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Limitations of the shear wave velocity as a liquefaction predictor

R. Verdugo[1]

ABSTRACT

The large earthquakes continue producing heavy damages, part of which are associated with failures induced by soil liquefaction. Accordingly, efforts to improve, or reduce uncertainties, of the existing methodologies to predict the liquefaction potential are needed. Obtaining "undisturbed" samples for laboratory testing is a difficult task, especially on saturated loose sandy soils (liquefiable), therefore, there is a consensus in favor of procedures for liquefaction analyses based on field testing. In this scenario, penetration resistances of SPT and CPT are the two most widely used indices for evaluating the onset of liquefaction. Alternatively, the normalized shear wave velocity has been proposed as a field parameter to be used as a liquefaction predictor, however, shear wave velocity measurements are associated with small strain level, making it insensitive to factors as initial fabric, overconsolidation ratio, aging and preshaking. Consequently, a discussion about Vs limitations as a liquefaction predictor is presented.

Introduction

Earthquakes of medium-to-large magnitude have systematically induced liquefaction in areas with sandy soil deposits. Recently, earthquakes in Chile 2010 (Mw = 8.8), Japan 2011 (Mw = 9.0) and New Zealand 2011 (Mw = 6.3) have induced the liquefaction of sands in many areas. As a consequence, these countries had to manage the extensive damage of buildings, ports, dams, routes, lifelines, and bridges, along with the significant human and economic cost related to the occurrence of a seismic event.

The state of the art and practice in geotechnical engineering provide analyses and methodologies to understand liquefaction phenomena, as well as tools to predict the triggering of liquefaction. However, although the phenomenon is reasonably understood, liquefaction is still one of the main sources of the large overall cost caused by earthquakes. Therefore, further efforts to develop new techniques and enhance the existing methodologies for analyzing liquefaction are necessary, using theoretical and practical approaches. These efforts must account for the inherent difficulties faced on daily basis by practitioners and researchers.

The assessment of liquefaction potential of loose saturated sandy soil deposits, soils with the highest liquefaction potential, could be done by retrieving "undisturbed" samples for laboratory tests; however, the completion of laboratory testing on this kind of soils is not always successful. To overcome this situation there is a consensus in favor of field testing procedures that have the advantage of addressing the complexity of soils in their natural, undisturbed in-situ conditions.
In this context, the penetration resistance obtained by either SPT or CPT, are well-accepted field parameters to characterize sandy soils, existing significant correlations with the liquefaction

[1]Civil Engineer, Ph.D., CMGI, Santiago, Chile, rverdugo@cmgi.cl

resistance. Figures 1 and 2 present the most current version of the state-of-practice to estimate the initiation of earthquake-induced liquefaction of sandy soils (Youd et al. 2001).
Alternatively, the normalized shear wave velocity, V_{s1}, has been proposed as a field parameter for liquefaction prediction. The chart using V_{s1} is presented in Figure 3. This chart uses the same framework of liquefaction charts developed based on the liquefaction performance of sites with seismic activities (Dobry et al, 1981a; Robertson et al. 1992; Andrus et al. 1997; 2000; 2004; Dobry 2010).

Considering that the shear wave velocity correlates with the soil density, it is relevant for the dynamic response of sands, and that can be measured in the field in a straightforward way, the V_s-based procedures to evaluate liquefaction resistance are of great interest to geotechnical engineers.

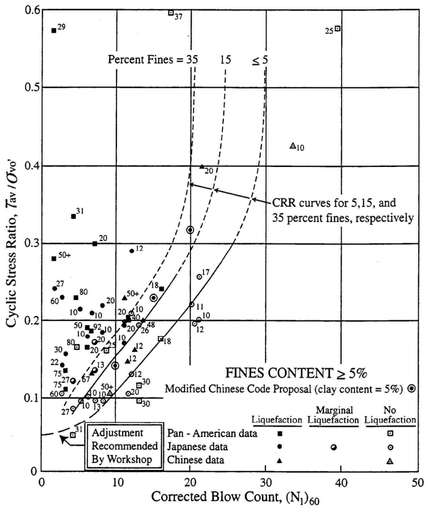

Figure 1. Liquefaction chart based based on SPT- $(N_1)_{60}$. $M_w = 7.5$ (Youd et al., 2001).

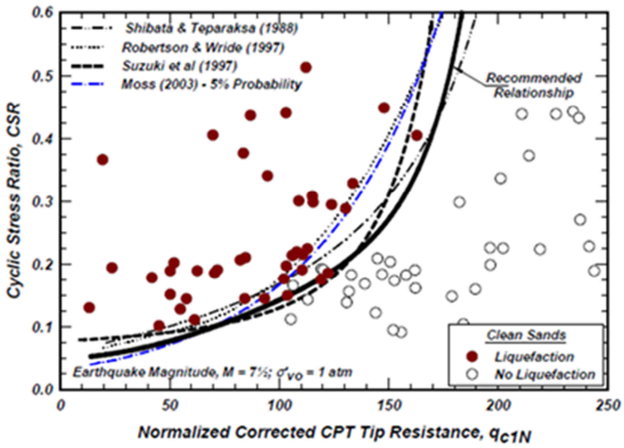

Figure 2. Liquefaction chart based on tip resistance of CPT. $M_w = 7.5$ (Idriss et al. 2004)

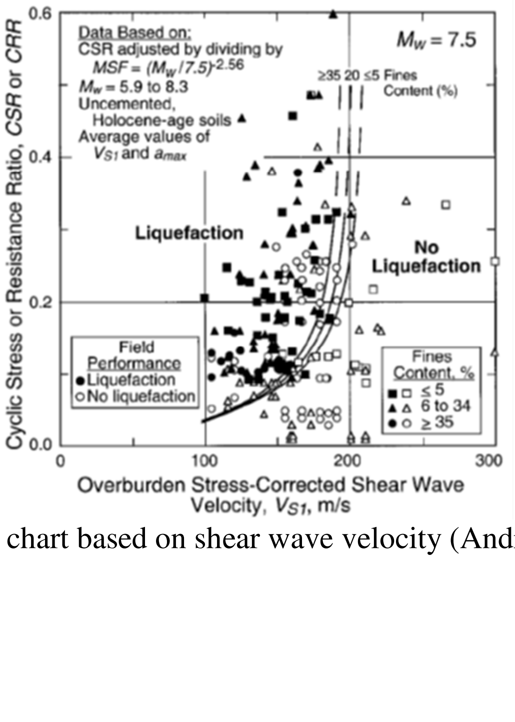

Figure 3. Liquefaction chart based on shear wave velocity (Andrus and Stokoe, 2000)

Despite its appealing features for engineering practice, there is an important concern that arises in the use of V_s as a liquefaction predictor. The shear wave velocity measurements are associated with small strain levels, of the order of 10^{-4} to 10^{-3}%. Therefore, this parameter should not be sensitive to relevant liquefaction parameters such as the initial fabric and overconsolidation ratio (Jamiolkowski et al. 1992; Verdugo, 1992b).

Based on this concern, the present paper discusses the intrinsic limitations of the use of the shear wave velocity as a liquefaction predictor. The paper intends to set a framework for fruitful discussion on how an elastic parameter can determine the triggering of a non-elastic phenomenon.

