Guide for Shear-Wave-Based Liquefaction Potential Evaluation

Ronald D. Andrus,[a] M.EERI, Kenneth H. Stokoe II,[b] M.EERI, and C. Hsein Juang[a]

Small-strain shear-wave velocity measurements provide a promising approach to liquefaction potential evaluation. In some cases, where only seismic measurements are possible, it may be the only alternative to the penetration-based approach. Various investigators have developed relationships between shear wave velocity and liquefaction resistance. Successful application of any liquefaction evaluation method requires that procedures used in their development also be used in their application. This paper presents detailed guidelines for applying the procedure described in Andrus and Stokoe that was developed using suggestions from two workshops and following the general format of the Seed-Idriss simplified procedure. Correction factors to velocity and liquefaction resistance for soil aging are suggested. Based on the work by Juang et al., factors of safety of 1.0, 1.2, and 1.5 correspond to probabilities of liquefaction of about 0.26, 0.16, and 0.08, respectively. Additional field performance data are needed from all soil types, particularly denser and older soil deposits shaken by stronger ground motions, to further validate the recommended procedure. [DOI: 10.1193/1.1715106]

INTRODUCTION

The procedure for predicting liquefaction resistance of soils widely used throughout much of the world is termed the *simplified procedure*. This simplified procedure was originally developed by Seed and Idriss (1971) using the Standard Penetration Test (SPT) blow counts correlated with a parameter representing the seismic loading on the soil, called *cyclic stress ratio*. Since 1971, this procedure has undergone several revisions and updates (Seed 1979, Seed and Idriss 1982, Seed et al. 1983, Seed et al. 1985). In addition, procedures based on the Cone Penetration Test (CPT), Becker Penetration Test (BPT), and small-strain shear-wave velocity (V_S) measurements have been developed. General reviews of the simplified procedure are contained in a report by the National Research Council (1985) and a summary report from the 1996 National Center for Earthquake Engineering Research (NCEER) and 1998 NCEER/National Science Foundation (NSF) workshops on evaluation of liquefaction resistance of soils by Youd et al. (2001).

As stated by Youd et al. (2001), "SPTs and CPTs are generally preferred (for assessment of liquefaction resistance) because of the more extensive databases and past experience, but the other tests may be applied at sites underlain by gravelly sediments or

[a] Dept. of Civil Engineering, Clemson Univ., Lowry Hall Box 340911, Clemson, SC 29634-0911
[b] Dept. of Civil Engineering, The Univ. of Texas at Austin, ECJ 9.227, Austin, TX 78712

Earthquake Spectra, Volume 20, No. 2, pages 285–308, May 2004;

Table 1. Advantages and disadvantages of various field tests for liquefaction potential evaluation (modified from Youd et al. 2001)

Feature	Test Type			
	SPT	CPT	V_S	BPT
Past measurements at liquefaction sites	Abundant	Abundant	Limited	Sparse
Type of stress-strain behavior influencing test	Large strain, partially drained	Large strain, drained	Small strain, no excess pore-water pressure	Large strain, partially drained
Quality control and repeatability	Poor to good	Very good	Good	Poor
Detection of variability of soil deposits	Good for closely spaced tests	Very good	Fair	Fair
Soil types in which test is recommended	Nongravel	Nongravel	All	Primarily gravel
Soil sample retrieved	Yes	No	No	No
Test measures index or engineering property	Index	Index	Engineering	Index

where access by large equipment is limited." Advantages and disadvantages of each test method are listed in Table 1. The advantages of using V_S include (1) V_S measurements are possible in soils where CPT and SPT may be unreliable, such as gravelly soils, and at sites where borings or soundings may not be permitted; (2) V_S is an engineering property, directly related to small-strain shear modulus, G_{max}; and (3) G_{max}, or V_S, is a parameter required for dynamic soil response and soil-structure interaction analyses. The disadvantages include (1) V_S is a small- ($<0.001\%$) strain measurement, whereas pore-water pressure buildup and liquefaction are medium- to large- ($>1\%$) strain phenomena; (2) V_S tests do not provide samples for classification and identification of nonliquefiable clayey soils; and (3) if the measurement interval is too large, thin, low V_S strata may not be detected.

The use of V_S for determining liquefaction resistance is soundly based because both V_S and liquefaction resistance are influenced by many of the same factors. Laboratory studies have shown that confining stress, soil type/plasticity, and void ratio/relative density are the most important factors influencing the variation of shear modulus, or shear wave velocity, with shear strain amplitude (e.g., Hardin and Drnevich 1972, Kramer 1996, Ishihara 1996). Liquefaction results from the rearranging of soil particles and tendency for decrease in volume. A threshold cyclic strain exists below which neither rearrangement of soil particles nor decrease in volume takes place (Drnevich and Richart 1970, Youd 1972, Pyke et al. 1975), and no pore-water pressure buildup occurs (Dobry et al. 1981, Seed et al. 1983). The threshold cyclic strain, at a mean effective confining stress of 100 kPa, varies with the gradation of the soil and ranges around 0.004% for normally consolidated, well-graded gravels to 0.01% for normally consolidated clean sands. As the materials become overconsolidated, the threshold cyclic strain increases. In addition, there is a predictable relationship between cyclic strain and pore pressure buildup of saturated soils that depends on soil type/plasticity, void ratio/relative density, and number of loading cycles (Martin et al. 1975, Park and Silver 1975, Finn and Bhatia 1981, Dobry et al. 1982, Hynes 1988). It should also be noted that the steady state approach to liquefaction evaluation by Poulos et al. (1985) is based on soil type, void ratio, and triggering strain level. These findings support the use of V_S for assessment of liquefaction resistance.

During the past two decades, several procedures for estimating liquefaction resistance based on V_S have been proposed. These procedures were developed from laboratory studies (Dobry et al. 1981, Dobry et al. 1982, de Alba et al. 1984, Hynes 1988, Tokimatsu and Uchida 1990, Tokimatsu et al. 1991, Rashidian 1995, Rauch et al. 2000), analytical studies (Bierschwale and Stokoe 1984, Stokoe et al. 1988b, Andrus 1994), penetration-V_S equations (Seed et al. 1983, Lodge 1994, Kayabali 1996, Rollins et al. 1998b), or in situ V_S measurements at sites shaken by earthquakes (Stokoe and Nazarian 1985, Robertson et al. 1992, Kayen et al. 1992, Andrus and Stokoe 1997, Juang and Chen 2000, Andrus and Stokoe 2000, Juang et al. 2002). Several of these procedures follow the general format of the Seed-Idriss simplified procedure, where V_S is corrected to a reference vertical stress and correlated with the cyclic stress ratio.

Presented in this paper are guidelines for using the evaluation procedure originally described in the workshop paper by Andrus and Stokoe (1997), and subsequently updated in the paper by Andrus and Stokoe (2000) and the report by Andrus et al. (2003). The procedure follows the general format of the Seed-Idriss simplified procedure, and the general recommendations of the 1996 NCEER and 1998 NCEER/NSF workshops (Youd et al. 2001). For the first time, advantages and disadvantages of in situ V_S test methods are discussed in terms of their application for liquefaction potential evaluation. Also, for the first time, guidance for selecting and calculating parameters required for the evaluation are given, including age correction factors. In addition, recent probability studies by Juang et al. (2002) for quantifying the potential for liquefaction are discussed. Finally, to illustrate the application of the updated procedure and guidelines, two case studies are presented.

IN SITU SHEAR WAVE VELOCITY

The in situ V_S can be measured by several seismic tests including crosshole, downhole, seismic cone penetrometer (SCPT), suspension logger, and spectral analysis of surface waves (SASW). Reviews of these test methods are given in Woods (1994), Kramer (1996), and Ishihara (1996). The accuracy of each test method can be sensitive to equipment and procedural details, soil conditions, and interpretation techniques. *ASTM D-4428M-91* (ASTM 1991) provides a standard test method for crosshole seismic testing. Standard test methods currently do not exist for the other seismic tests. Primary features of the various in situ V_S test methods for liquefaction evaluations are presented in Table 2.

When selecting the in situ V_S test method for liquefaction evaluations, it is important to keep in mind that test procedures are often tailored to a particular application, and V_S measurements made for one application sometimes should not be used in another application. For example, the National Earthquake Hazards Reduction Program (NEHRP)

Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (BSSC 2000) and the *International Building Code* (ICC 2000) classify sites based on average V_S in the upper 30 m of the ground. The average V_S in the upper 30 m, V_{S30}, is sometimes determined using very simplified test procedures. Measurements based on simplified test procedures should not be used for final site-specific liquefaction assessment. The surface reflection/refraction method is appropriate for screening large areas for liquefaction hazards as long as velocity continually increases with depth. The SASW method is appropriate for both screening large areas and detailed site-specific assessment, depending on the rigorousness of the inversion, or forward modeling, process. The crosshole, downhole, SCPT, and suspension logger methods are most appropriate for detailed site-specific assessment, especially for thinner layers. As a general rule, V_S should be determined at depth intervals of at least one-quarter the thickness of the critical layer.

