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In situ V_s of gravelly soils which liquefied

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ABSTRACT: Spectral-Analysis-of-Surface-Waves (SASW) and crosshole seismic tests were conducted at four sites where liquefaction had occurred during the 1983 Borah Peak, Idaho earthquake (M_S = 7.3). Samples taken from the layer most likely to have liquefied at these four sites range from clean gravelly sand to sandy gravel with silt. Minimum values of shear wave velocity, V_S, were on the order of 90 m/sec in clean gravelly soils at two gently dipping (less than 5 percent) sites and were on the order of 160 m/sec in dirty gravelly soils at two more steeply sloping (more than 12 percent) sites. Two liquefaction assessment methods based on V_S and developed for sands directly applied to the V_S measurements in these gravelly soils correctly predict the observed liquefaction behavior at all four sites.

1 INTRODUCTION

Since the Niigata and Alaskan earthquakes of 1964, efforts in assessing the liquefaction potential of soils have been mainly directed towards clean sands and silty sands. Until recently, little information had been gathered on the characteristics and field performance of gravelly soils. Liquefaction of gravelly soils has been reported in Japan, China, New Zealand, and the United States (Andrus, 1992). These reports suggest that loose gravelly soils can be as susceptible to liquefaction as sands when they are capped by an impermeable layer (such as clay, silt, fine sand, ice, asphalt or concrete) or when they contain a significant amount of fine sand and silt.

During the 1983 Borah Peak, Idaho earthquake (M_S = 7.3), liquefaction was generated at numerous locations within and adjacent to the floodplains of the Big Lost River and Thousand Springs Creek. In the reconnaissance report by Youd et al. (1985), liquefaction effects included a lateral spread of a low-lying river terrace at the Pence Ranch, several fissured gravel bars within the modern channel of the Big Lost River, and a lateral spread of the distal ends of two alluvial fans along the Thousand Springs Creek. Because surficial soils and sand boils were reported to contain gravel, the four sites shown in Figure 1 were selected for investigation. The sites are called Pence Ranch, Andersen Bar, Whiskey Springs and Larter Ranch. Each site and the associated liquefaction effects are briefly described below.

1.1 *Gravelly sites*

<ins>Pence Ranch</ins>--The Pence Ranch site is located on a gently sloping, less than 5 percent, river terrace about 8 km southeast of the southern end of the 1983 surface fault rupture, see Figure 1. The zone of lateral spreading at Pence Ranch was over 240 m long and 30 m wide.

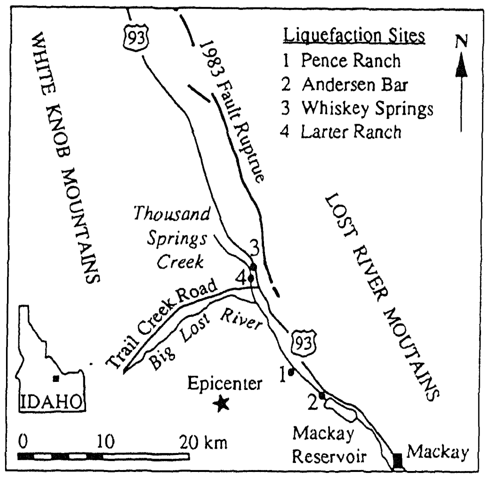

Figure 1. Regional map of the Big Lost River and Thousand Springs Creek areas showing approximate trace of fault rupture (Youd et al., 1985) and the four liquefaction sites.

Fissures, as wide as 30 cm, and gravelly sand boils marked the head of the lateral spread. The Pence home and a steel-frame barn were severely damaged as the ground beneath them opened in extension. Maximum lateral movement was on the order of 30 cm, downslope into the marsh (Youd et al., 1985).

<ins>Andersen Bar</ins>--A fisherman, Mr. Andersen, who happen to be wading across the Big Lost River at the time of the earthquake gave the following 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."

This particular gravel bar (called the Andersen Bar site herein) is located about 4.6 km downstream from Pence Ranch and 12 km southeast of the 1983 surface rupture, see Figure 1. In 1991, the bar was over 60 m long and up to 24 m wide. The top of the bar dipped very gently downstream and the sides of the bar sloped gently into the channel. However, a 60-cm-high slope at the angle of repose formed the downstream edge.