Shear Strain Levels and Behavior of Sandy Soils

Depending on the shear strain level that an element of sandy soil experiences, the mechanical behavior could be significantly different. For shear strains below 10^{-5} (10^{-3}%), the stress-strain response is fairly linear, as shown by the experimental results obtained by Tatsuoka et al. (1994), and presented in Figure 4. This observation is also supported by the rather limited degradation experienced by the shear modulus of sands in this range of shear strains, as depicted in Figure 5 (Kokusho 1980).

For shear strains greater than 10^{-5} (10^{-3}%), sandy soils show an elasto-plastic behavior, where both permanent and recoverable mechanical strains are observed after unloading. In this scenario, plastic deformations take place, despite no volumetric strain accumulations are observed up to a strain level of the order of 10^{-4} (10^{-2}%). Based on experimental evidence and theoretical considerations, Dobry et al. (1982) introduced the concept of "threshold strain". This parameter separates the cyclic response of the soil with and without volumetric strain accumulations. This concept has been supported by several studies that have provided vast experimental evidence on the existence of this limit strain, below which soils do not present volumetric strain accumulations (Dyvik et al. 1984; Vucetic 1994; Dobry et al. 2011). This singular strain level has been renamed as "volumetric threshold shear strain" to emphasize that this threshold relates to the volumetric strains. Figure 6 shows experimental results supporting the existence of this threshold strain. From these experimental data, shear strains of the order of 10^{-2} % can be identified as a limit shear strain value.

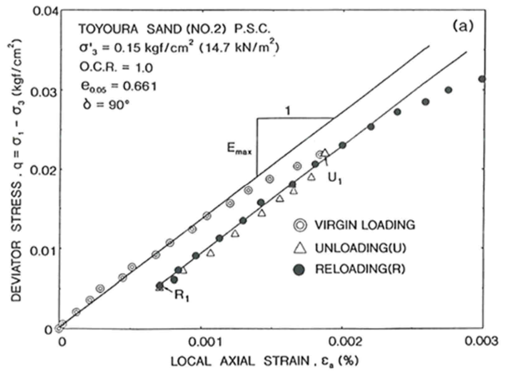

Figure 4. Stress-strain curve with elastic behavior for $\varepsilon \leq 10^{-3}$% ($\gamma \leq 1.3\text{x}10^{-3}$%)

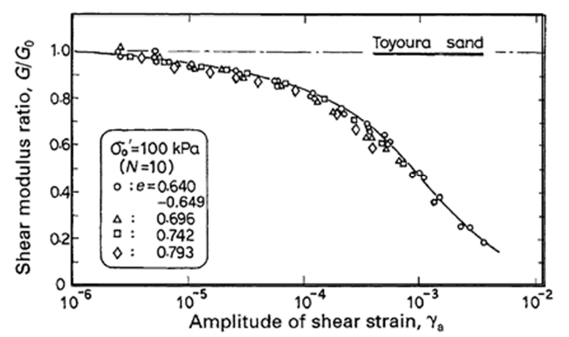

Figure 5. Shear modulus degradation Toyoura sand (Kokusho 1980)

For shear strain levels higher than of the order of 10^{-3} (10^{-1}%), the strain rate effect appears. In this case, the loading speed alters the stiffness as well as the strength of the soil (Ishihara, 1981; 1982). The experimental evidence shows that the strain rate effect is significant in clayey materials and less relevant in sandy soils.

Under cyclic loadings that induce shear strain levels larger than 10^{-2} (1%), the mechanical properties of the soil are significantly affected, as well as the soil experiences noticeable changes with the progression of the cycles. Figure 7 shows an example of this behavior (Towhata, 1982); after each cycle of loading, a clear modification of the stress-strain loop is observed. The magnitude of the changes associated with the progress of cycles is more relevant in loose sandy soils, in which an important rearrangement of particles takes place.

It is not realistic to identify clear and well-defined frontiers, in terms of shear strains, to separate the main features of the behavior of sands. Accordingly, the thresholds described above have to be understood as the transition points from where the mechanical behavior of the soil is gradually modified. Figure 8 shows a summary of the main characteristics of the mechanical behavior of sandy soils, which can be associated with the shear strain level.

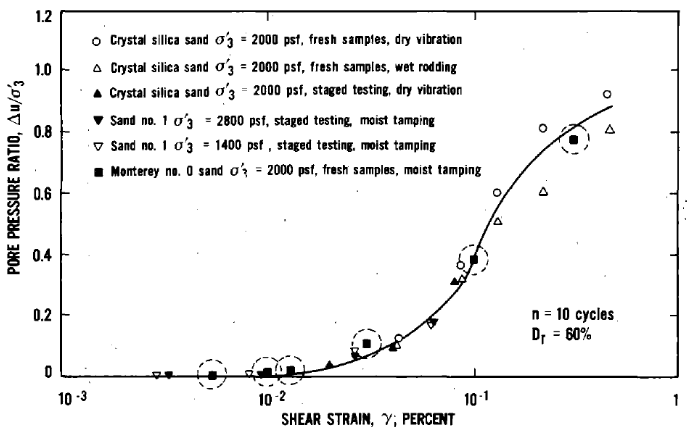

Figure 6. Experimental evidence about the threshold strain (Dobry et al, 1982)

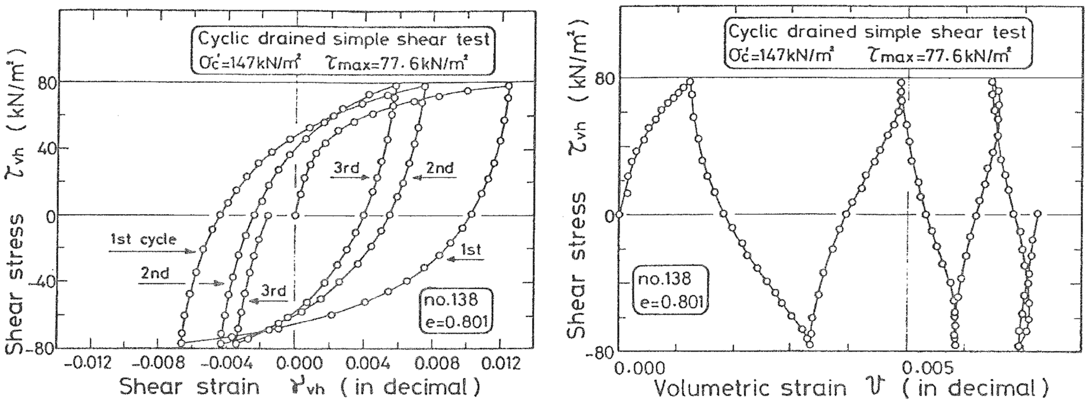

Figure 7. Cyclic soil response for shear strain level larger than 1% (Towhata, 1982)