It is also worth noting that the crosshole and surface refraction tests can be used to determine if the potentially liquefiable soil is fully saturated or partially saturated. In fully saturated soil, values of compression-wave velocity are on the order of 1500 m/s. Ishihara et al. (1998) and Tsukamoto et al. (2002) have shown that the cyclic strength can be twice as high in partially saturated soil than in fully saturated soil.

In situ V_S tests for liquefaction evaluations should be conducted so that at least a major component of particle motion or wave propagation is in the vertical direction. The reason for this requirement is that liquefaction in the field often depends on the induced shear strain in the vertical plane. Thus it is the stiffness of the soil structure in the vertical plane that is generally assumed to be of primary concern. Laboratory studies have shown that stiffness (or V_S) depends equally on principal stresses in the directions of wave propagation and particle motion (Roessler 1979, Stokoe et al. 1985, Santamarina et al. 2001). If both particle motion and wave propagation directions were in the horizontal plane, only the soil stiffness in the horizontal plane would be determined. To have a major component of wave propagation or particle motion in the vertical direction, crosshole tests should be conducted with particle motion in the vertical direction, downhole and seismic cone tests should be conducted such that the distance between the shear-beam source and receiver hole is less than the depth to the receiver, and SASW tests should be conducted with a vertical source.

In general, borings should always be a part of the field investigation. Surface seismic tests, including reflection/refraction and SASW, usually involve making measurements at several different locations on the ground surface. The ability of surface seismic methods to resolve a layer at depth depends on the thickness, depth, velocity contrast, and continuity of that layer, as well as the test and interpretation procedures employed. The preferred practice when using V_S measurements to evaluate liquefaction potential is to drill sufficient boreholes and conduct sufficient tests to detect and delineate thin liquefiable strata, to identify silty soils above the groundwater table that might have lower values of V_S should the water table rise, to detect liquefiable weakly cemented soils, and to identify nonliquefiable clay-rich soils. According to the Chinese criteria (Seed and Idriss 1982, Andrews and Martin 2000), nonliquefiable clayey soils have clay contents (particles smaller than 2 μm) $\geqslant$10% and liquid limits $>$35%. It should be noted that some exceptions to those criteria have been observed in recent earthquake studies (e.g., Sancio et al. 2002, 2003).

Table 2. Comparison of various in situ V_S test methods for liquefaction assessment

Feature	Measurement Method: Crosshole	Downhole & Seismic Cone Penetrometer	Suspension Logger	Spectral Analysis of Surface Waves	Surface Reflection/ Refraction
Number of holes required	2 or more	1	1	None	None
Quality control and repeatability[1]	Good	Good	Good	Good to fair; complex interpretation technique at sites with large velocity contrasts	Fair; often difficult to distinguish shear wave arrival
Resolution of variability in stiffness of soil deposits[2]	Good; constant with depth	Good to fair; decreases with depth	Good at depth; poor very close (3 to 6 m) to the ground surface	Good to fair; decreases with depth; provides good global average	Fair to poor; provides coarse global average
Major component of particle motion or wave propagation in vertical direction?	Yes, with vertically polarized shear waves	Yes, with test depth greater than distance between hole and shear-beam source	Yes, with refracted shear waves traveling parallel to vertical borehole	Yes, with vertical source	Yes, with horizontal source for reflection and vertical source for refraction
Limitations	Possible refraction problems; senses stiffer material at test depth; most expensive test method	Possible refraction problems with shallow layers; wave travel path increases with depth	Fluid-filled hole required; may not work well near the surface in cased holes and soft soils	Horizontal layering assumed; poor resolution of thin layers and soft material adjacent to stiff layers; no samples recovered	In refraction test, only works for velocity increasing with depth; no samples recovered
Other	Highly reliable test; measurements at each depth independent of other depths; well suited for tomographic imaging; independent checking of saturation with compression waves is possible	Penetration data also obtained from seismic cone; detailed layered profile with cone	Well suited for deep borehole testing; method assumes shear waves travel in undisturbed soil	Well suited for tomographic imaging large areas and testing difficult to penetrate soils	Well suited for screening large areas; independent checking of saturation with compression waves is possible

[1] Good quality depends on good equipment and procedural details, and good interpretation techniques for all methods.
[2] Resolution depends on test spacing for all methods.

LIQUEFACTION EVALUATION PROCEDURE

The evaluation procedure described in Andrus and Stokoe (2000) requires the calculation of three parameters: (1) the level of cyclic loading on the soil caused by the earthquake, expressed as a cyclic stress ratio; (2) the stiffness of the soil, expressed as a stress-corrected shear-wave velocity; and (3) the resistance of the soil to liquefaction, expressed as a cyclic resistance ratio. Guidelines for calculating each parameter are presented below.

CYCLIC STRESS RATIO

The cyclic stress ratio, *CSR*, at a particular depth in a level soil deposit can be calculated from (Seed and Idriss 1971)

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

where τ_{av} is the average equivalent uniform cyclic shear stress caused by the earthquake and 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, σ'_v is the initial effective vertical (overburden) stress at the depth in question, σ_v is the total overburden stress at the same depth, and r_d is a shear stress-reduction coefficient.

Peak Horizontal Ground Surface Acceleration

Peak horizontal ground surface acceleration is a characteristic of the ground shaking intensity. For liquefaction evaluations, $a_{\max}$ is defined as the peak value in a horizontal ground acceleration record that would occur at the site without the influence of excess pore-water pressures or liquefaction that might develop (Youd et al. 2001). Peak accelerations are commonly estimated using empirical attenuation relationships of $a_{\max}$ as a function of earthquake magnitude, distance from the energy source or surface projection of the fault rupture, and local site conditions. Because many published attenuation relationships are based on both peak values obtained from ground motion records for the two horizontal directions (sometimes referred to as the randomly oriented horizontal component), the geometric mean (square root of the product) of the two peak values is used. According to Youd et al. (2001), use of the geometric mean is consistent with the derivation of the SPT-based procedure and is preferred for use in engineering practice. However, use of the larger of the two horizontal peak accelerations would be conservative and is allowable.

Regional or national seismic hazard maps (http://geohazards.cr.usgs.gov/eq/, Frankel et al. 2000) are also often used to estimate $a_{\max}$. If $a_{\max}$ is estimated from a map, the magnitude and distance information should be obtained from the deaggregated matrices used to develop the map. The value of $a_{\max}$ selected will depend on the target level of risk and compatibility of site conditions. For site conditions not compatible with available probabilistic maps or attenuation relationships, the value of $a_{\max}$ may be corrected based on dynamic site response analyses or site class coefficients given in the latest building codes.

Total and Eeffective Overburden Stress

Required in the calculation of σ_v and σ'_v are densities of the various soil layers and characteristics of the groundwater. For noncritical projects involving hard-to-sample soils below the groundwater table, densities are often estimated from typical values for soils with similar grain size and penetration, or velocity, characteristics. Fortunately, *CSR* is not very sensitive to density, and reasonable estimates of density yield adequate results. The values of σ'_v and *CSR* are sensitive to the groundwater table depth. Other groundwater characteristics that may be significant to liquefaction evaluations include seasonal and long-term water level variations, depth of and pressure in artesian zones, whether the water table is perched or normal, and whether there are unsaturated zones below the water table due to undissolved gases. For liquefaction design evaluations, the highest possible water table and artesian pressure should be used to calculate *CSR* unless additional credible information is available.

Another factor to consider when calculating σ_v and σ'_v is the induced stress due to any applied load. If the project involves a wide embankment fill, then the induced stress is simply calculated by multiplying the height of the fill by its unit weight. For narrow embankments and buildings, induced stresses are generally non-uniform and significant effort may be required for their determination. In addition, the applied load may induce static shear stresses acting in the vertical and horizontal planes. Static shear stresses acting in the vertical and horizontal planes are commonly assumed for sloping ground sites. Although correction factors for sloping ground sites have been published (Harder and Boulanger 1997), Youd et al. (2001) recommend these factors “should not be used by nonspecialists in geotechnical earthquake engineering or in routine engineering practice.” Also, it should be noted that correction factors have been recommended when $\sigma'_v > 100$ Pa. The reader is referred to Hynes and Olsen (1999) or Youd et al. (2001) for these factors.

Stress reduction coefficient

For routine practice and noncritical projects, Youd et al. (2001) suggest the following equations be used to estimate average values of r_d (Liao and Whitman 1986):

$$r_d = 1.0 - 0.00765z \quad \text{for } z \leqslant 9.15 \text{ m} \tag{2a}$$

$$r_d = 1.174 - 0.0267z \quad \text{for } 9.15 \text{ m} < z \leqslant 23 \text{ m} \tag{2b}$$

where z is the depth below the ground surface in meters. Equations 2a and 2b represent a bilinear fit of the average curve proposed by Seed and Idriss (1971). Below a depth of 23 m, Youd et al. (2001) did not recommend values of r_d because “evaluation of liquefaction at these greater depths is beyond the depths where the simplified procedure is verified and where routine applications should be applied.” Also they emphasized that the user should understand that there is a wide range of possible r_d, and that range increases with depth.

