# A Liquefaction Evaluation Procedure Based on Shear Wave Velocity

by

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ABSTRACT

This paper outlines a procedure for evaluating liquefaction resistance of soils using shear wave velocity measurements. The procedure follows the general format of the Seed-Idriss simplified procedure based on SPT blow count. It was developed following suggestions from industry, researchers, and practitioners, and using case history data from 26 earthquakes and over 70 measurement sites in soils ranging from sand to sandy gravel with cobbles to profiles including silty clay layers. The procedure correctly predicts moderate to high liquefaction potential for over 95 % of the liquefaction case histories. A case study is provided to illustrate the application of the proposed procedure.

KEYWORDS: Building technology; earthquake engineering; *in situ* measurements; seismic testing; shear wave velocity; soil liquefaction.

## 1. INTRODUCTION

Evaluating the liquefaction resistance of soils is an important step in the engineering design of new structures and the retrofit of existing structures in earthquake-prone regions. The evaluation procedure widely used in the United States and throughout much of the world is termed the *simplified procedure*. This simplified procedure was originally developed by Seed and Idriss (1971) using blow counts from the Standard Penetration Test (SPT) correlated with a parameter representing the seismic loading on the soil, called the *cyclic stress ratio*.

Small-strain shear wave velocity, $V_S$, measurements provide a promising alternative, or supplement, to the penetration-based approach. The use of $V_S$ as an index of liquefaction resistance is soundly based, since both $V_S$ and liquefaction resistance are similarly influenced by void ratio, state of stress, stress history, and geologic age. Furthermore, the strong theoretical basis underlying stress wave propagation offers the opportunity for additional advances in the approach.

During the past two decades, several simplified procedures for evaluating liquefaction resistance based on $V_S$ have been proposed (Dobry et al., 1981; Dobry et al., 1982; Seed et al., 1983; Bierschwale and Stokoe, 1984; de Alba et al., 1984; Hynes, 1988; Stokoe et al., 1988; Tokimatsu and Uchida, 1990; Tokimatsu et al., 1991; Robertson et al., 1992; Kayen et al., 1992; Andrus, 1994; Lodge, 1994; Rashidian, 1995; Kayabali, 1996; Andrus and Stokoe, 1997; Rollins et al., 1998; and Andrus et al., 1999). Some of these procedures follow the general format of the Seed-Idriss simplified procedure, where $V_S$ is corrected to a reference overburden stress and correlated with the cyclic stress ratio. Nearly all were developed with limited or no field performance data. This paper outlines the procedure proposed by Andrus and Stokoe (1997), and updated by Andrus et al. (1999) using an expanded database.

The expanded database compiled by Andrus et al. (1999) consists of field performance data from 26 earthquakes and $V_S$ measurements at

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over 70 sites. Much of the new data are from the 1995 Hyogoken-Nanbu (Kobe), Japan earthquake (moment magnitude, $M_w = 6.9$).

## 2. LIQUEFACTION RESISTANCE FROM $V_S$

### 2.1 Cyclic Stress Ratio ($CSR$)

The cyclic stress ratio, $\tau_{av}/\sigma'_v$, at a particular depth in a level soil deposit can be expressed as (Seed and Idriss, 1971):

$$\mathrm{CSR} = \frac{\tau_{av}}{\sigma'_v} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_v}{\sigma'_v}\right) r_d \qquad (1)$$

where $\tau_{av}$ is the average equivalent uniform cyclic shear stress caused by the earthquake and is assumed to be 0.65 of the maximum induced stress, $a_{max}$ is the peak horizontal ground surface acceleration, $g$ is the acceleration of gravity, $\sigma'_v$ is the initial effective vertical (overburden) stress at the depth in question, $\sigma_v$ is the total overburden stress at the same depth, and $r_d$ is a shear stress reduction coefficient to adjust for flexibility of the soil profile.