Whiskey Springs and Larter Ranch--The zone of lateral spreading along the Thousand Springs Creek was about 2.1 km long and 75 m wide. Large subparallel fissures and cracks developed on two alluvial fans and the soil at the toe of the fans buckled into ridges as high as 1.2 m. Maximum lateral movement was estimated to be 0.8 to 1.0 m (Andrus and Youd, 1987). Mr. Larter, who drove from his house to the slide area just after the earthquake, saw several 90-cm-high water spouts coming up through cracks along the toe. He estimates that water flowed for about 30 minutes after the earthquake. Large amounts of sand and silt were carried with the water to the ground surface. Investigations were conducted at the northern end of the lateral spread near Whiskey Springs and on the Larter Ranch, about 1 km south and downstream of the Whiskey Springs site. The fan slope at the Whiskey Springs and Larter Ranch sites are 12 and 34 percent, respectively. The lateral spread lies about 1.6 km west of the 1983 surface rupture as shown in Figure 1.

Field investigations were conducted at these four sites in 1983, 1984, 1985, 1990 and 1991. The field work included reconnaissance studies, seismic testing, penetration testing, drilling, sampling, trenching and in-place density measurements. Results from the earlier studies at Pence Ranch and Whiskey Springs were present in project reports by Andrus and Youd (1987), Stokoe et al. (1988a), and Harder (1988). Penetration and sample data from Pence Ranch and Whiskey Springs were summarized in publications by Andrus and Youd (1989), Stokoe et al. (1989a), and Andrus et al. (1991). The purpose of this paper is to discuss the shear wave velocity, V_S, measurements determined by the Spectral-Analysis-of-Surface-Waves (SASW) and crosshole techniques.

2 IN SITU SEISMIC MEASUREMENTS

The SASW method is based on the principal that surface seismic waves of different frequencies (or wavelengths) sample different depths in a layered medium and, therefore, will propagate at different velocities if stiffness varies with depth. SASW testing procedures outlined by Stokoe et al. (1988b) were followed. Field tests were conducted by placing two vertical receivers on the ground at preselected spacings equally distance from a common centerline, as shown in Figure 2. Hammers, dropped weights and a bulldozer were used as sources to generate the surface waves. For each receiver spacing, the time delay between the two receivers was calculated as a function of frequency from the phase of the cross-power spectrum. Surface wave phase velocities were then determined by dividing the distance between receivers by the time delay for various frequencies. A plot of phase velocity versus frequency, called a dispersion curve, was assembled from the results of all the receiver spacings. Finally, a shear wave velocity profile was obtained through an iterative process of matching the assembled dispersion curve to theoretical dispersion curves using two- and three-dimensional computer models described by Roesset et al. (1991). A major advantage of the SASW method is that no boreholes are needed; thus, the method is well-suited for undisturbed testing of gravelly soils.

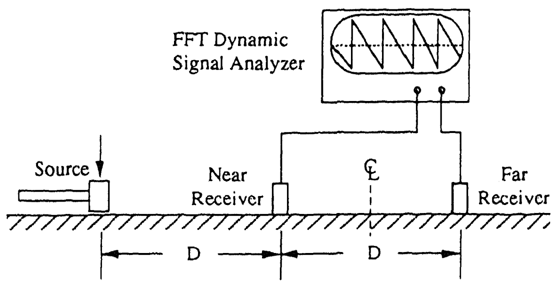

Figure 2. SASW field testing configruration.