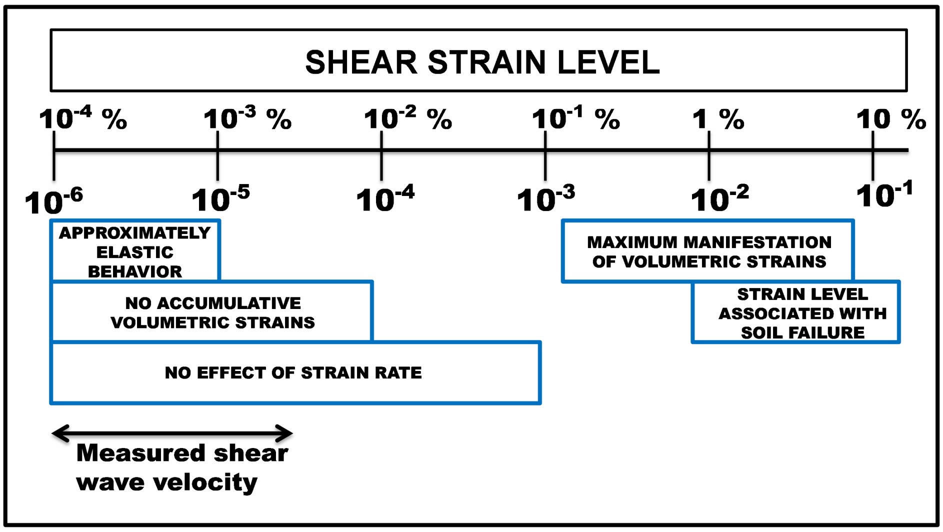

Figure 8. Shear strain level and characteristic behavior of cohesionless soils

On the other hand, measurements of the shear wave velocity are associated to shear strain levels in the range of 10^{-6} to $5x10^{-5}$, where sandy soils do not present volumetric strain accumulation, neither significant plastic deformations. The shear wave velocity is a linear-elastic soil parameter, related to the maximum soil stiffness at a particular state of stresses. In this regard, V_s should not be capable of capturing the potential of volumetric strains of sands, which mainly depends on a combination of soil packing and confining pressure.

Liquefaction Phenomenon

The liquefaction phenomenon is intrinsically related to the natural tendency of loose sands, and low plasticity silty-sands, to experience positive volumetric strains (contraction) when subjected to either monotonic or cyclic loading. When the applied loads are fast enough, as compared to the capacity of soil drainage, the potential volumetric strains are impeded to be developed, and thus they are converted into pore water pressures.

Depending on the field conditions, two scenarios for the occurrence of liquefaction are possible: (1) a flow failure type, in which driving shear forces are larger than the post-liquefaction strength (residual undrained strength), and (2) cyclic softening of level ground.

Loose saturated cohesionless soils may undergo a liquefaction-induced flow failure type, characterized by a sudden loss of strength and the subsequent flow of the soil mass in a short period of time. This kind of failure can be triggered not only by earthquakes but also by disturbances that are quick enough to induce an undrained response (Casagrande 1975; Ishihara 1993; Verdugo et al, 1996, among others). Figure 9 shows the contractive response of a sand tested in undrained conditions. In this test, the initial static deviator stress is greater than the ultimate undrained shear strength. As a consequence, a flow failure is developed. In this test, the observed drop in shear strength starts at a vertical strain level which is greater than 0.5%.

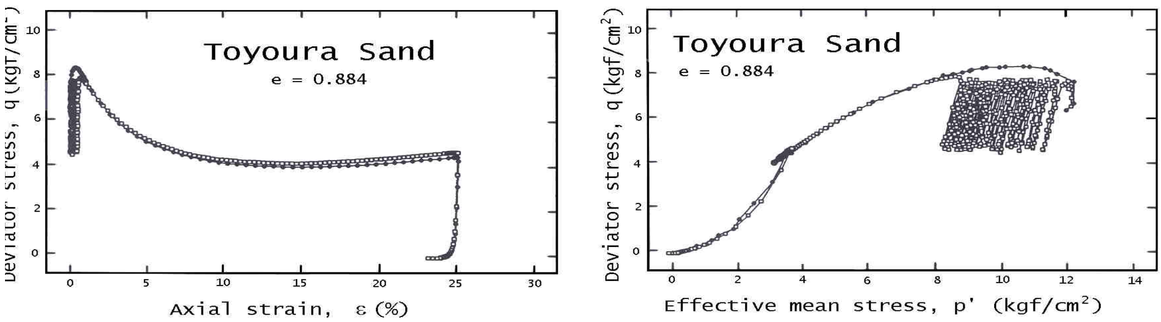

Figure 9. Undrained soil response with strength drop (Verdugo, 1992a)

In the case of the level ground type of failure, loose saturated cohesionless soils subjected to cyclic seismic loadings may experience important pore pressure buildup, causing a systematic reduction of the soil stiffness, or cyclic softening. Additionally, the most common outcome of the large buildup of excess pore pressure is the action of seepage forces that induce upward flow. This flow can transport soil particles to the ground surface, generating the sand boils, in a typical volcano shape. It is important to remark that cyclic softening does not imply strength loss.

The available experimental information indicates that liquefaction resistance is controlled by factors that also have influence on the penetration resistance, which may explain the success of the penetration-based charts for predicting the liquefaction resistance (Dobry et al. 2011).

The SPT Test Features

The SPT blow count provides the penetration resistance of the soil, associated with its failure. Therefore, in this field test, the granular material is forced to mobilize all its available shear strength. The SPT is considered a partially drained test; experimental results obtained using a small tank suggest that the excess pore pressure generated during the SPT depends on the velocity of blow application, as illustrated in Figure 10 (Verdugo et al. 1995). According to these experimental data the SPT N-value tends to reflect the undrained soil response.

Despite the non-negligible deficiencies of the SPT, this field test continues to be significantly used by the geotechnical community around the world. Additionally, due to its application as an index for liquefaction resistance, efforts to improve its standardization have been done. Accordingly, the SPT blow count normalized to an overburden pressure of 1 ton/ft^2 (1.08 kg/cm^2) and a hammer energy ratio of 60%, $(N_1)_{60}$, has been introduced (Seed et al. 1985). Additional corrections include factors for borehole diameter, rod length and sampler with or without a liner (Youd et al. 2001). The SPT-based procedure was the first method empirically developed for predicting the initiation of earthquake-induced liquefaction of sands. It started by Kishida (1966) and Ohsaki (1966) observing the liquefaction-induced failures during the 1964 Niigata Earthquake. The procedure was consolidated by Seed and co-investigators (Seed et al. 1983, 1984, 1985) by analyzing a vast number of actual case histories with and without liquefaction. The SPT-based procedure has been confirmed and improved by several studies, adding case histories provided by recent large earthquakes (Youd et al, 2001; Cetin et al. 2004; Dobry et al. 2011; Boulanger et al, 2012 – 2014, among others).