Subsequent analytical work by Golesorkhi (1989), under the supervision of the late Prof. H. B. Seed, indicated that r_d also depends on magnitude. Based on that work, Idriss (1999) proposed a new procedure for determining magnitude-dependent values of r_d.

When applying these new values of r_d, a compatible set of magnitude scaling factors must also be used. As an alternative approach, the variation of r_d with depth may be calculated analytically using site-specific layer thicknesses and stiffnesses.

STRESS-CORRECTED SHEAR-WAVE VELOCITY

Following the traditional procedures for correcting penetration resistance, V_S should be corrected to a reference overburden stress by (Sykora 1987, Robertson et al. 1992)

$$V_{S1} = V_S \; C_{VS} = V_S \left(\frac{P_a}{\sigma'_v} \right)^{0.25} \tag{3}$$

where V_{S1} is stress-corrected shear-wave velocity, C_{VS} is a factor to correct measured V_S for overburden pressure, P_a is a reference stress of 100 kPa, and σ'_v is initial effective overburden stress in kPa. A maximum C_{VS} value of 1.4 is applied to V_S data at shallow depths. (The value of 1.4 is less than the maximum value of 1.7 commonly assumed in the penetration-based methods due to the exponent of 0.25 versus 0.5 in the penetration corrections.) In applying Equation 3, two assumptions are implicitly made. First, it is assumed that the initial effective horizontal stress, σ'_h, is a constant factor of the effective vertical stress. The factor, referred to as the coefficient of effective earth pressures at rest, K'_o, is about 0.5 at sites where liquefaction has occurred. Second, it is 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). Thus Equation 3 is most appropriate for level ground sites where at-rest, normally consolidated conditions can be assumed and K'_o is around 0.5.

For soil deposits where K'_o is significantly different from 0.5, the following overburden correction equation is suggested:

$$V_{S1} = V_S \left(\frac{P_a}{\sigma'_v} \right)^{0.25} \left(\frac{0.5}{K'_o} \right)^{0.125} \tag{4}$$

The K'_o term in Equation 4 assumes that V_S depends equally on σ'_v and σ'_h, with each stress having an exponent of 0.125. Equations 4 provides a lower value of V_{S1} than Equation 3, when K'_o is greater than 0.5. When K'_o is less than 0.5, Equation 4 provides a higher value of V_{S1} than Equation 3.

If V_S measurements are made when the groundwater table is low and a higher water table is possible, values of V_S above the water table may be too high due to negative pore-water pressures. Negative pore pressures can be particularly significant in silty soils. This effect should be considered in the estimation of σ'_v for correcting V_S to V_{S1}, and for computing CSR using Equation 1.

CYCLIC RESISTANCE RATIO

The cyclic resistance ratio, CRR, can be thought of as the value of CSR separating liquefaction and nonliquefaction occurrences for a given V_{S1}. Shown in Figure 1 are the CRR-V_{S1} curves by Andrus and Stokoe (2000) for magnitude 7.5 earthquakes. Also

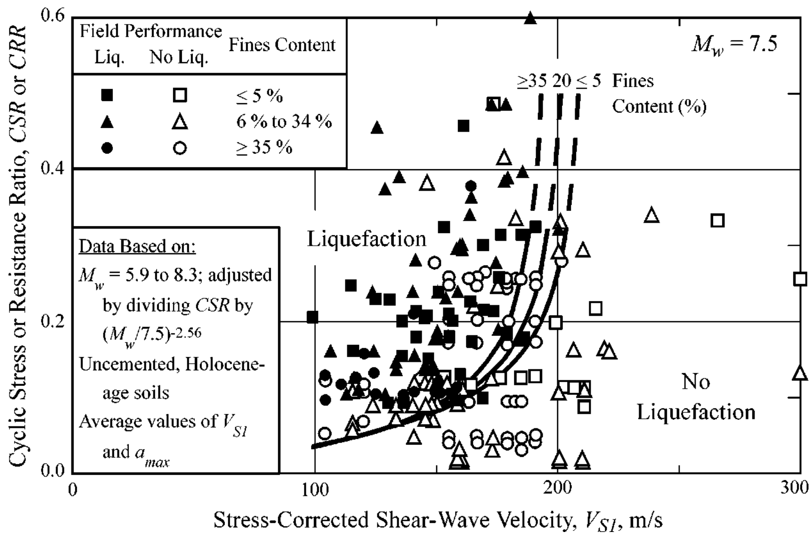

Figure 1. Liquefaction resistance curves by Andrus and Stokoe (2000) for magnitude 7.5 earthquakes and uncemented soils of Holocene age with case history data.

shown are the case histories used to establish the curves. The curves are drawn to bound all but two of the liquefaction cases. The case history data are limited to relatively level ground sites with the following characteristics: (1) uncemented soils of Holocene age (<10,000 years), (2) average depths less than 10 m, (3) groundwater table depths between 0.5 m and 6 m, and (4) V_S measurements performed below the water table. The curves are dashed above CRR of about 0.35 to indicate that they are based on limited case history data. They do not extend much below 100 m/s, since there are no field data to support extending them to the origin. Extra care should be exercised when applying the CRR-V_{S1} curves shown in Figure 1 to sites where conditions are different from the general characteristics of the case history data.

Although there is lack of a good distinction between liquefaction and nonliquefaction cases shown in Figure 1, particularly for soils with greater than 5% fines (soil particles passing the No. 200 sieve), some overlap should be expected. One contributing factor to the overlap is rooted in the definition of liquefaction occurrence. Andrus and Stokoe (2000) determined liquefaction occurrence primarily based on the observance of surface manifestations (i.e., sand boils, ground cracks, and settlement). It is possible that high pore-water pressures were generated at some of the nonliquefaction sites, but no surface manifestations occurred because of a thick capping layer. Ishihara (1985) suggested that surface manifestations of liquefaction depend on the thickness of the liquefiable layer, thickness of the nonliquefiable capping layer, and peak horizontal ground surface acceleration. Based on Ishihara's (1985) criteria, about 15 of the nonliquefaction cases plotted in the liquefiable region would be expected to not have surface manifestations of liquefaction. On the other hand, for at least 24 of the other nonliquefaction cases plotted in the liquefaction region, the capping layer is considered not a factor based on

Ishihara's criteria and/or other available measurements, such as low pore pressure measurements recorded by piezometers installed in the selected critical layer. Additional evidence supporting the overlap of liquefaction and nonliquefaction cases is provided by the laboratory cyclic triaxial test results reported by Tokimatsu and Uchida (1990) for sands with less than 10% fines. As noted by Andrus and Stokoe (2000), the lower bound of the laboratory test results plots close to the CRR-V_{S1} curve for $\leqslant$5% fines (see Figure 1). Tokimatsu and Uchida's (1990) "best fit" curve plots significantly above the lower-bound curve and through the middle of the nonliquefaction cases plotted in the liquefaction region. Thus some of the overlap between liquefaction and nonliquefaction cases in Figure 1 is believed to be the result of differing physical soil behavior.

The CRR-V_{S1} curves shown in Figure 1 are defined by (modified from Andrus and Stokoe 2000)

$$CRR = MSF\left\{0.022\left(\frac{K_{a1}V_{S1}}{100}\right)^2 + 2.8\left(\frac{1}{V_{S1}^{*} - (K_{a1}V_{S1})} - \frac{1}{V_{S1}^{*}}\right)\right\}K_{a2} \tag{5}$$

where MSF is the magnitude scaling factor, V_{S1}^{*} is the limiting upper value of V_{S1} for liquefaction occurrence, K_{a1} is a factor to correct for high V_{S1} values caused by aging, and K_{a2} is a factor to correct for influence of age on CRR. The first (or squared) term in Equation 5 is based on a relationship between CRR and V_{S1} for constant average cyclic strain derived by R. Dobry, as cited in Andrus and Stokoe (2000). The second term is a hyperbola with a small value at low values of V_{S1} and a large value as V_{S1} approaches V_{S1}^{*}.

Magnitude Scaling Factor

The magnitude scaling factor is traditionally applied to CRR, rather than the CSR, and equals 1 for earthquakes with a magnitude of 7.5. For magnitudes other than 7.5, Youd et al. (2001) recommended $MSFs$ calculated from the following relationship:

$$MSF = \left(\frac{M_w}{7.5}\right)^{-2.56} \tag{6}$$

where M_w is moment magnitude, the preferred scale for liquefaction resistance calculations. Equation 6 is based on revised $MSFs$ calculated by Idriss (1997), and should be used with the r_d factors defined by Equation 2.

Idriss (1999) proposed new $MSFs$ based on laboratory data from Yoshimi et al. (1984) and a revised relationship between representative cycles of loading and earthquake magnitude. As discussed by Andrus and Stokoe (2000), there is little difference in using these new $MSFs$, along with corresponding r_d values, proposed by Idriss (1999) and the factors defined by Equations 2 and 6 for earthquakes with magnitudes of about 7-7.5, the range of the majority of the V_S-based case history data. At magnitudes less than about 6, however, the difference is significant. Unfortunately, at this time, there are insufficient well-documented liquefaction case histories for earthquake magnitudes less than 6 and greater than 8 to resolve differences between factors.