### 2.2 Stress-Corrected Shear Wave Velocity

Following the traditional procedures for correcting SPT blow count to account for overburden stress, one can correct $V_S$ to a reference overburden stress by (Sykora, 1987; Robertson et al., 1992):

$$V_{S1} = V_S\left(\frac{P_a}{\sigma'_v}\right)^{0.25} \qquad (2)$$

where $V_{S1}$ is the overburden stress-corrected shear wave velocity, $P_a$ is a reference stress, 100 kPa or about atmospheric pressure, and $\sigma'_v$ is initial effective overburden stress in kPa. In using Eq. (2), it is implicitly assumed that the initial effective horizontal stress, $\sigma'_h$, is a constant factor of the effective overburden stress. The factor, generally referred to as $K'_o$, is assumed to be approximately 0.5 at sites where liquefaction has occurred. Also, in applying Eq. (2), it is implicitly assumed that $V_S$ is measured with both the directions of particle motion and wave propagation polarized along principal stress directions and one of these directions is vertical (Stokoe et al., 1985).

### 2.3 Cyclic Resistance Ratio ($CRR$)

The value of $CSR$ separating liquefaction and non-liquefaction occurrences for a given $V_{S1}$, or corrected blow count, is called the *cyclic resistance ratio*.

Andrus and Stokoe (1997) proposed the following relationship between $CRR$ and $V_{S1}$:

$$CRR = \left\{a\left(\frac{V_{S1}}{100}\right)^2 + b\left(\frac{1}{V^*_{S1} - V_{S1}} - \frac{1}{V^*_{S1}}\right)\right\} MSF \qquad (3)$$

where $V^*_{S1}$ is the limiting upper value of $V_{S1}$ for liquefaction occurrence, $a$ and $b$ are curve fitting parameters, and $MSF$ is the magnitude scaling factor. The first term of Eq. (3) is based on a modified relationship between $V_{S1}$ and $CSR$ for constant average cyclic shear strain suggested by R. Dobry (personal communication to R. D. Andrus, 1996; Andrus and Stokoe, 1997). The second term is a hyperbola with a small value at low values of $V_{S1}$, and a very large value as $V_{S1}$ approaches $V^*_{S1}$.

The magnitude scaling factor, which accounts for the effect of earthquake magnitude on $CRR$, can be expressed by:

$$MSF = \left(\frac{M_w}{7.5}\right)^n \qquad (4)$$

where $n$ is an exponent. The lower bound for the range of magnitude scaling factors recommended by the 1996 National Center for Earthquake Engineering Research (NCEER) Workshop on Evaluation of Liquefaction Resistance of Soils (Youd et al., 1997) is defined by Eq. (4) with $n$ = -2.56 (Idriss, personal communication to T. L. Youd, 1995).

Figure 1 presents the case history data for magnitude 5.9 to 8.3 earthquakes adjusted using Eq. (4) with n = -2.56. Also presented in Fig. 1 are the proposed $CRR$-$V_{S1}$ curves. The curves are defined by Eq. (3) with $a = 0.022$, $b = 2.8$, $V^*_{S1}$ = 200 m/s for fines content $(FC) \geq 35$ %, $V^*_{S1}$ = 208 m/s for $FC$ = 20 %, and $V^*_{S1}$ = 215 m/s for $FC \leq 5$ %. The case histories, and $CRR$-$V_{S1}$ curves, are limited to relatively level ground sites with average depths less than 10 m, uncemented soils of Holocene age, ground water table depths between 0.5 m and 6 m, and $V_S$ measurements made below the water table.

Of the 90 liquefaction case histories shown in Fig. 1, only two incorrectly lie in the no-liquefaction region. The two liquefaction cases that lie in the no-liquefaction region are for sites at Treasure Island, California. These sites are located along the perimeter of the island where liquefaction was marginal during the 1989 Loma Prieta earthquake ($M_w$ = 7.0).