Crosshole tests were performed at Pence Ranch, Andersen Bar and Larter Ranch by driving two AW steel casings (57 mm outside diameter) into the ground with the aid of a portable tripod, cathead and 63.5 kg hammer. The purpose of the two steel casings were to act as the receiver and source holes. First, the receiver casing was driven to its final depth (about 6 m). The source casing was then driven 2 to 2.5 m away from the receiver hole. Driving of the source casing was temporarily stopped after each 30-cm advancement for velocity measurements. Compression (P) and vertically-polarized shear (SV) waves were generated by tapping downward on the steel casing of the source hole, as shown in Figure 3. A small diameter (42 mm), three-component (3-D) geophone wedged in the receiver casing was used to measure the waveforms. The 3-D geophone was oriented using orientation rods which extended to the surface. The radial geophone was used to record the P-wave and the vertical geophone was used to record the SV-wave. For a zero-time reference, an accelerometer was clamped to the top of the source casing. Travel times were determined from the initial arrivals identified in the geophone and accelerometer records. Corrections were made to the travel time to account for the time required for the wave to travel to the tip of the source casing. Borehole inclination was estimated at the ground surface with a capenter's level. Seismic wave velocities were calculated by dividing the horizontal distance between the source and receiver casings by the corrected travel time.

At Whiskey Springs, crosshole tests were conducted through the ends of hollow-stem augers. The source borehole was first augered to a depth 15 cm above the test level. A split-spoon sampler attached to AW drilling

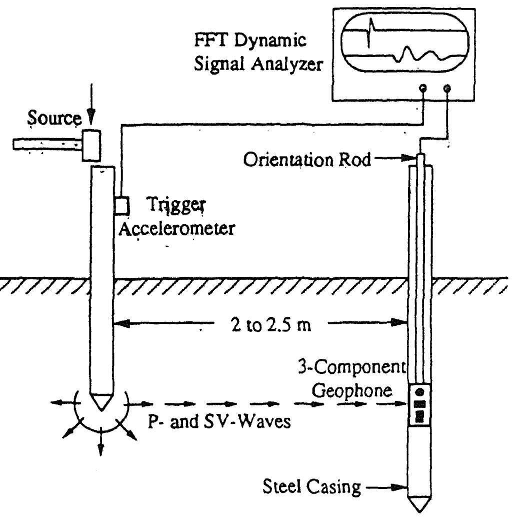

Figure 3. Crosshole seismic testing configuration.

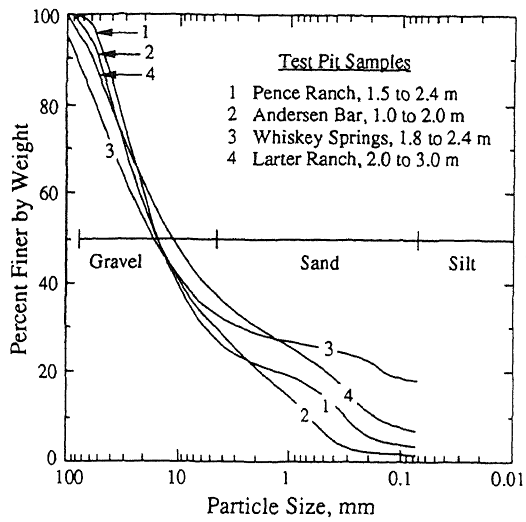

Figure 4. Grain-size distribution curves of test-pit samples taken from the critical layer at the four sites.

rod was then lowered down the source hole and seated 30 cm into the natural soil so that the center of the sampler was at the test level. Next, two receiver boreholes were auger to the test level. Three-dimensional geophones were oriented and lowered down the augers and seated at the test level. P- and SV-waves were then generated in the soil by a vertical downward hit on the source rod. The arriving waveforms were recorded by the geophone in each of the two boreholes allowing both direct and interval travel times to be determined.

3 IN SITU TEST RESULTS

Shear wave velocity, V_s, and penetration measurements from the critical layer at the four sites are summarized in Table 1. (Standard penetration, SPT, and cone penetration, CPT, tests were also performed at three of the sites [Andrus, 1992].) Grain-size distribution curves of test pit samples taken from the critical layer at each site are shown in Figure 4.

Table 1. Range of velocity and penetration data from the critical layer at each liquefaction site.