Limiting Upper Value of V_{S1}

The assumption of a limiting (or maximum) upper value of V_{S1} for liquefaction occurrence is equivalent to the assumption commonly made in the penetration-based procedures dealing with clean sands, where liquefaction is considered not possible above a corrected SPT blow count of about 30 (Seed et al. 1985) and a corrected cone tip resistance of about 160 (Robertson and Wride 1998). Current estimates of V_{S1}^* rely, in part, on penetration-V_S equations and, in part on the case histories shown in Figure 1. Andrus and Stokoe (2000) suggested the following relationship for estimating V_{S1}^*:

$$V_{S1}^* = 215 \text{ m/s} \quad \text{for } FC \leqslant 5\% \tag{7a}$$

$$V_{S1}^* = 215 - 0.5(FC - 5) \text{ m/s} \quad \text{for } 5\% < FC < 35\% \tag{7b}$$

$$V_{S1}^* = 200 \text{ m/s} \quad \text{for } FC \geqslant 35\% \tag{7c}$$

where FC is average fines content in percent by mass. Equations 5 and 7a yield a CRR value of about 0.6 at $V_{S1}=210$ m/s. Based on penetration-V_S correlations, a V_{S1} value of 210 m/s is considered equivalent to a corrected SPT blow count of 30 in clean sands.

Because several of the case history data shown in Figure 1 above CRR of 0.2 are for soils with significant amounts of gravel, Andrus and Stokoe (2000) also suggested Equations 7a–7c as preliminary limiting upper values of V_{S1} for gravelly soils. A penetration-V_S equation by Andrus (1994) based on tests at two Holocene-age sandy gravel sites that liquefied during the 1983 Borah Peak, Idaho, earthquake suggests a V_{S1} value of about 200 m/s as equivalent to a corrected blow count of 30. This finding provides further support for the use of Equation 7. On the other hand, penetration-V_S equations by Ohta and Goto (1978) and Rollins et al. (1998a) suggest a V_{S1} value of about 230 m/s for Holocene gravels at an equivalent corrected SPT blow count of 30. This uncertainty should be considered when using Equation 7 for design. Additional work is needed to better understand the relationship between V_{S1} and liquefaction resistance of gravels.

Age Correction Factors

The factors K_{a1} and K_{a2} are included in Equation 5 to extend the original CRR-V_{S1} equation by Andrus and Stokoe (2000) for uncemented Holocene-age soils to older soils. Two correction factors are suggested because it is believed that two mechanisms influence the position of the CRR-V_{S1} curve for older soils. The first mechanism involves the effect of aging on V_{S1}. The second mechanism involves the effect of aging on CRR. Both K_{a1} and K_{a2} are 1.0 for uncemented soils of Holocene age. For older soils, the following methods for estimating K_{a1} and K_{a2} are suggested based on information that is currently available.

The suggested method for approximating K_{a1} involves using SPT-V_{S1} relationships to estimate V_{S1} in Holocene-age soil for a similar $(N_1)_{60}$ value, and dividing the estimated V_{S1} value by the measured V_{S1} value. The SPT-V_{S1} equations by Ohta and Goto (1978) and Rollins et al. (1998a) suggest average K_{a1} values of 0.76 and 0.61, respectively, for Pleistocene soils (10,000 years to 1.8 million years). Better estimates may be obtained from local SPT-V_{S1} equations and measurements, as illustrated in Figure 2.

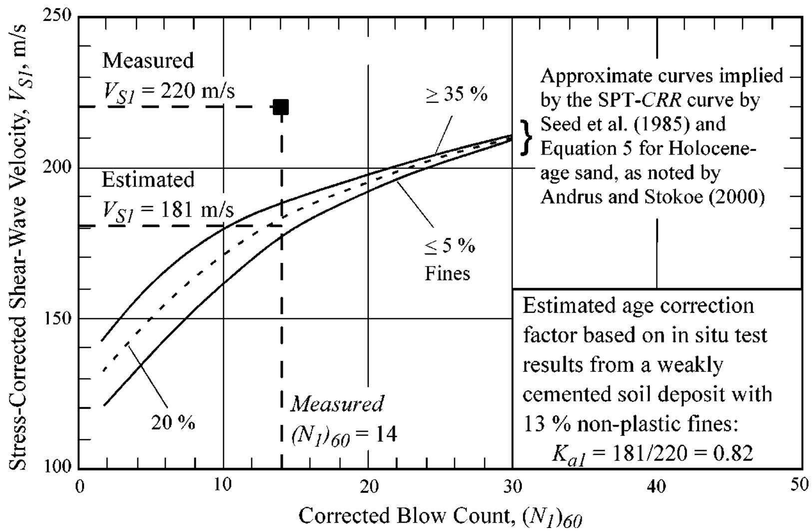

Figure 2. Suggested method for estimating K_{a1} from SPT and V_S measurements at the same site (after Andrus and Stokoe 2000).

Plotted in the figure are curves for Holocene-age sand with 5% and ⩾35% nonplastic fines implied by the *CRR*-V_{S1} curves defined by Equation 5 and *CRR*-SPT curves recommended by Youd et al. (2001). In the example, the measured values of V_{S1}, corrected blow count $(N_1)_{60}$, and fines content are 220 m/s, 14 blows/0.3 m, and 13%, respectively. These values are from actual measurements in a late Pleistocene-age (10,000 to 15,000 years) soil deposit at the Larter Ranch site, that are discussed later in this paper. Base on the curves in Figure 2, Holocene-age soil with $(N_1)_{60}=14$ and $FC=13\%$ generally have an average V_{S1} value of 181 m/s. By dividing 181 m/s by the measured value of 220 m/s, one can obtained a K_{a1} value of 0.82. This approach assumes SPT measurements are not affected by aging and cementation, and K_{a1} is the ratio of the estimated value to the measured value of V_{S1}. It also assumes that the strain level induced during SPT is the same strain level causing liquefaction, which may not be true because pore-water pressure buildup leading to liquefaction can occur at medium strains in several loading cycles (Dobry et al. 1982, Seed et al. 1983).

Approximate lower-bound values of K_{a2} are presented in Table 3. These values are based on the study by Arango et al. (2000) that involved (1) a review of data from sites shaken by the 1886 Charleston, South Carolina earthquake, and (2) stress-controlled cyclic triaxial testing of high-quality undisturbed samples from two sites. During the 1886 Charleston earthquake, soils as old as 200,000 years liquefied. Case history data from 33 sites ranging in age from 85,000 to over 200,000 years old suggested cyclic strengths are 1.3 to 3 times higher than those predicted by the SPT-based liquefaction chart for a Holocene clean sand of similar penetration resistance. Concerning the cyclic triaxial tests, the test samples were taken from two soil deposits about 2,000,000 and 30,000,000 years old. Strength gain factors obtained for the younger soil are 1.6 to 2.7 times greater than

Table 3. Approximate lower-bound estimates of K_{a2} based on study by Arango et al. (2000)

Time (years)	Lower-bound Estimate of K_{a2}
<10,000	1.0
10,000	1.1
100,000	1.3
1,000,000	1.5

predicted for Holocene clean sand with similar penetration resistance. For the older soil, strength gain factors obtained are 2.6 to 3.0 times greater. Although there is a high degree of uncertainty associated with these results, they demonstrate that cyclic strength increases with age. It is suggested in this paper that lower-bound values of these results be used as estimates of K_{a2}. Use of the lower-bound value provides a lower estimate of *CRR*. Additional work is needed to better quantify the influence of age on *CRR*, as well as V_S and penetration resistance.

LIQUEFACTION POTENTIAL

FACTOR OF SAFETY

One way to quantify the potential for liquefaction is in terms of a factor of safety. The factor of safety, F_S, against liquefaction is commonly defined by

$$F_S = \frac{CRR}{CSR} \tag{8}$$

By convention, liquefaction is predicted to occur when $F_S \leqslant 1$. When $F_S > 1$, liquefaction is predicted not to occur.

It is possible that liquefaction could occur outside the region of predicted liquefaction shown in Figure 1, as is also the case with the penetration-based liquefaction evaluation charts. Consequently, the BSSC (2000, Part 2, page 196) suggests a F_S value of 1.2 to 1.5 is appropriate when applying the Seed-Idriss simplified procedure in engineering design. The acceptable value of F_S for a particular site will depend on several factors, including the type and importance of structure and the potential for ground deformation. Based on SPT-V_S equations (Andrus and Stokoe 2000) and probability studies (Juang et al. 2002), the recommended V_S-based procedure is as conservative as the SPT-based procedure outlined by Seed et al. (1985) and updated by Youd et al. (2001). Thus the same range of F_S is recommended for the V_S-based method.

PROBABILITY OF LIQUEFACTION

A second way to quantify the potential for liquefaction is in terms of probability. One advantage of expressing liquefaction potential in terms of probability is that probability of liquefaction can be derived in a more objective manner than the deterministic bounding curves, which traditionally have been visually drawn. Another important advantage is that probability of liquefaction is required information for making risk-based design decisions.