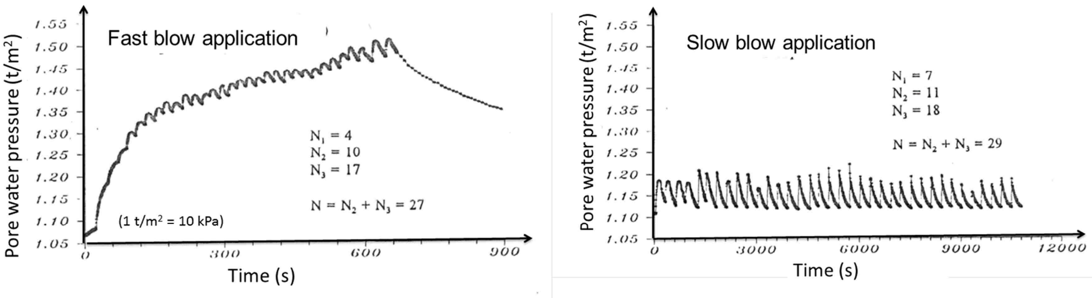

Figure 10. Effect of blow velocity in the excess pore water pressure. SPT

Since the experimental work carried out by Gibbs and Holtz (1957), empirical correlations between the SPT N-value, the vertical effective stress, and the relative density have been proposed (Cubrinovski et al. 1999). A comprehensive study by Skempton (1986) confirmed that the SPT N-value varies with the relative density, D_r, and the vertical effective stress, σ'_v, according to the expression:

$$N = (a + b \cdot \sigma'_V) \cdot D_r^2 \tag{1}$$

where, *a* and *b* are constants for a given kind of sand. These values tend to increase with the grain size, aging, and over-consolidation ratio. The relative density, D_r, is expressed as a ratio (not as a percentage). Considering the energy correction (60%) and the normalization at $\sigma'_v = 1$ kg/cm^2 (≈1 ton/sq ft ≈ 1 atm ≈ 1 bar), the previous relationship becomes:

$$(N_1)_{60} = (a + b) \cdot D_r^2 \tag{2}$$

Therefore:

$$(N_1)_{60} = \frac{a/b + 1}{a/b + \sigma'_V} N_{60} \tag{3}$$

According to the experimental data from Skempton (1986), for normally consolidated sands, the ratio a/b varies roughly between 1 and 2. However, for overconsolidated fine sands, this ratio varies between 0.6 and 0.8. Youd and co-workers recommend to adopt a/b = 1.2, considering the good fit with the original curve specified by Seed and Idriss (1982) for normalizing the SPT N-value to $\sigma'_v = 1$ bar (Kayen et al. 1992; Youd et al. 2001). Therefore, for normally consolidated and overconsolidated sands, the following expressions can be considered:

$$(N_1)_{60} = \frac{2.2}{1.2 + \sigma'_V} N_{60} \quad \text{(Normally Consolidated Sands)} \tag{4}$$

$$(N_1)_{60} = \frac{1.7}{0.7 + \sigma'_V} N_{60} \quad \text{(Overconsolidated Sands)} \tag{5}$$

For normally consolidated natural sandy soil deposits, Skempton (1986) found that the sum *(a+b)*, or the quotient $(N_1)_{60}/D_r^2$, has an average value of around 60, then:

$$(N_1)_{60} \approx 60 \cdot D_r^2 \tag{6}$$

In the case of overconsolidated sandy soil deposits, the SPT N-value is significantly influenced by the horizontal effective stress, which is a function of the OCR. In any case, for heavily overconsolidated sands, K_0 is not greater than one (Jamiolkowski et al. 1988), which results in the following approximation, for overconsolidated sands:

$$(N_1)_{60} \approx 73 \cdot D_r^2 \tag{7}$$

These empirical expressions are the outcome of the following facts: $(N_1)_{60}$ is strongly influenced by the relative density and the ground stress history, and $(N_1)_{60}$ correlates with the soil shear strength. Consequently, the use of $(N_1)_{60}$ as a liquefaction predictor makes sense.

The CPT Test Features

The use of CPT and its popularity in geotechnical engineering practice have grown all around the world due to the significant amount of research. These works have encouraged a significant progress in the electronic tools as well as in the development of semi-empirical methodologies to estimate different soil parameters. The CPT has several advantages over the SPT. For example, the CPT provides nearly continuous data, it is well normalized, and it produces repeatable test results. It is widely recognized that in sandy soils with low fines contents, the cone penetration obtained at the standard rate of 2 cm/s generates a drained soil response. Therefore, at the standard velocity of penetration, CPT reflects the mobilized drained strength of sandy soils, according to their in-situ state of stresses and packing.

The CPT-based procedure to evaluate the liquefaction resistance of sands was developed by replacing the corrected standard penetration resistance $(N_1)_{60}$ by the corrected tip resistance q_{1c} (Stark et al, 1995; Robertson et al, 1998; Youd et al, 2001; Suzuki et al, 2003; Idriss et al, 2004). Basically using calibration chamber tests, a relationship between CPT tip resistance, q_c, vertical effective stress and relative density has been developed (Schmertmann, J.H. 1978; Lunne et al. 1983; Baldi et al. 1986; Jamiolkowski et al. 1988, among others). However, it has been pointed out that factors such as sand compressibility, age, and stress history may affect this type of correlations, making them not unique (Robertson et al. 1983; Bellotti et al. 1989). For normally consolidated, unaged and uncemented sandy soil deposits, the following expression has been proposed:

$$q_c = c_O \cdot P_a \cdot \left(\frac{\sigma_V{}'}{P_a} \right)^{C_1} \exp^{(C_2 \cdot Dr)} \tag{8}$$

where, c_o, c_1 and c_2, are empirical non-dimensional coefficients. P_a is the atmospheric pressure expressed in the same unit of the vertical stress and tip penetration resistance. Relative density is expressed as a fraction of the unity.

Analogously, a normalized tip resistance, q_{c1}, at $\sigma'_v = 1$ atmosphere is defined:

$$q_{c1} = c_O \cdot P_a \cdot \exp^{(C_2 \cdot Dr)} \tag{9}$$

Therefore:

$$q_{c1} = \frac{q_C}{\left(\frac{\sigma_V'}{P_a}\right)^{C_1}} \tag{10}$$

A comprehensive investigation performed by Jamiolkowski and co-workers (Jamiolkowski et al. 2001) using silica sands (Ticino, Toyoura and Hokksund sands), permitted the establishment of the following relationship between CPT tip resistance, q_c, vertical effective stress and relative density, for normally consolidated, unaged sands:

$$q_c = 17.68 \cdot P_a \cdot \left(\frac{\sigma_V'}{P_a}\right)^{0.5} \exp^{(3.1 \cdot Dr)} \tag{11}$$

Then, for $\sigma'_v = 1$ atm (≈ 1 kg/cm^2):

$$q_{c1} = 17.68 \cdot P_a \cdot \exp^{(3.1 \cdot Dr)} \tag{12}$$

In the case of overconsolidated sands, Jamiolkowski and co-workers proposed to replace the vertical stress by the mean stress, σ'_m, and assume K_0=1. For overconsolidated silica sands, the previous relationship becomes (Jamiolkowski et al. 2001):

$$q_c = 24.94 \cdot P_a \cdot \left(\frac{\sigma_V'}{P_a}\right)^{0.46} \exp^{(2.96 \cdot Dr)} \tag{13}$$

These empirical expressions show that the CPT tip resistance is strongly influenced by the relative density, and also by the stress history of the soil. Also, the CPT tip resistance correlates with the drained shear strength. These facts give the conceptual support for using the CPT tip resistance as a liquefaction predictor.

Shear Wave Velocity

The shear wave velocity, V_s, measured either in the field or the laboratory, is an important material property that is directly related to the soil stiffness at small strain level. In the field, V_s can be measured by different methods such as down-hole, cross-hole, suspension logging and surface wave methods. In the laboratory, it can be measured using resonant column tests, bender elements, and compression tests implemented with local strain transducers. Due to the existing methods for measuring V_s, this property is especially attractive for characterizing soils that are difficult to sample, like saturated loose sandy materials. This real advantage is likely the most important attribute that has promoted the use of V_s to predict liquefaction potential.