Site	Depth m	V_s m/sec	CPT q_c MPa	SPT N blows/ft
Pence Ranch	1.5 -3.3	92 - 150	3 - 18	3 - 16
Andersen Bar	1.0 - 3.2	87 - 130	----	----
Whiskey Springs	1.8 - 4.0	172 - 190	1 - 15	3 - 14
Larter Ranch	2.0 - 4.5	157 - 190	6 - 18	5 - 18

Pence Ranch and Andersen Bar--Sediment beneath the Pence Ranch and Andersen Bar sites ranges from clean gravelly sand to sandy gravel, with minor silt and sand lenses. The gravel-size particles are hard and subrounded with low sphericity. Depending on the amount of sand, the gravel occurs floating in a matrix of sand (matrix-supported) or in contact with other gravel (clast-supported) with sand partially filling the space between the gravel particles. These fluvial sediments are quite variable, as reflected in the six SASW velocity profiles from Pence Ranch shown in Figure 5. Liquefaction most likely occurred in the low-V_s material between about 1.5 and 3.3 m (Stokoe et al., 1988; Andrus et al., 1991). Result from the SV-crosshole test

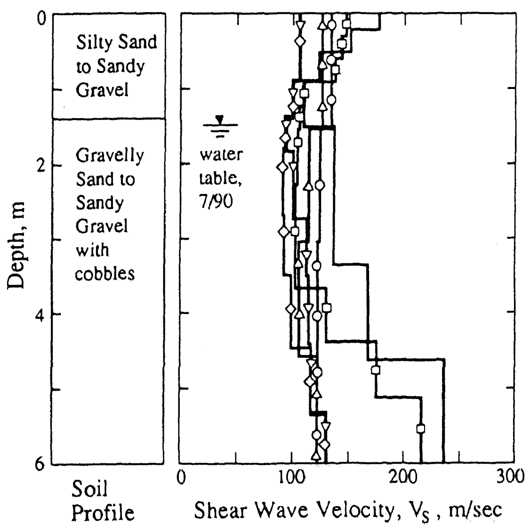

Figure 5. Six SASW shear wave velocity profiles at Pence Ranch.

at Pence Ranch along with the nearest SASW profile are shown in Figure 6. Although the values of V_S shown in Figure 6 are somewhat higher than at the other SASW test locations, V_S from both methods agree reasonably well. CPT logs near the crosshole location also show a greater penetration resistance than CPT logs at the other SASW locations.

Soil and V_S profiles obtained at the Andersen Bar site are shown in Figure 7. According to Mr. Andersen, however, the bar had been covered by a thin layer of gravelly sand and the river was flowing about 1 m higher at the time of the earthquake. The low velocity sandy gravel, less than 130 m/sec, that extends to a depth of about 3.2 m most likely liquefied. P-wave velocities, greater than 1500 m/sec, at both Andersen Bar and Pence Ranch show saturated conditions, from a dynamics point of view, below the water table.

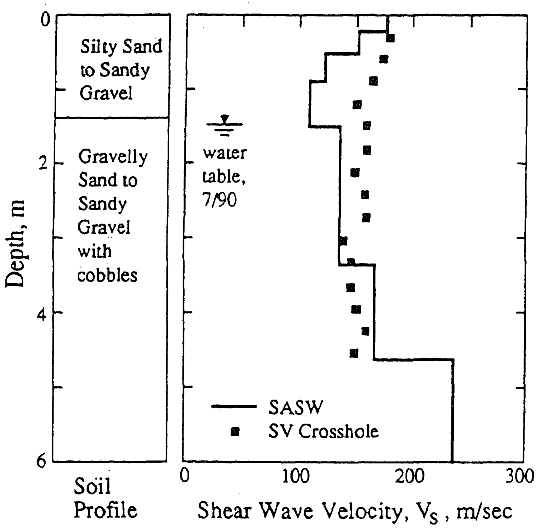

Figure 6. Shear wave velocity profiles at Pence Ranch.

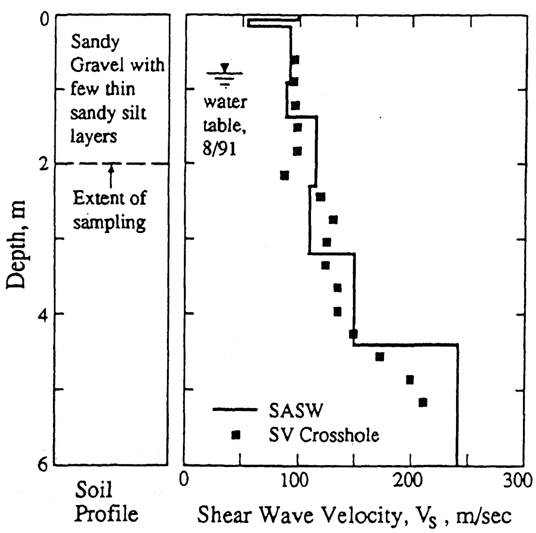

Figure 7. Shear wave velocity profiles at Andersen Bar.