Juang et al. (2001, 2002) developed three different probability models for the V_S case histories plotted in Figure 1. To develop the models, values of V_{S1} were adjusted to a clean soil equivalent. This adjustment involved two steps. First, a *CRR* value was determined for each case history using Equation 5. Second, for each *CRR* value, a clean soil equivalent V_{S1} value was determined using Equation 5 with V^*_{S1}=215 m/s while maintaining the ratio of *CRR* to *CSR* (or F_S). The clean soil adjustment can be expressed by

$$V_{S1cs} = K_{cs} \ V_{S1} \tag{9}$$

where V_{S1cs} is the equivalent clean soil value of V_{S1}, and K_{cs} is a fines content correction to adjust V_{S1} values to a clean soil equivalent. Values of K_{cs} can be approximated using the following equation (Juang et al. 2002):

$$K_{cs} = 1 \quad \text{for} \ FC \leqslant 5\% \tag{10a}$$

$$K_{cs} = 1 + (FC - 5)T \quad \text{for} \ 5\% < FC \leqslant 35\% \tag{10b}$$

$$K_{cs} = 1 + 30T \quad \text{for} \ FC \geqslant 35\% \tag{10c}$$

where

$$T = 0.009 - 0.0109\left(\frac{V_{S1}}{100}\right) + 0.0038\left(\frac{V_{S1}}{100}\right)^2 \tag{11}$$

The fines-corrected case history data and three curves from the probability models developed by Juang et al. (2002) are plotted in Figure 3. The three curves correspond to a probability of liquefaction, P_L, of 0.26. Model 1 is a logistic regression-based model, similar in form to the model used by Liao et al. (1988) for analyzing SPT-based case histories. Model 2 is also a logistic regression-based model, but differs from Model 1 by one additional term. These results clearly show that P_L curves determined by logistic regression techniques depend on the form of the regression equation, particularly outside the range of the case history data. Model 3 is based on Bayesian interpretation techniques developed by Juang et al. (1999). In the Bayesian approach, values of F_S were first determined for the liquefaction and nonliquefaction case histories using the *CRR* curve for $FC \leqslant 5\%$ shown in Figure 1. Values of P_L were then estimated from the probability density functions of F_S for liquefaction and nonliquefaction case histories by applying Bayes' theorem. The resulting F_S-P_L relationship can be approximated by (Juang et al. 2002)

$$P_L = \frac{1}{1 + \left(\dfrac{F_S}{0.73}\right)^{3.4}} \tag{12}$$

For a F_S value of 1, Equation 12 provides a P_L value of 0.26. Thus the Model 3 curve

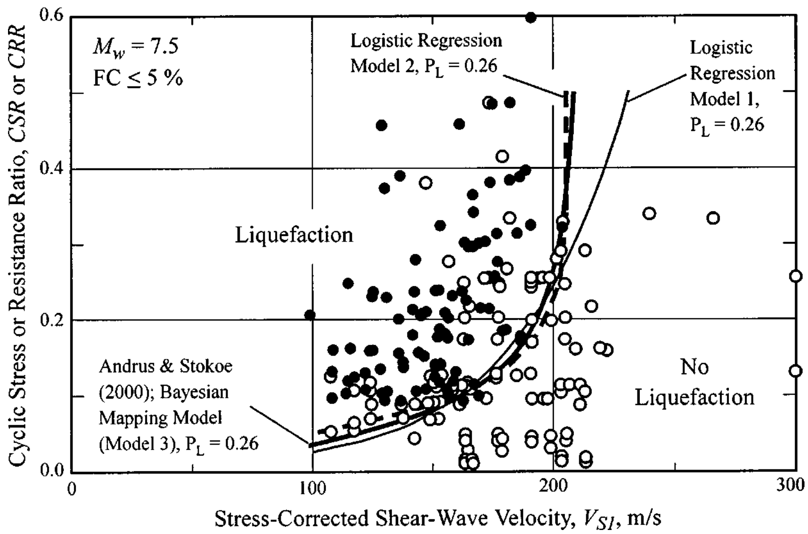

Figure 3. Comparison of three probability curves determined by Juang et al. (2002) for $P_L = 0.26$ along with case history data corrected for stress and fines content.

shown in Figure 3 and the *CRR* curve for $FC \leqslant 5\%$ shown in Figure 1 are the same. Because all three curves shown in Figure 3 are in close agreement below a V_{S1cs} value of about 200 m/s, the deterministic *CRR*-V_{S1} curves defined by Equation 5 are curves of about $P_L = 0.26$. This value is slightly less than the P_L value of 0.31 determined by Juang et al. (2002) for the deterministic SPT-based procedure by Seed et al. (1985) and updated by Youd et al. (2001), indicating that slightly different degrees of conservatism were assumed in drawing the two deterministic curves.

The F_S-P_L relationship defined by Equation 12 provides an important link between the probabilistic and deterministic methods. By combining Equations 5, 8, and 12, one can obtain a family of P_L curves for probability-based design. The family of P_L curves for magnitude 7.5 earthquakes and soils with $FC \leqslant 5\%$ is presented in Figure 4. These curves represent the complete Bayesian mapping model developed by Juang et al. (2002). They converge to a V_{S1cs} value of 215 m/s, the assumed value of V_{S1}^* for clean soils, at high values of *CSR*. The tendency for the P_L curves to converge to some maximum upper value reflects the tendency of dense soils to exhibit dilative behavior at large strains, causing decreased pore-water pressure. If pore-water pressure decreases, surface manifestations of liquefaction are less likely to occur. The wider spread exhibited in similar logistic regression-based P_L curves at high values of V_S and *CSR* (Juang et al. 2002) is believed to be the result of an inherent property of the models, and not physical soil behavior. Thus the model shown in Figure 4 is considered to be an improvement over the logistic regression models, and is suggested for engineering risk-based design. From Equation 12, P_L values of 0.16 and 0.08 are considered equivalent to the BSSC (2000) suggested F_S values of 1.2 and 1.5 when applying the V_S-based procedure described in this paper.

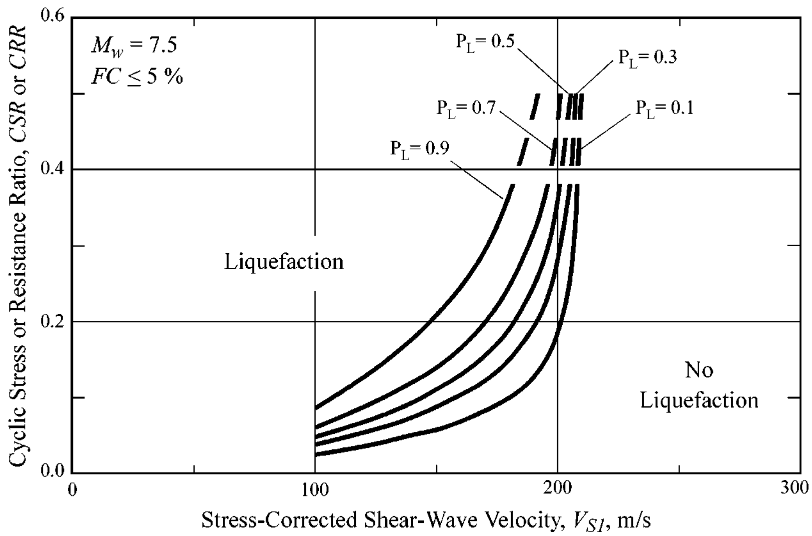

Figure 4. Curves suggested for probability-based evaluation in clean, uncemented soils of Holocene age (after Juang et al. 2002).

APPLICATION OF THE RECOMMENDED PROCEDURE

To illustrate the recommended procedure, the liquefaction potential evaluations for two sites that liquefied during the 1983 Borah Peak, Idaho, earthquake (M_w=6.9) are presented. The two sites are called Andersen Gravel Bar and Larter Ranch. Liquefaction effects at these sites were first described in the reconnaissance report by Youd et al. (1985). During subsequent field investigations, values of V_S were measured by the crosshole and SASW test methods (Stokoe et al. 1988a, Andrus et al. 1992, Andrus 1994).

ANDERSEN GRAVEL BAR SITE

The Andersen Gravel Bar site is named after Mr. Wendall Andersen, who was out fishing on the morning of the 1983 earthquake. He had waded across a small channel of the Big Lost River to a gravel sandbar when the earthquake struck. The following is Mr. Andersen's account (Youd et al. 1985):

> "I was standing on a gravel sandbar when the quake struck. Cracks appeared in the bar and began to gurgle water. Then three or four water spouts with 3 to 4 in. [75 to 100 mm] holes opened up and water shot up to 3 ft [0.9 m] in the air. The gravel bar shook like a marshmallow, and it was very difficult to stand. Some of the water spouts spewed black water; others spewed clear water."

