 326 24.7</td>
<td>1908 643 20.1</td>
<td>Schneider and Lehane (2010)</td>
</tr>
<tr>
<td>Perth Center</td>
<td>206 11.1 390 77 326 24.7</td>
<td>1908 643 20.1</td>
<td>Schneider and Lehane (2010)</td>
</tr>
<tr>
<td>Note: liq = liquefaction; no liq = no liquefaction; RO98 = Robertson and Wride (1998); KE13 = Schneider and Lehane (2010); AET09 = Andrus et al. (2009); SM11 = Schneider and Moss (2011).</td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>
Summary

A comparison has been made between the CPT-based liquefaction triggering method by Robertson and Wride (1998) and the $V_s$-based method by Kayen et al. (2013) to evaluate the associated CPT-based corrections. Although the comparison requires an average relationship between CPT and $V_s$, which has some uncertainty, the comparison has shown that the Robertson and Wride (1998)'s CPT-based corrections, based on a generalized $I_R$ relationship, provided generally good agreement between the two independent approaches. A slight modification to the CPT-based "fines" correction is suggested to provide better agreement between the two methods in soils with high fines content. The comparison indicates that the current Robertson and Wride (1998) corrections are slightly conservative compared to the $V_s$-based trigger relationship of Kayen et al. (2013) in soils with high fines content where $I_R > 1.8$.

The comparison also highlights the importance of recognizing the limits in the existing liquefaction case history database. The existing CPT-based and $V_s$-based methods to evaluate liquefaction triggering apply to "ideal" soils that are young (Holocene-age) and have no significant microstructure, such as bonding and are essentially normally consolidated (i.e., $K_o\sim 0.5$). Kayen et al. (2013) correctly cautioned applying the $V_s$-based method to soils that have "unusual" characteristics. Although an average relationship between CPT and $V_s$ has been shown, it is recommended for soils with "unusual" characteristics, such as aging and/or bonding. The methods suggested by Schneider and Moss (2011) based on the parameter $K_G$, and Hayati and Andrus (2009) based on the MEVR, show promise as simple methods to detect "unusual" characteristics. The approach suggested by Schneider and Moss (2011) has the advantage that a generalized value for $K_G$ (~200) can be assumed that does not require selection of a specific relationship between CPT and $V_s$ with the associated uncertainty. Further research is needed to clarify the role of threshold strain on the response of soils with microstructure (e.g., aging and/or bonding) and if the effects of age are different than cementation on the liquefaction resistance of soil.

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References