Experimental results have shown that V_s is a function of the principal stresses acting in the directions of wave propagation and particle motion, being insensitive to the out-of-plane principal stress (Roesler 1979; Stokoe et al. 1985; Belloti et al. 1996; among others). Based on empirical evidence, V_s is given by:

$$V_S = A \cdot F(e) \cdot \left(\frac{\sigma'_a}{P_a} \right)^m \cdot \left(\frac{\sigma'_b}{P_a} \right)^n \tag{14}$$

where, A is a soil property parameter, in units of velocity. F(e) is the void ratio function, while σ'_a and σ'_b represent the principal effective stresses in the direction of the wave propagation and particle motion, respectively. P_a is the atmospheric pressure expressed in the same units as σ'_a and σ'_b. The parameters *n* and *m* are dimensionless exponents.

When V_s is measured for a condition of either vertical wave propagation or vertical particle motion, the vertical and effective horizontal stresses can be associated with σ_a and σ_b, respectively. Additionally, the horizontal and vertical effective stresses are related through the coefficient of earth pressure at rest, K_o. Without loss of generality in the analysis, according to reported data, the values of m and n can be fixed equal to 0.125. Thus, the expression of V_s becomes:

$$V_S = A \cdot F(e) \cdot K_O^{0.125} \cdot \left(\frac{\sigma'_V}{P_a} \right)^{0.25} \tag{15}$$

Introducing the normalized shear wave velocity, V_{s1}, which is associated with a vertical effective stress σ'_v = Pa = 1 kg/cm^2, the following expression is obtained:

$$V_{S1} = A \cdot F(e) \cdot K_O^{0.125} \tag{16}$$

Therefore,

$$V_{S1} = V_S \left(\frac{P_a}{\sigma'_V} \right)^{0.25} \tag{17}$$

The chart used for liquefaction evaluation, based on the shear wave velocity, uses the normalized shear wave velocity, V_{s1}. Its philosophy follows the empirical approach of both the SPT and the CPT-based procedures used to evaluate the earthquake-induced liquefaction of sands.

Implications of Dependency of Shear Wave Velocity on Void Ratio

The pioneer experimental work carried out by Hardin and Richart (1963), using different gradations of Ottawa sand, concluded that V_s decreases linearly with increasing void ratio. Figure 11 shows the experimental data from Hardin and Richart (1963). At the bottom of the plot, the intervals between e_{max} and e_{min} of each gradation have been added as shown in this

Figure. The V_s resulted to be independent of the grain size, gradation, and relative density of the sand. This feature is critical, and it must be analyzed in greater depth due to the impact it may have on the real capability of V_s as a predictor of liquefaction.

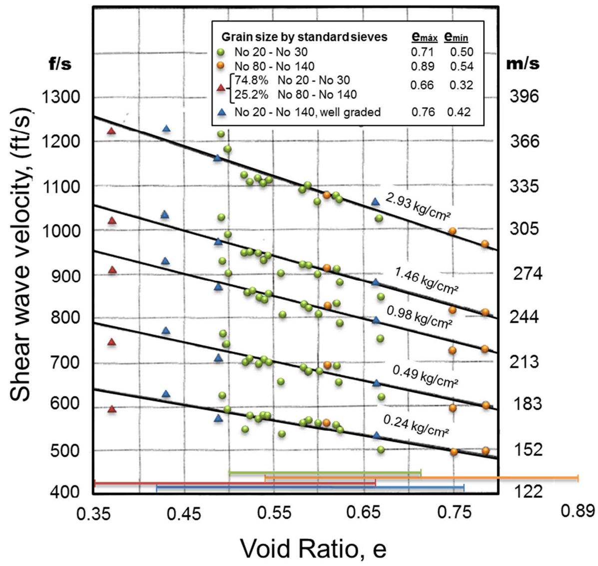

Figure 11. Shear wave velocity vs void ratio (modified from Hardin et al. 1963)

From Figures 11 and 12a, it seems that V_s is a unique function of the void ratio, but not a unique function of relative density. This observation is confirmed in Figure 12b, where the same original data of $V_{s,}$ for a confining pressure of 0.98 kg/cm^2 (2000 psf), have been re-plotted in terms of relative density. It is observed that the single relationship governed by the void ratio is divided into new relationships for each sample of Ottawa sand.

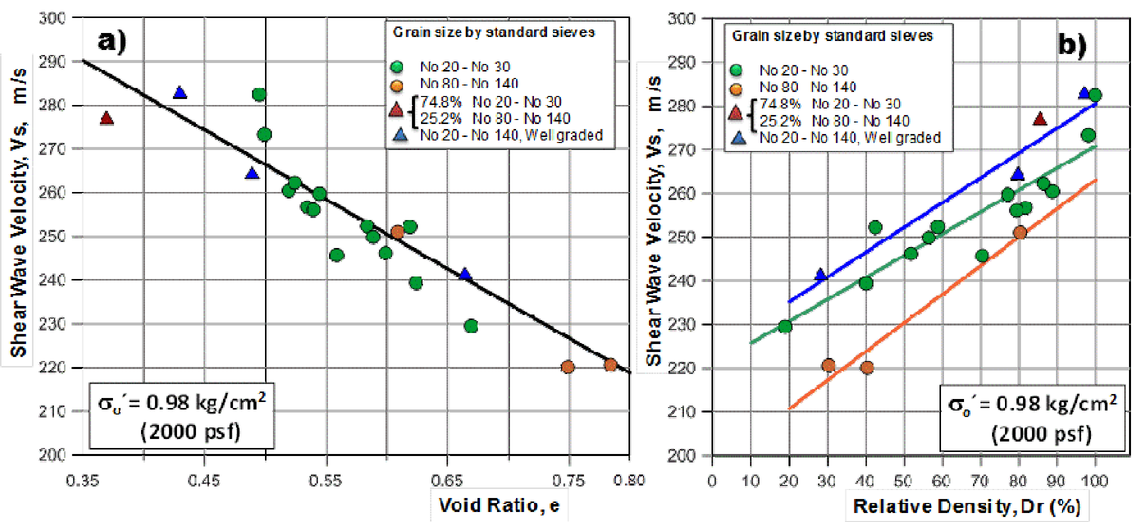

Figure 12. Shear wave velocity as function of a) void ratio and b) relative density (data from Hardin et al. 1963)

It is important to pay attention to the enormous range of relative density that result in the same V_s (Figure 12b). For example, Ottawa sand No. 20 - No. 140, at a relative density of 30%, has a V_s around 240 m/s, and Ottawa sand No. 80 - No. 140, at a relative density of 64%, has similar V_s around 240 m/s.

All the experimental evidence reported consistently indicates that V_s is a function of the void ratio, being considered in the void ratio function, *F(e)*, previously introduced. The experimental data show that V_s is not especially affected when is close to the maximum and minimum void ratios (see Figure 11). This implies that V_s is unable to discriminate whether the soil packing is dense or loose. This inherent feature of V_s generates a limitation for the use of this parameter in analyzes where the soil response is strongly dependent on the soil packing, as the case of liquefaction phenomenon.

Effect of Overconsolidation on the Shear Wave Velocity

Bender element tests were carried out on two types of sand to evaluate the impact of the mechanical overconsolidation on the shear wave velocity of sands. The first sand was from Sweden, denominated Sand-S, while the other was from Chile, denominated Sand-C. While Sand-S is a natural sand, Sand-C is a copper tailings material retrieved from a tailings dam and washed on sieve #200 (ASTM) to eliminate any fine content. Table 1 presents the main physical properties of the sands investigated.