Shown in Figures 5a–5f are the values of V_S measured by crosshole testing and the liquefaction evaluation for sediments within the zone of cracks and waterspouts identified by Mr. Andersen. Profiles of soil type and fines content shown in Figures 5b and 5c are based on test pit samples taken near the crosshole test location. The upper 2.5 m of soil at the site consists of sandy gravel with less than a few percent fines. The groundwa-

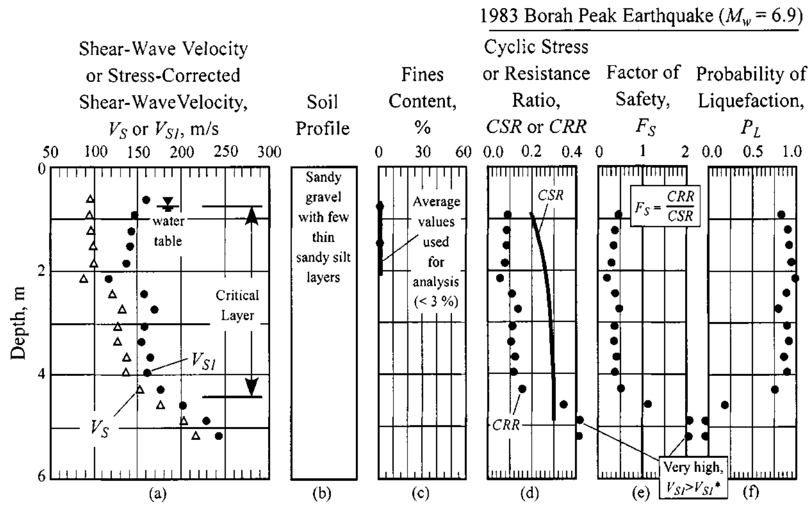

Figure 5. Application of the recommended procedure to the Andersen Gravel Bar site.

ter table at the time of the field investigations (August 1991) was located at a depth of about 0.7 m. Measured values of compression-wave velocity below the water table ranged from 1690 m/s to 2070 m/s and averaged 1910 m/s, indicating full saturation. Values of V_{S1} and CSR are shown in Figures 5a and 5d, respectively. These values are calculated assuming total densities of 2.08 Mg/m^3 above the water table and 2.08–2.16 Mg/m^3 below the water table. Also assumed in the evaluation are the average values of r_d originally proposed by Seed and Idriss (1971). Located 12 km from the 1983 surface rupture, peak horizontal ground surface acceleration at the Andersen Gravel Bar site is estimated to be about 0.29 g, based on various attenuation relationships. Values of CRR shown in Figure 5d are calculated assuming a MSF value of 1.24, based on Equation 6. Because the site is located within an active river channel, soils are considered to be of modern age. Therefore, K_{a1} and K_{a2} are assumed equal to 1.

Values of F_S are less than 0.5 below the water table to a depth of about 4.2 m, indicating high potential for liquefaction. These F_S values correspond to P_L values of 0.75 and higher. Thus, by the V_S procedure, the layer predicted to liquefy, or the critical layer, lies between the depths of 0.8 m and 4.4 m. A prediction of high liquefaction potential agrees with the field behavior observed by Mr. Andersen.

LARTER RANCH SITE

The Larter Ranch site is located about 15 km upstream from the Andersen Gravel Bar site on the Elkhorn alluvial fan and adjacent to the Thousand Springs Creek. Liquefaction beneath the distal end of the Elkhorn fan and a smaller unnamed fan generated a 2.1-km-long, nearly continuous zone of fissures, buckled soil, and sand boils. The slide area was about 75 m wide. The maximum horizontal ground movement was about 1 m towards Thousand Springs Creek. Mr. Gary Larter, who lived 0.8 km from the site, saw a huge dust cloud rising up along the creek just after the earthquake. Wondering what

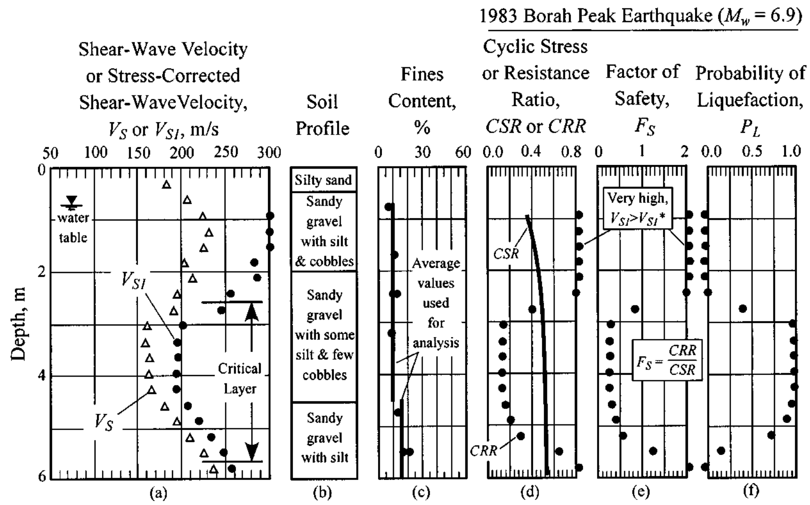

Figure 6. Application of the recommended procedure to the Larter Ranch site.

had happened, he drove to the area of the dust cloud. Upon reaching the area, he saw numerous waterspouts flowing up to 0.9 m into the air along the toe of the slide. Mr. Larter estimated the waterspouts flowed for 30 minutes after the earthquake.

Shown in Figures 6a–6f are the values of V_S measured by crosshole testing and the liquefaction evaluation beneath the toe of the slide. Profiles of soil type and fines content shown in Figures 6b and 6c are based on split-spoon samples taken near the crosshole test location. The sediment ranges from sandy silt to sandy gravel with silt and cobbles. The groundwater table at the time of the field investigations (August 1991) was located at a depth of 0.7 m. Measured values of compression-wave velocity between the depths of the groundwater table and 5.2 m ranged from 250 m/s to 1200 m/s and averaged 380 m/s, indicating partial saturation. Values of V_{S1} and *CSR* are shown in Figures 6a and 6d, respectively. These values are calculated assuming total densities of 2.06–2.11 Mg/m^3 above the water table and 2.11–2.21 Mg/m^3 below the water table. Also assumed in the evaluation are the average values of r_d originally proposed by Seed and Idriss (1971). Located 2 km from the 1983 fault rupture, peak horizontal ground surface acceleration at the Larter Ranch Site is estimated to be about 0.5 g, based on various attenuation relationships. Values of *CRR* shown in Figure 6d are calculated assuming a *MSF* value of 1.24. From stratigraphic relationships, thicknesses of carbonate buildup on stones, and radiometric dates of pedogenic carbonate and charcoal, the age of the fan sediments is estimated to be about 10,000 to 15,000 years old (Andrus and Youd 1987, Andrus 1994). The V_{S1} and $(N_1)_{60}$ values used in the example illustrated in Figure 2 are based on measurements between 2.4 m and 5.8 m at this site. Thus K_{a1} and K_{a2} are assumed equal to 0.82 and 1.1, respectively.

As noted in Figure 6a, the layer predicted to liquefy, or the critical layer, lies between the depths of 2.7 m and 5.5 m. Between the depths of 3.0 m and 4.9 m, high liquefaction potential is predicted with values of F_S ranging from 0.24 to 0.40. This range of F_S values corresponds to P_L values of 0.98 to 0.88. It is possible that the actual liquefaction potential was lower, however, due to the fact that the soil in the critical layer was partially saturated. As mentioned earlier in the paper, the cyclic strength can be twice as high in partially saturated soil than in fully saturated soil (Ishihara et al. 1998, Tsukamoto et al. 2002). If *CSR* values in the critical layer (see Figure 6d) were increased by a factor of two, values of F_S would range from 0.48 to 0.80. That range of F_S values would correspond to P_L values of 0.80 to 0.43, which is still a fairly high liquefaction potential. A prediction of high liquefaction potential agrees with the observed field behavior.

CONCLUSIONS

Guidelines for evaluating liquefaction potential using V_S measurements and the procedure described in Andrus and Stokoe (2000) are presented. The guidelines can be summarized as follows:

1. When selecting the test method, it is important to keep in mind that there is more than one use of V_S measurements. Measurements made for one application, such as NEHRP site class determination, may not be at suitable intervals for liquefaction potential assessment due to the averaging in the NEHRP evaluation versus localized testing needed in the liquefiable material. Some test methods are more appropriate for screening large areas, while others are better for site-specific evaluations. It is the authors' position that final site-specific liquefaction evaluations using only or primarily the V_S method should be limited to situations where (1) crosshole, downhole, suspension logger, or SASW tests are conducted such that high-quality V_S values are determined at intervals of at least one-quarter the thickness of the critical layer, (2) appropriate consideration is given to the limitations listed in Table 2, (3) sufficient borings or soundings are conducted to identify the material type and to insure that thin liquefiable strata are not present, and (4) the critical layer is Holocene in age and contains little or no carbonate. In general, borings should always be a part of the field investigations.
2. The procedure by Andrus and Stokoe (2000) was developed using data limited to relatively level ground sites, uncemented soils of Holocene age, average depths less than about 10 m, groundwater table depths between 0.5 m and 6 m, and measurements from below the water table. Greater care should be exercised when applying the procedure to sites with different conditions.
3. Age correction factors were suggested for extending the procedure to sites older than Holocene age. These correction factors are based on penetration-V_S equations, cyclic triaxial tests, and limited case history data. More work is needed to better quantify the influence of age on V_S and liquefaction resistance of soils.
4. The F_S-P_L relationship developed by Juang et al. (2002) provides an important link between the deterministic and probabilistic methods for determining liquefaction potential. Based on this relationship, the *CRR*-V_S deterministic curves by Andrus and Stokoe (2000) correspond to P_L of about 0.26. This value is slightly less than the P_L value of 0.31 determined for the SPT-based procedure by Seed et al. (1985).