Whiskey Springs and Larter Ranch--Beneath the Whiskey Springs and Larter Ranch sites, the sediment ranges from sandy silty gravel to sandy gravel with some silt, cobbles and even boulder sizes. The gravels are hard, subangular and in a matrix-supported to clast-supported structure. Soil and V_S profiles near the toe of the lateral spread at Whiskey Springs are shown in Figure 8. Liquefaction occurred in the low V_S silty sandy gravel between 1.8 and 4.0 m (Andrus and Youd, 1987; 1989). Fines content of the liquefiable material is about 18 percent. Measured V_S from both methods agree reasonably well; however, these V_S values are much higher than those measured in the liquefying layers at Pence Ranch and Andersen Bar.

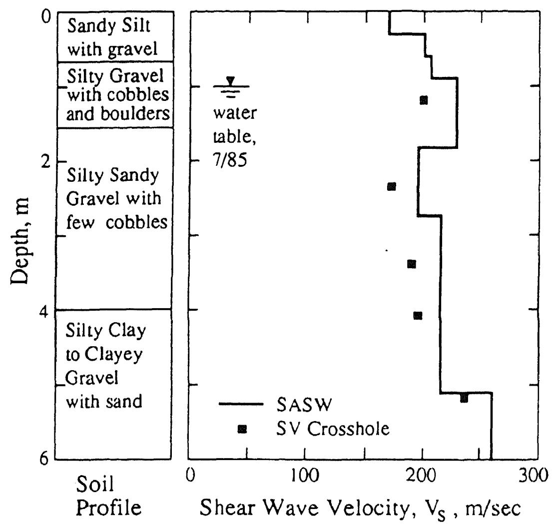

Figure 8. Shear wave velocity profiles at Whiskey Springs.

Soils and V_S values from the Larter Ranch site are presented in Figure 9. Soil layering at Larter Ranch is similar to Whiskey Springs, except the soil at Larter Ranch contains few boulders and less fines (7 percent fines compared to 18 percent at Whiskey Springs). The relatively low V_S material below 2 meters is considered most likely to have liquefied. It is interesting to note that the measured P-wave velocities down to 5.2 m were less than 450 m/sec at this site. This indicates unsaturated conditions (say 99 percent saturated) between the measured water table (1.5 m) and 5.2 m. The soil below 5.2 m is saturated, as shown by P-wave velocities greater than 1425 m/sec.

4 QUALITATIVE MEASURE OF IN SITU DENSITY

Seed et al. (1985) have suggested an empirical constant, K_2, as a qualitative measure of in situ density which can be determined as follows:

$$G_{max} = 1000\, K_2\, (\sigma'_m)^{0.5} \qquad (1)$$

where G_{max} is the low-amplitude shear modulus in

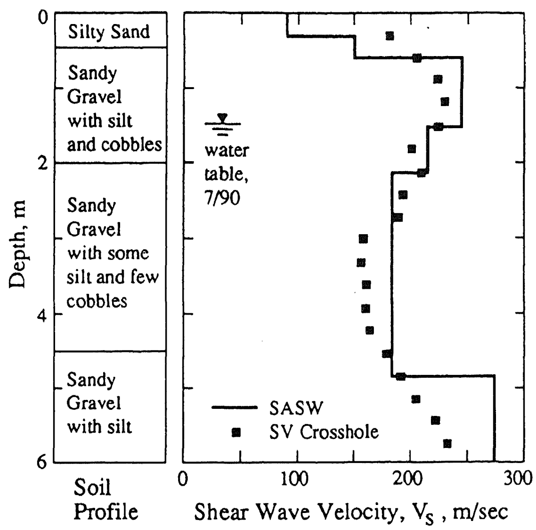

Figure 9. Shear wave velocity profiles at Larter Ranch.