### 2.4 Factor of Safety

A common way to quantify the hazard for liquefaction is in terms of a factor of safety, $FS$. The $FS$ against liquefaction can be defined by:

$$FS = \frac{CRR}{CSR} \tag{5}$$

Liquefaction is predicted to occur when $FS \leq 1$, and liquefaction is predicted not to occur when $FS > 1$. The acceptable value of $FS$ for a particular site will depend on several factors, including the acceptable level of risk for the project, the extent and accuracy of seismic measurements, the availability of other site information, and the conservatism in determining the design earthquake magnitude and the expected value of $a_{max}$.

## 3. CASE STUDY

Figure 2 presents the liquefaction evaluation for a crosshole test array at the Treasure Island Fire Station site and the 1989 Loma Prieta earthquake. Values of $V_{S1}$ and $CSR$ shown in Figs. 2(a) and 2(d), respectively, were calculated assuming soil densities of 1.76 Mg/m$^3$ above the water table and 1.92 Mg/m$^3$ below the water table. Based on $a_{max}$ of 0.16 g and 0.11 g recorded in two horizontal directions at the fire station during the 1989 earthquake (Brady and Shakal, 1994), a geometric mean value of 0.13 g was used to calculate $CSR$.

Values of $FS$ shown in Fig. 2(e) are less than 1 for the depths of 4 m to 9 m. Between the depths of 4 m and 7 m, the sand contains non-plastic fines and is considered liquefiable. Between the depths of 7 m and 9 m, the soil exhibits plastic characteristics and may be non-liquefiable by the so-called Chinese criteria. According to the Chinese criteria, non-liquefiable clayey soils have clay contents (particles smaller than 5 μm) ≥ 15 %, liquid limits ≥ 35 %, or moisture contents ≤ 90 % of the liquid limit (Seed and Idriss, 1982). Thus, the layer most likely to liquefy, or the critical layer, lies between the depths of 4 m and 7 m.

Although no sand boils or ground cracks occurred at the site during the 1989 earthquake, there is a sudden drop in the fire station strong ground motion recordings at about 15 seconds and small motion afterwards (Idriss, 1990). This behavior is unlike behavior observed in recordings at other seismograph stations located on soft-soil sites in the San Francisco Bay area. De Alba et al. (1994) attributed this behavior to liquefaction of an underlying sand. It is possible that the 4 m thick layer capping the site, predicted not to liquefy in Figs. 2(d) and 2(e), prevented the formation of sand boils at the ground surface (Ishihara, 1985).

## 4. CONCLUSIONS

Outlined in this paper is a procedure for evaluating liquefaction resistance through $V_S$ measurements. The proposed procedure follows the general format of the Seed-Idriss simplified procedure based on SPT blow count. Liquefaction criteria based on $V_S$ have been developed with case history data from soils ranging from sand to sandy gravel with cobbles to profiles including silty clay layers. Caution should be exercised when applying the procedure to sites where conditions are different from the database. Additional well-documented case histories with all types of soil that have and have not liquefied during earthquakes are needed, particularly from denser soils ($V_{S1}$ > 200 m/s) shaken by stronger ground motions ($a_{max}$ > 0.4 g), to further validate the procedure.

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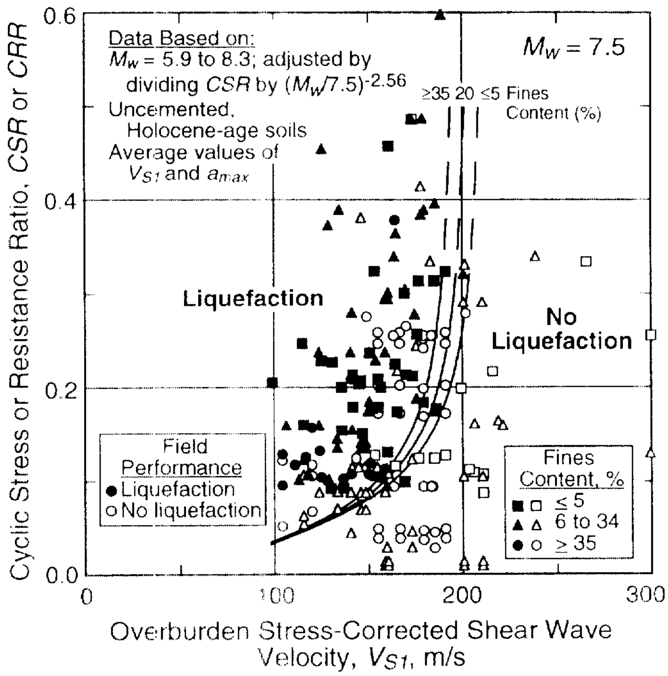