5. BSSC (2000) has suggested that a F_S value of 1.2 to 1.5 is appropriate when applying the Seed-Idriss simplified procedure in engineering design. The same range of F_S is recommended for the V_S-based procedure. When applying the V_S-based procedure, these F_S values are equivalent to P_L values of 0.16 to 0.08, respectively.

As a final comment, the number of V_S tests conducted in the U.S. has increased dramatically during the past few years. This increase use of V_S is expected to continue. As future earthquakes occur in the U.S. and other areas of the world, the current disadvantage of limited V_S measurements at liquefaction sites will no longer exist and factors suggested in this paper will likely be refined and improved.

ACKNOWLEDGMENTS

The development of this guide began while the first author was employed at the National Institute of Standards and Technology (NIST). The completion of the guide was partially funded under NIST Contract No. 43NANB912365. We thank Riley M. Chung, formerly with NIST, and Nicholas Carino and John Gross at NIST for their support and encouragement. The United States Geological Survey under Contract No. 14-08-0001-G1779 funded the investigation at the two case study sites in Idaho. We also thank the participants of the 1996 NCEER and 1998 NCEER/NSF workshops on Evaluation of Liquefaction Resistance of Soils for their comments and suggestions, in particular, Gonzalo Castro of GEI Consultants, Ricardo Dobry of the Rensselaer Polytechnic Institute, Mary Ellen Hynes of the U.S. Army Corps of Engineers, I. M. Idriss of the University of California at Davis, Maurice S. Power of Geomatrix Consultants, Peter K. Robertson of the University of Alberta, and T. Leslie Youd of Brigham Young University. The review comments of earlier versions of this work by Richard Woods of the University of Michigan, Ann Arbor, Robert Pyke, Consulting Engineer, and many others are sincerely appreciated.

REFERENCES

American Society for Testing and Materials (ASTM), 1991. *ASTM D-4428M-91, Standard Test Methods for Crosshole Seismic Testing*, West Conshohocken, PA.

Andrews, D. C. A., and Martin, G. R., 2000. Criteria for liquefaction of silty soils, *12th World Conference on Earthquake Engineering, Auckland, New Zealand, Proceedings*.

Andrus, R. D., 1994. In situ Characterization of Gravelly Soils That Liquefied in the 1983 Borah Peak Earthquake, Ph.D. dissertation, The University of Texas at Austin, 533 pp.

Andrus, R. D., and Stokoe, K. H., II, 1997. Liquefaction resistance based on shear wave velocity, *NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Salt Lake City, UT, Technical Report NCEER-97-0022*, T. L. Youd and I. M. Idriss, eds., National Center for Earthquake Engineering Research, Buffalo, NY, 89–128.

Andrus, R. D., and Stokoe, II, K. H., 2000. Liquefaction resistance of soils from shear wave velocity, *J. Geotech. Geoenviron. Eng.,* ASCE, **126** (11), 1015–1025.

Andrus, R. D., and Youd, T. L., 1987. Subsurface investigation of a liquefaction-induced lateral spread, Thousand Springs Valley, Idaho, *Geotechnical Laboratory Miscellaneous Paper GL-87-8*, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, 131 pp.

Andrus, R. D., Stokoe, K. H., II, Bay, J. A., and Youd, T. L., 1992. In situ V_S of gravelly soils which liquefied, *10th World Conference on Earthquake Engineering, Madrid, Spain, Proceedings*, Balkema, Rotterdam, The Netherlands, 1447–1452.

Andrus, R. D., Stokoe, K. H., II, Chung, R. M., and Juang, C. H., 2003. Guidelines for evaluating liquefaction resistance using shear wave velocity measurements and simplified procedures, *NIST GCR 03-854*, National Institute of Standards and Technology, Gaithersburg, MD, 151 pp.

Arango, I., Lewis, M. R., and Kramer, C., 2000. Updated liquefaction potential analysis eliminates foundation retrofitting of two critical structures, *Soil Dyn. Earthquake Eng.* **20**, 17–25.

Bierschwale, J. G., and Stokoe, K. H., II, 1984. Analytical evaluation of liquefaction potential of sands subjected to the 1981 Westmorland earthquake, *Geotechnical Engineering Report GR-84-15*, The University of Texas at Austin, 231 pp.

Building Seismic Safety Council (BSSC), 2000. *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, Parts 1 and 2, *FEMA-368*, Federal Emergency Management Agency, Washington, D.C.

de Alba, P., Baldwin, K., Janoo, V., Roe, G., and Celikkol, B., 1984. Elastic-wave velocities and liquefaction potential, *Geotech. Test. J.,* ASTM, **7** (2), 77–87.

Dobry, R., Ladd, R. S., Yokel, F. Y., Chung, R. M., Powell, D., 1982. Prediction of pore water pressure buildup and liquefaction of sands during earthquakes by the cyclic strain method, *NBS Building Science Series 138*, National Bureau of Standards, Gaithersburg, MD, 152 pp.

Dobry, R., Stokoe, K. H., II, Ladd, R. S., and Youd, T. L., 1981. Liquefaction susceptibility from S-wave velocity, *In Situ Tests to Evaluate Liquefaction Susceptibility, Proceedings*, ASCE National Convention, St. Louis, MO.

Drnevich, V. P., and Richart, Jr., F. E., 1970. Dynamic prestraining of dry sand, *J. Soil Mech. Found. Div.,* ASCE, **96** (SM2), 453–469.

Finn, W. D. L., and Bhatia, S. K., 1981. Prediction of seismic pore-water pressures, *10th International Conference on Soil Mechanics and Foundation Engineering, Proceedings*, Vol. 3, Balkema, Rotterdam, The Netherlands, pp. 201–206.

Frankel, A. D., Mueller, C. S., Barnhard, T. P., Leyendecker, E. V., Wesson, R. L., Harmsen, S. C., Klein, F. W., Perkins, D. M., and Dickman, N., 2000. USGS national seismic hazard maps, *Earthquake Spectra* **16** (1), 1–19.

Golesorkhi, R., 1989. Factors Influencing the Computational Determination of Earthquake-Induced Shear Stresses in Sandy Soils, Ph.D. dissertation, University of California, Berkeley, 369 pp.

Harder, L. F., Jr., and Boulanger, R. W., 1997. Application of K_σ and K_α correction factors, *NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Salt Lake City, UT, Technical Report NCEER-97-0022*, T. L. Youd and I. M. Idriss, eds., National Center for Earthquake Engineering Research, Buffalo, NY, pp. 167–190.

Hardin, B. O., and Drnevich, V. P., 1972. Shear modulus and damping in soils: Design equations and curves, *J. Soil Mech. Found. Div.,* ASCE, **98** (SM7), 667–692.

Hynes, M. E., 1988. Pore Pressure Generation Characteristics of Gravel under Undrained Cyclic Loading, Ph.D. dissertation, University of California, Berkeley.

Hynes, M. E., and Olsen, R. S. 1999. Influence of confining stress on liquefaction resistance *International Workshop on Physics and Mechanics of Soil Liquefaction, Proceedings*, Balkema, Rotterdam, The Netherlands, 145–152.

Idriss, I. M., 1997. Personal communication to T. L. Youd.

Idriss, I. M., 1999. An update of the Seed-Idriss simplified procedure for evaluating liquefaction potential, *TRB Workshop on New Approaches to Liquefaction Analysis, Proceedings*, Transportation Research Board, Washington, D.C.

International Code Council (ICC), 2000. *International Building Code*, Falls Church, VA.

Ishihara, K., 1985. Stability of natural deposits during earthquakes, *11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, CA, Proceedings*, vol. 1, pp. 321–376.

Ishihara, K., 1996. *Soil Behavior in Earthquake Geotechnics*, Oxford University Press Inc., New York, NY, 350 pp.

Ishihara, K., Huang, Y., Tsuchiya, H., 1998. Liquefaction resistance of nearly saturated sand as correlated with longitudinal wave velocity, *Poromechanics: A Tribute to Maurice A. Biot*, Balkema, 583–586.

Juang, C. H., and Chen, C. J., 2000. A rational method for development of limit state for liquefaction evaluation based on shear wave velocity, *Int. J. Numer. Analyt. Meth. Geomech.* **24**, 1–27.

Juang, C. H., Rosowsky, D. V., and Tang, W. H., 1999. A reliability-based method for assessing liquefaction potential of sandy soils, *J. Geotech. Geoenviron. Eng.,* ASCE, **125** (8), 684–689.

Juang, C. H., Andrus, R. D., Jiang, T., and Chen, C. J., 2001. Probability-based liquefaction evaluation using shear wave velocity measurements, *4th International Conference on Recent Advances in Geotechnical Engineering and Soil Dynamics, San Diego, California, Proceedings*, S. Prakash, ed., University of Missouri-Rolla, Paper 4.24.