The shear wave velocity measurements of sand-S were carried out by the author in the geotechnical laboratory of the Norwegian Geotechnical Institute, in 1996. The tests were performed on sands deposited in a consolidation cell equipped with bender elements (Dyvik et al. 1985).

Table 1. Physical properties of tested sands

Sand	**Grain shape**	**D_{50} (mm)**	**FC (%)**	**G_s**	**e_{max}**	**e_{min}**
Sand S	Subangular to angular	0.15	7	2.60	0.862	0.505
Sand C	Angular	0.15	0	2.72	1.147	0.664

FC: Fines content; Gs: Specific gravity

The specimens were vertically loaded at a stress of 0.5 kg/cm^2 and then saturated. Afterwards, vertical pressures of 1, 2, 4, and 8 kg/cm^2 were applied on the sand specimens. A subsequent unloading process was performed, decreasing the load from 8 kg/cm^2, in steps of 0.5 kg/cm^2, to generate over-consolidation ratios varying from 1 to 16. The shear wave velocity was measured at each state of stress induced by the loads.

The shear wave velocities of sand-C were measured in the geotechnical laboratory of University of Chile (Sanchez, 2002). The sandy soil specimens were prepared in triaxial cells equipped with bender elements. The samples, 10 cm in high and 5 cm in diameter, were saturated (B-value greater than 0.95) and isotropically consolidated at effective confining pressures of 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5 and 4.0 kg/cm^2. Afterwards, the specimens were unloaded, following the same steps. At each effective confinement, for both loading and unloading, the shear wave velocities of the specimens were measured

Figure 13 shows the linear plot of the shear wave velocities as a function of the void ratio, for various vertical stress levels (0.5, 1, 2, 4, and 8 kg/cm^2), measured on Sand-S specimens during the loading and unloading stages.

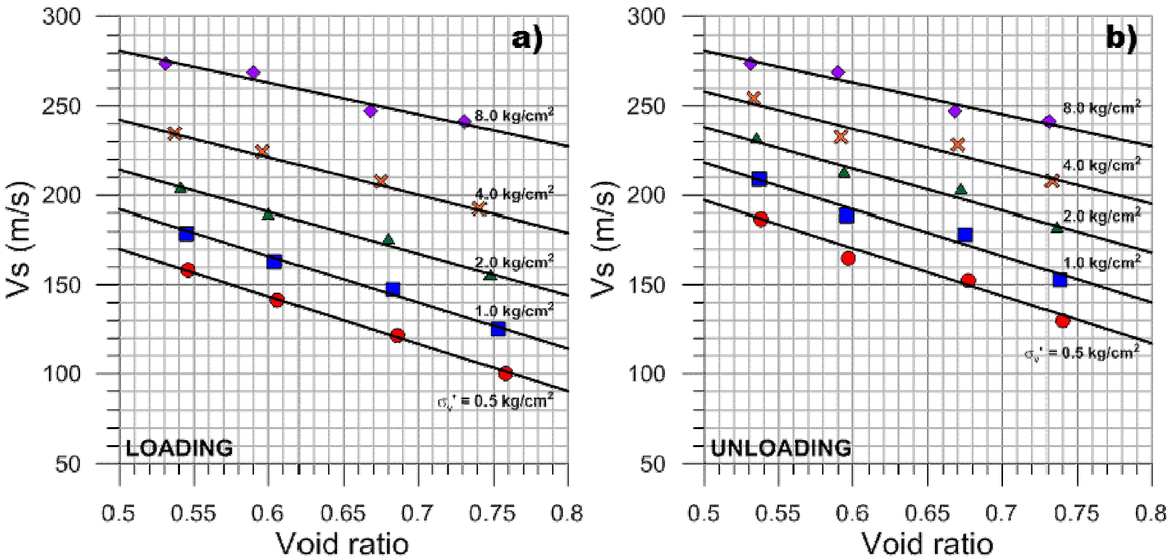

Figure 13. Shear wave velocities measured during a) loading and b) unloading. Sand S.

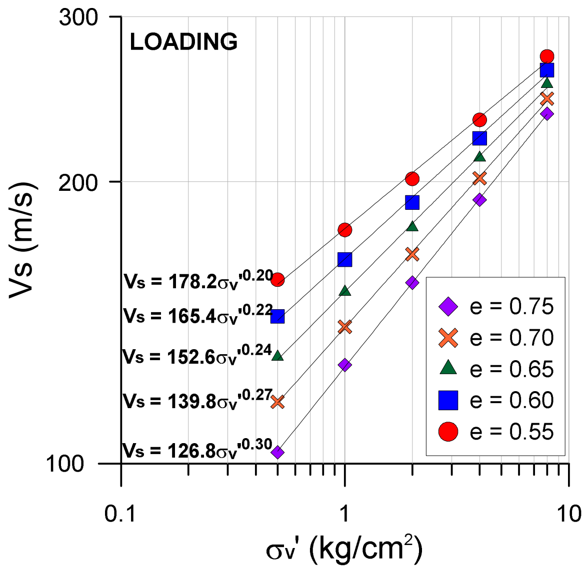

Figure 14. V_s as function of the vertical pressure for given void ratios. Sand S.

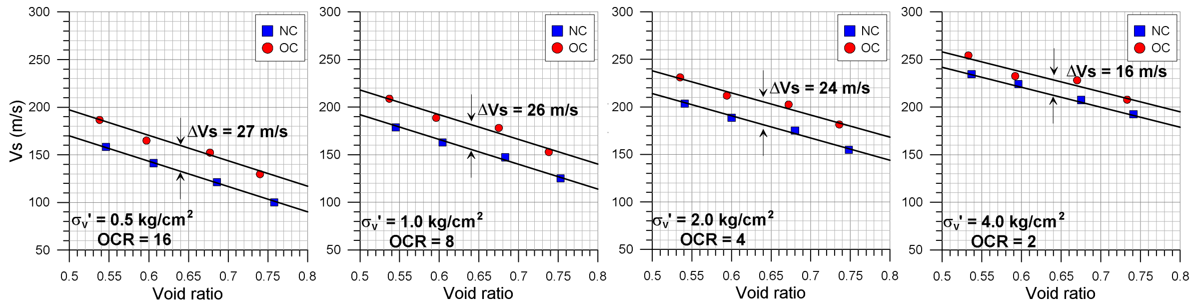

Figure 15. V_s measured at different vertical stresses and overconsolidation ratios. Sand S.

For both stages of loading and unloading of the specimens, the plots in Figures 13a and 13b show that (1) V_s increase with the confinement and that (2) V_s decreases as the void ratio increases, regardless of the confinement. These observations are in agreement with previous studies that highlighted the effect of both the confinement and void ratio on the shear modulus. From Figure 13a, the values of V_s were obtained at various effective confining pressures and at given void ratios. The results are plotted in Figure 14, with both the V_s and the effective confinement in logarithmic scale. A power regression of the data confirms the power relation between the shear wave velocity and the effective confinement of the granular material. These results suggest that the exponent of the vertical pressure increases as the void ratio increases, with the values ranging between 0.2 and 0.3, and an average value around 0.25.