pounds per square foot (psf) and σ'_m is the mean effective stress in psf. In situ V_s is directly related to G_{max} by:

$$G_{max} = (\gamma/g)\, V_s^2 \qquad (2)$$

where γ is total unit weight and g is gravitational acceleration. As a first approximation, the mean effective stress, σ'_m, was estimated by assuming simple stress conditions (level ground; major principle effective stress oriented in the vertical direction; intermediate and minor principle effective stresses oriented in the horizontal directions and equal to one half the major principle stress), and unit weights of 2.0 g/cm^3 for loose gravelly soil and 2.2 g/cm^3 for medium dense to very dense materials. Values of K_2 estimated for the depths and velocities shown in Table 1 ranged from 15 to 40 at Pence Ranch, 16 to 30 at Andersen Bar, 55 to 70 at Whiskey Springs, and 39 to 60 at Larter Ranch.

Seed et al (1985) have concluded that values of K_2 for very loose sands are in the general range of 30 and very loose gravel are 1.35 to 2.5 times higher (40 to 75). However, the K_2 values determined for the clean gravelly soils at Pence Ranch and Anderson Bar show that K_2 values of gravelly soils can be the same as very loose sands.

5 LIQUEFACTION ANALYSES

Liquefaction assessment procedures based on V_s have been proposed only for sands. Velocity measurements presented in this paper provide the first opportunity to apply these procedures to gravelly soils that liquefied.

One method for evaluating the liquefaction potential of sands from shear wave velocity has evolved from the strain approach by Dobry and his colleagues (1982). Bierschwale and Stokoe (1984) and Stokoe et al. (1989) used the strain approach in analytical studies to generated liquefaction assessment charts based on measured V_s and the maximum ground surface acceleration estimated for a stiff site at the candidate-site location. The assessment chart for 15 cycles of shaking at a level ground site with the liquefiable sand in the upper 12 m is shown in Figure 10. The lowest values of V_s from each of the four gravelly sites are also shown in Figure 10. The plotted data from the four gravelly sites lie within the liquefiable region, except Whiskey Springs which lies in the region where liquefaction is likely. Therefore, this procedure correctly predicts liquefaction at all four sites. Although the liquefaction potential appears to be lowest at Whiskey Springs and Larter Ranch, the effect of sloping ground may have increased the potential for liquefaction and shear deformation.

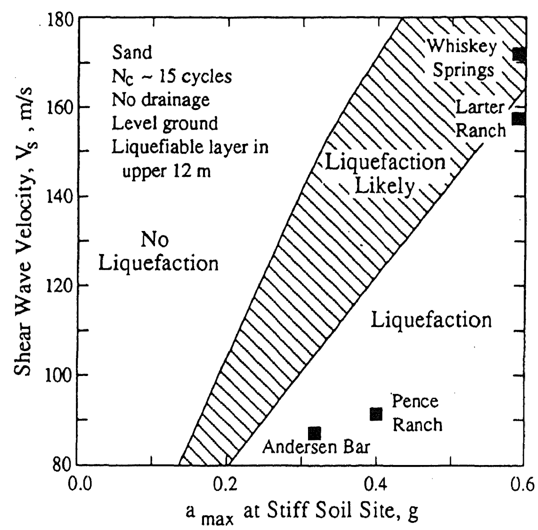

Figure 10. Liquefaction assessment chart based on shear wave velocity (Stokoe et al., 1989) with results from the four liquefaction sites.

Another method relating liquefaction potential and shear wave velocity has been proposed by Robertson (1990). An empirical correlation between in situ cyclic stress ratio and normalized shear wave velocity, V_{s1}, has been proposed for evaluating the liquefaction potential of sands. The cyclic stress ratio is calculated using the following expression:

$$\tau_{av}/\sigma'_o = 0.65\, (a_{max}/g)\, (\sigma_v/\sigma'_v)\, r_d \qquad (3)$$

in which a_{max} = maximum ground acceleration, σ_v = total overburden pressure, σ'_v = effective overburden pressure, and r_d = a stress reduction coefficient. Several predictive approaches were used to estimate a_{max} at soil sites. Analytical studies suggest (Bierschwale and Stokoe, 1984) that a_{max} on top of sites which liquefy is somewhat less than on top of stiff sites. Therefore, the lower estimates of a_{max} were used in the analysis. Overburden pressures were estimated using densities of 2.0 g/cm^3 for loose gravelly soil and 2.2 g/cm^3 for medium dense to very dense materials. Estimates of r_d ranged from 0.99 to 0.97.