Juang, C. H., Jiang, T., and Andrus, R. D., 2002. Assessing probability-based methods for liquefaction potential evaluations, *J. Geotech. Geoenviron. Eng.,* ASCE, **128** (7), 580–589.

Kayabali, K., 1996. Soil liquefaction evaluation using shear wave velocity, *Engineering Geology*, Elsevier Publisher, New York, NY, **44** (4), 121–127.

Kayen, R. E., Mitchell, J. K., Seed, R. B., Lodge, A., Nishio, S., and Coutinho, R., 1992. Evaluation of SPT-, CPT-, and shear wave-based methods for liquefaction potential assessment using Loma Prieta data, *Fourth Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures for Soil Liquefaction, Honolulu, Hawaii, Proceedings, Technical Rep. NCEER-92-0019*, M. Hamada and T. D. O'Rourke, eds., National Center for Earthquake Engineering Research, Buffalo, NY, **1**, 177–204.

Kramer, S. L., 1996. *Geotechnical Earthquake Engineering*, Prentice Hall, Upper Saddle River, NJ.

Liao, S. S. C., and Whitman, R. V., 1986. Overburden correction factors for SPT in sands, *J. Geotech. Eng.,* ASCE, **112** (3), 373–377.

Liao, S. S. C., Veneziano, D., and Whitman, R. V., 1988. Regression models for evaluating liquefaction probability, *J. Geotech. Eng.,* ASCE, **114** (4), 389–411.

Lodge, A. L., 1994. Shear Wave Velocity Measurements for Subsurface Characterization, Ph.D. dissertation, University of California at Berkeley.

Martin, G. R., Finn, W. D. L., and Seed, H. B., 1975. Fundamentals of liquefaction under cyclic loading, *J. Geotech. Eng. Div.,* ASCE, **101** (GT5), 423–483.

National Research Council, 1985. *Liquefaction of Soils During Earthquakes*, National Academy Press, Washington, D.C., 240 pp.

Ohta, Y., and Goto, N., 1978. Empirical shear wave velocity equations in terms of characteristic soil indexes, *Earthquake Eng. Struct. Dyn.* **6**, 167–187.

Park, T., and Silver, M. L., 1975. Dynamic soil properties required to predict the dynamic behavior of elevated transportation structures, *Report DOT-TST-75-44*, U.S. Department of Transportation, Washington, D.C.

Poulos, S. J., Castro, G., and France, J. W., 1985. Liquefaction evaluation procedure, *J. Geotech. Eng.,* ASCE, **111** (6), 772–792.

Pyke, R. M., Seed, H. B., and Chan, C. K., 1975. Settlement of sands under multi-directional shaking, *J. Geotech. Eng. Div., Am. Soc. Civ. Eng.,* ASCE, **101** (GT4), 379–398.

Rashidian, M., 1995. Undrained Shearing Behavior of Gravelly Sands and Its Relation with Shear Wave Velocity, Ph.D. dissertation, University of Tokyo, Japan, 343 pp.

Rauch, A. F., Duffy, M., and Stokoe, K. H., II, 2000. Laboratory correlation of liquefaction resistance with shear wave velocity, *Computer Simulation of Earthquake Effects, Geotech. Special Pub. No. 110*, K. Arulanandan, A. Anandarajah, and X. S. Li, eds., ASCE, 686–695.

Robertson, P. K., Woeller, D. J., and Finn, W. D. L., 1992. Seismic Cone Penetration Test for evaluating liquefaction potential under cyclic loading, *Can. Geotech. J.* **29** (4), 686–695.

Robertson, P. K., and Wride, C. E., 1998. Evaluating cyclic liquefaction potential using the Cone Penetration Test, *Can. Geotech. J.* **35** (3), 442–459.

Roessler, S. K., 1979. Anisotropic shear modulus due to stress anisotropy, *J. Geotech. Eng. Div.,* ASCE, **105** (GT7), 871–880.

Rollins, K. M., Evans, M. D., Diehl, N. B., and Daily, III., W. D., 1998a. Shear modulus and damping relationships for gravels, *J. Geotech. Geoenviron. Eng.,* ASCE, **124** (5), 396–405.

Rollins, K. M., Diehl, N. B., and Weaver, T. J., 1998b. Implications of V_S-BPT $(N_1)_{60}$ correlations for liquefaction assessment in gravels, *Geotechnical Earthquake Engineering and Soil Dynamics III, Geotech. Special Pub. No. 75*, P. Dakoulas, M. Yegian, and B. Holtz, eds., ASCE, **I**, 506–517.

Sancio, R. B., Bray, J. D., Stewart, J. P., Youd, T. L., Durgunoglu, H. T., Baturay, M. B., Cetin, K. O., Christensen, C., Karadayilar, T., and Emrem, C., 2002. Correlation between ground failure and soil conditions in Adapazari, Turkey, *Soil Dyn. Earthquake Eng.* **22** (9–12), 1093–1102.

Sancio, R. B., Bray, J. D., Riemer, M. F., and Durgunoglu, H. T., 2003. An assessment of the liquefaction susceptibility of Adapazari silt, *2003 Pacific Conference on Earthquake Engineering, Christchurch, New Zealand, Proceedings*.

Santamarina, J. C., Klein, K., and Fam, M., 2001. *Soils and Waves*, John Wiley and Sons, Chichester, England, 488 pp.

Seed, H. B., 1979. Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes, *J. Geotech. Eng. Div.,* ASCE, **105** (GT2), 201–255.

Seed, H. B., and Idriss, I. M., 1971. Simplified procedure for evaluating soil liquefaction potential, *J. Soil Mech. Found. Div.,* ASCE, **97** (SM9), 1249–1273.

Seed, H. B., and Idriss, I. M., 1982. *Ground Motions and Soil Liquefaction During Earthquakes*, Earthquake Engineering Research Institute, Berkeley, CA, 134 pp.

Seed, H. B., Idriss, I. M., and Arango, I., 1983. Evaluation of liquefaction potential using field performance data, *J. Geotech. Eng.,* ASCE, **109** (3), 458–482.

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

Stokoe, K. H., II, and Nazarian, S., 1985. Use of Rayleigh waves in liquefaction studies, *Measurements and Use of Shear Wave Velocity for Evaluating Dynamic Soil Properties*, ASCE, 1–17.

Stokoe, K. H., II, Lee, S. H. H., and Knox, D. P., 1985. Shear moduli measurements under true triaxial stresses, *Advances in the Art of Testing Soil Under Cyclic Conditions, Proceedings*, ASCE, 166–185.

Stokoe, K. H., II, Andrus, R. D., Rix, G. J., Sanchez-Salinero, I., Sheu, J.-C., and Mok, Y. J., 1988a. Field investigations of gravelly soils which did and did not liquefy during the 1983 Borah Peak, Idaho, earthquake, *Geotech. Engineering Report GR 87-1*, The University of Texas at Austin.

Stokoe, K. H., II, Roesset, J. M., Bierschwale, J. G., and Aouad, M., 1988b. Liquefaction potential of sands from shear wave velocity, *9th World Conference on Earthquake Engineering, Tokyo, Japan, Proceedings*, **III**, 213–218.

Sykora, D. W., 1987. Creation of a data base of seismic shear wave velocities for correlation analysis, *Geotechnical Laboratory Miscellaneous Paper GL-87-26*, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Tokimatsu, K., Kuwayama, S., and Tamura, S., 1991. Liquefaction potential evaluation based on Rayleigh wave investigation and its comparison with field behavior, *2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, MO, Proceedings*, S. Prakash, ed., University of Missouri at Rolla, **I**, 357–364.

Tokimatsu, K., and Uchida, A., 1990. Correlation between liquefaction resistance and shear wave velocity, *Soils Found., Japanese Society of Soil Mechanics and Foundation Engineering,* **30** (2), 33–42.

Tsukamoto, Y., Ishihara, K., Nakazawa, H., Kamada, K., and Huang, Y., 2002. Resistance of partly saturated sand to liquefaction with reference to longitudinal and shear wave velocities, *Soils Found., Japanese Society of Soil Mechanics and Foundation Engineering,* **42** (6), 93–104.

Woods, R. D., ed., 1994. *Geophysical Characterization of Sites*, Balkema, Rotterdam, The Netherlands.

Yoshimi, Y., Tokimatsu, K., Kaneko, O., and Makihara, Y., 1984. Undrained cyclic shear strength of dense Niigata sand, *Soils Found., Japanese Society of Soil Mechanics and Foundation Engineering,* **24** (4), 131–145.

Youd, T. L., 1972. Compaction of sands by repeated shear straining, *J. Soil Mech. Found. Div.* ASCE, **98** (SM7), 709–725.

Youd, T. L., Harp, E. L., Keefer, D. K., and Wilson, R. C., 1985. The Borah Peak, Idaho earthquake of October 28, 1983, chap. on Liquefaction, *Earthquake Spectra* **2** (4), 71–89.

Youd, T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Harder, Jr., L. F., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson, III, W. F., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B., and Stokoe, II, K. H., 2001. Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, *J. Geotech. Geoenviron. Eng.,* ASCE, **127** (10), 817–833.

(Received 6 August 2002; accepted 8 September 2003)