Figure 15 presents the linear plots of the shear wave velocity as a function of the void ratio, for normally consolidated and overconsolidated specimens of Sand-S, at different effective confinements and different overconsolidation ratios.

In addition to the previous observation related to the effect of the confinement, and the void ratio, on the shear wave velocity, it is observed that, regardless of the effective confinement and the overconsolidation ratio, the shear wave velocity of overconsolidated specimens is higher than the shear wave velocity of normally consolidated specimens. Also, at a given OCR and vertical pressure, the difference between the shear wave velocity of overconsolidated specimens and normally consolidated specimens is fairly constant, regardless of the sample void ratio.

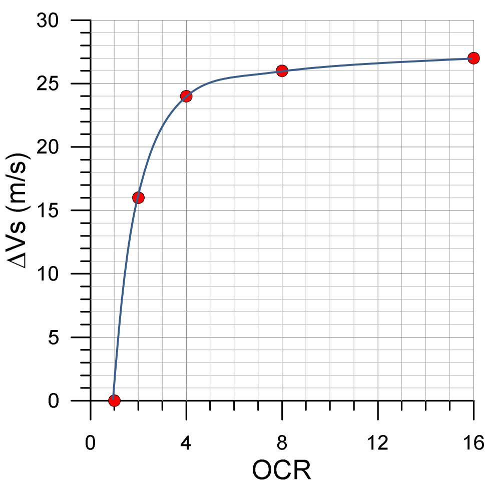

Figure 16. Increment of shear wave velocity due to overconsolidation. Sand S.

Figure 16 shows the results in terms of the increment of the shear wave velocity at different overconsolidation ratio. It is observed that the shear wave velocity increases with the overconsolidated ratio. The plot of this increase, however, suggests that the increment of the shear wave velocity rapidly reaches a plateau for overconsolidation ratios higher than 8. In this particular case, for overconsolidation ratios of over 8, the increment would be less than 30 m/s.

For Sand-C, a similar interpretation of the results presented in Figs. 17 to 20 can be done. In the case of the effect of overconsolidation, the trends follow a similar pattern of Sand-S, with V_s marginally increasing; less than 15 m/s for overconsolidation ratios higher than 4.

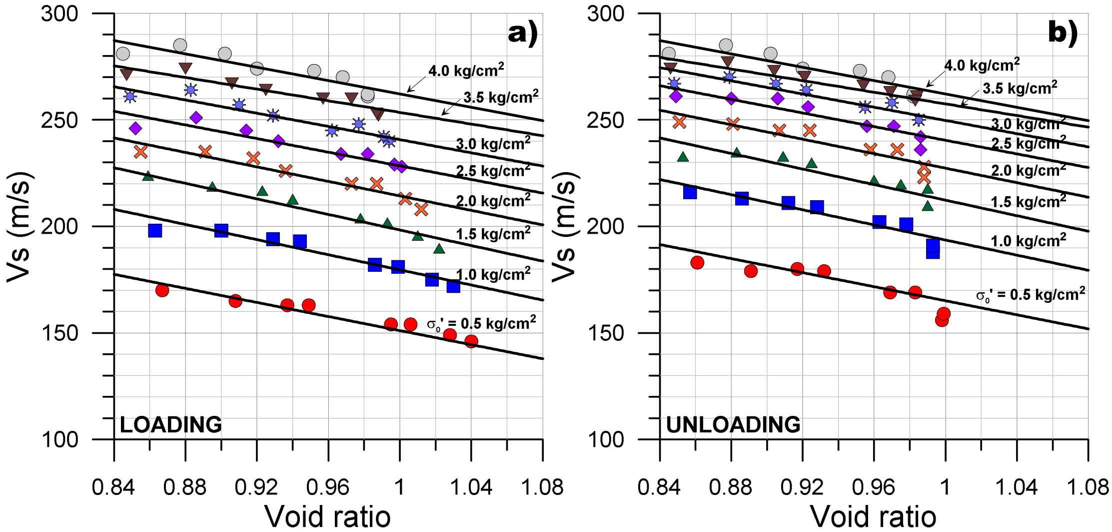

Figure 17. Shear wave velocities measured during loading and unloading. Sand C.

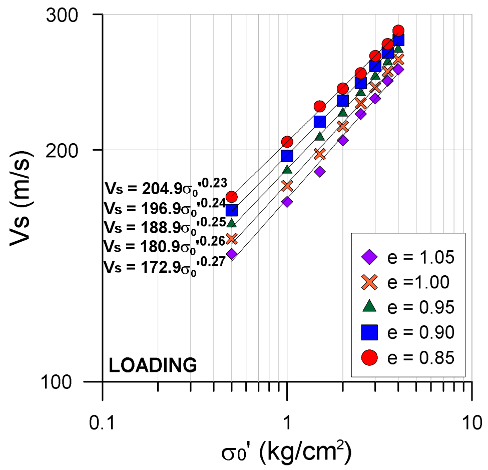

Figure 18. V_s as function of confining pressure for given void ratios. Sand C.

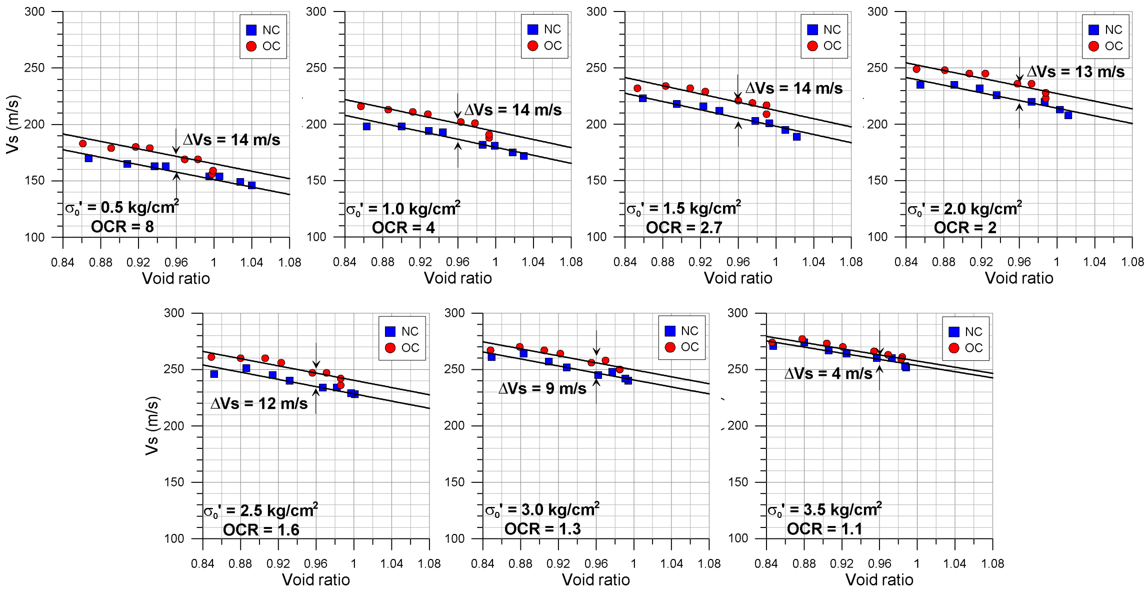

Figure 19. V_s measured at different confining pressure and over consolidation ratios. Sand C.