Fig. 1 - Curves Proposed by Andrus et al. (1999) for Calculation of $CRR$ from $V_S$ Measurements Along with Case History Data Based on Lower Bound Values of $MSF$ for the Range Recommended by the 1996 NCEER Workshop (Youd et al., 1997) and Average $r_d$ Values Developed by Seed and Idriss (1971).

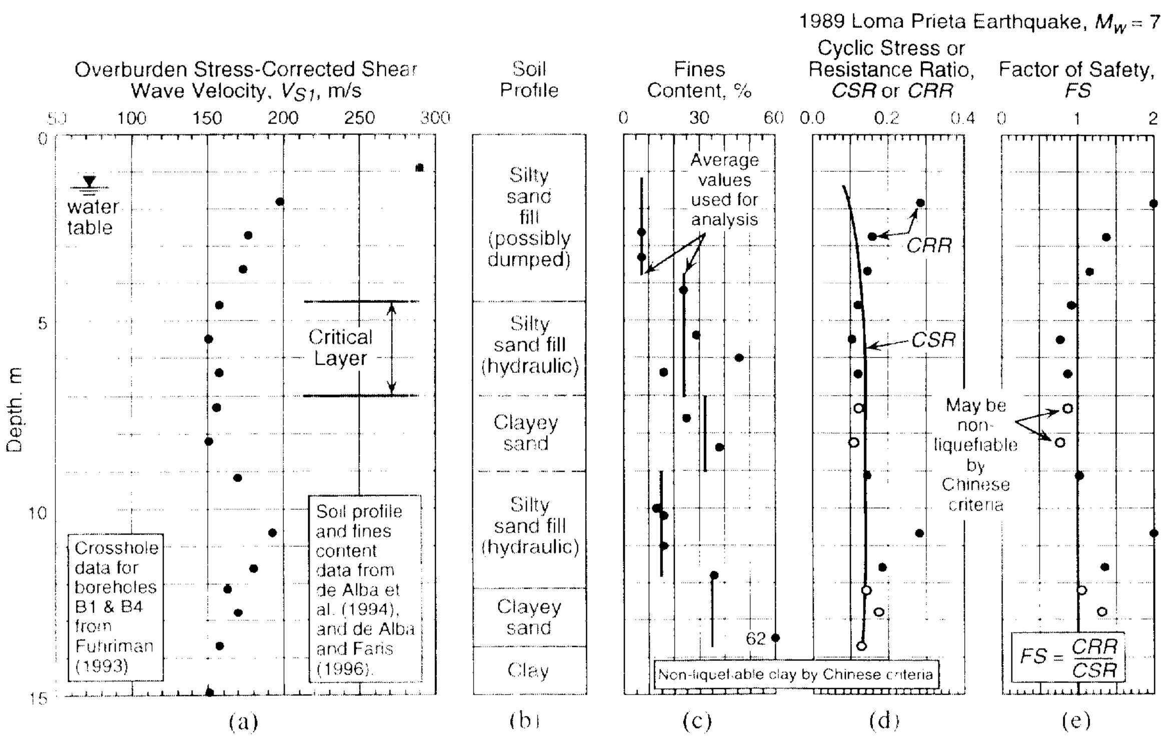


Fig. 2 - Application of the Recommended Procedure to the Treasure Island Fire Station Site, Crosshole Test Array B1-B4.