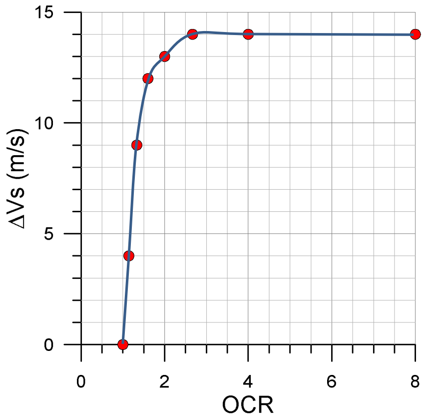

Figure 20. Increment of shear wave velocity due to overconsolidation. Sand C.

The results presented above provide consistent evidence regarding the little sensitivity of V_s to changes in the overconsolidation ratio. This observation disagrees with the consistent evidence suggesting that the liquefaction resistance of sands significantly increases with the rise in the overconsolidation ratio.

Effect of Overconsolidation on the Cyclic Strength

An experimental program that considered the performance of undrained cyclic triaxial tests was carried out on both normally consolidated (NC) and overconsolidated (OC) specimens of Sand-C (Sanchez, 2002). These specimens were prepared initially with relative densities of the order of 67%. The NC specimens were isotropically consolidated at a confining effective stress of 1 kg/cm^2. The OC specimens were isotropically loaded to a confining pressure of 6 kg/cm^2, initially. Afterwards, the specimens were unloaded at an isotropic effective stress of 1 kg/cm^2. Therefore, the cyclic triaxial tests on these specimens were performed with an overconsolidation ratio of 6.

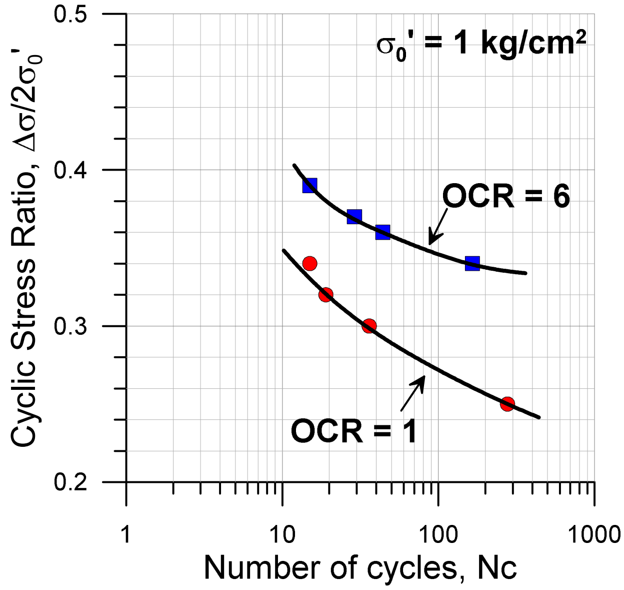

Figure 21. Cyclic strength in normally and overconsolidated samples of Sand-C.

Figure 21 presents the plot of the cyclic stress ratio, in linear scale, as a function of the number of cycles to liquefaction, in logarithmic scale. The liquefaction criterion used in these tests was based on deformations, as the number of cycles at which the axial deformation of the sand specimens reached a 5% of axial strain in double amplitude.

It is observed a significant effect of the overconsolidation ratio on the liquefaction resistance. For a number of cycles in the range of 20 to 30, overconsolidated specimens (OCR=6) present a cyclic resistance in the order of 20% higher than normally consolidated specimens. The increase is greater as the number of cycles increases. The experimental results obtained from the undrained cyclic tests suggest that the effect of the overconsolidation ratio on the cyclic strength is significant. This observation is in agreement with previous studies that already proved this well-known effect (Ishihara et al. 1979; Finn 1981; Dobry et al. 1981b; Adalier et al. 2005).

Other Factors that do not Affect Shear Wave Velocity

The main factors associated with the soil state that control, or have an important effect on the liquefaction resistance of sandy soil deposits are the followings: relative density, soil structure or fabric (sample preparations methods), aging, overconsolidation, K_o (lateral pressure), seismic prestraining or preshaking. It is possible to indicate that there is a general consensus about the importance of these factors on the onset of liquefaction (Seed 1979, Finn 1981; Ishihara 1985, 1993, Dobry 2011).

On the other hand, in sandy soils some of these factors have only a marginal effect on V_s. Specifically, V_s is weakly influence by: soil structure or fabric (sample preparations methods), aging, overconsolidation and seismic prestraining or preshaking.

Tatsuoka et al. (1979) carried out an extensive experimental program to investigate the effect of sample preparation on the shear modulus. The conclusion was that the shear modulus at small strain level is insensitive to the sample preparation including pouring, compacting, moistening, saturating, unsaturating, freezing and thawing. Similar conclusion regarding the insensitivity of

V_s to sand fabric has been reported by Alarcon et al. (1989). On the contrary, there is robust experimental evidence showing that the initial soil fabric, or sample preparation, has a significant effect on the onset of liquefaction (Park et al. 1975; Mulilis et al. 1975; Tatsuoka et al. 1986).
Aging is also a factor that has been reported to have an important effect on the cyclic strength of sandy soils (Troncoso et al. 1988; Mori et al. 1978). Experimental results on tailings sands indicate that the cyclic stress ratio required for generating 5% strain in double amplitude increases by a factor of 3.5 just in 30 years of sustained deposition (Troncoso et al. 1988). On the other hand, Afifi el al. (1973) have reported for sandy soils a relatively unimportant increase of the shear modulus at small strain level with time.
Seismic prestraining or preshaking has an important effect on the liquefaction resistance, however, it has little effect on V_s as it has been observed in experimental results (Drenevich et al. 1970; Witchmann et al. 2004)

Concluding Remarks

The shear strains thresholds that characterize the behavior of sandy soils have been described, especially important are the elastic threshold and the volumetric threshold shear strain. At very small shear strains, below 10^{-5} (10^{-3} %), the stress-strain response is fairly linear, and the shear strain level in the order of 10^{-4} (10^{-2} %), separates the cyclic soil response with and without volumetric strain accumulations.
The liquefaction phenomenon is intrinsically related to the natural tendency of loose cohesionless soils to generate positive volumetric strains (contraction) when subjected to monotonic or cyclic loads. Therefore, the onset of liquefaction takes place well above the volumetric threshold shear strain.
On the other hand, the measured shear wave velocity is a soil parameter essentially associated with a shear strain level in the elastic range, where the particle media do not show volumetric strains and only a marginal plastic strain.
The main factors with a significant impact on the liquefaction resistance of sandy soil deposits are: relative density, soil structure or fabric (sample preparations methods), aging, overconsolidation, Ko (lateral pressure), seismic prestraining or preshaking. However, among these factors, soil structure or fabric, aging, overconsolidation and seismic prestraining or preshaking have a modest effect on Vs. Specifically, laboratory experimental results showing the low sensitivity of Vs with OCR are presented. Additionally, Vs correlates linearly with the void ratio, regardless of the maximum and minimum void ratios. In other words, Vs is unable to give information about the soil packing.
Shear wave velocity is an index parameter that can be measured in the field with fewer efforts and difficulties compared to other field tests, and therefore, its use highly appealing. In the case of using shear wave velocity as liquefaction predictor, it is recommended to take into account the limitations presented in this work.

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