The shear wave velocity is normalized with respect to an effective overburden stress as follows:

$$V_{S1} = V_S \, (Pa/\sigma'_v)^{0.25} \qquad (4)$$

were Pa = reference stress, typically 100 kPa, and σ'_v = effective overburden pressure. Cyclic stress ratios and the lowest V_{S1} values for the four gravelly sites are plotted in the proposed liquefaction assessment chart shown in Figure 11. The plotted velocity data lie within the zone of predicted liquefaction which agrees with observed field behavior.

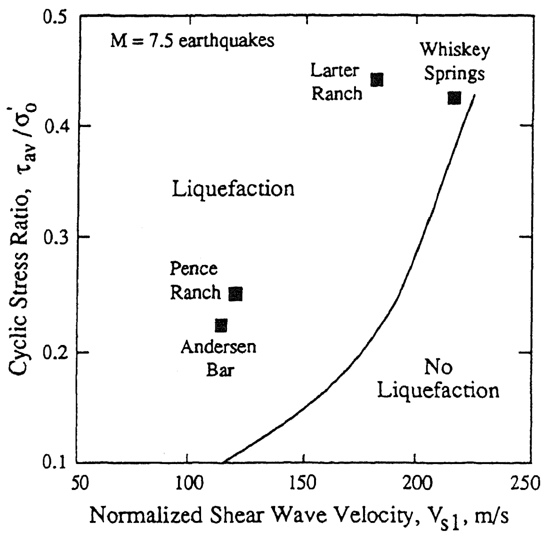

Figure 11. Proposed liquefaction assessment chart based on normalized shear wave velocity (Finn, 1991) with results from the four liquefaction sites.

CONCLUSIONS

Liquefaction occurred in clean gravelly soils at two gently sloping sites and in dirty gravelly soils at two more steeply dipping sites. Two liquefaction assessment procedures developed for sands based on shear wave velocity, V_S, were directly applied to these four gravelly soils. Both methods correctly predicted liquefaction at all four sites. This study further supports the usefulness of in situ V_S for predicting liquefaction susceptibility, especially for these hard-to-sample materials which contained more than 50 percent of the particles in the gravel to cobble range.

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REFERENCES

Andrus, R.D. 1992. In situ characterization of gravelly soils that liquefied. *Ph.D. Dissertation*. Univ. TX at Austin.

Andrus, R.D. & T.L. Youd 1987. Subsurface investigation of a liquefaction-induce lateral spread, Thousand Springs Valley, ID. *Geotech. Laboratory Misc. Paper GL-87-8*. U.S. Army Corps of Engineers.

Andrus, R.D. & T.L. Youd 1989. Penetration tests in liquefiable gravels. *Proc. 12th ICSMFE*, Rio de Janeiro, Brazil: 679-682. Rotterdam: Balkema.

Andrus, R.D., K.H. Stokoe, II, & J.M. Roesset 1991. Liquefaction of gravelly soil at Pence Ranch during the 1983 Borah Peak, ID earthquake. *Proc. 5th ICSDEE*, Karlsurhe, Germany: 251-262.

Bierschwale, J.G. & K.H. Stokoe, II 1984. Analytical evaluation of liquefaction potential of sands subjected to the 1981 Westmorland earthquake. *Geotech. Eng. Rpt. GR84-15*. Univ. of TX at Austin.

Dobry, R., R.S. Ladd, F.Y. Yokel, R.M. Chung & D. Powell 1982. Prediction of pore water pressure buildup and liquefaction of sands during earthquakes by the cyclic strain method. *N.B.S. Bldg. Science Series 138*. U.S. Dept. of Commerce.

Finn, W.D.L. 1991. Assessment of liquefaction potential and post-liquefaction behavior of earth structures: developments 1981-1991. In S. Prakash (ed.), *Proc. 2nd ICRAGEESD* II, St. Louis, MI: 1833-1850.

Harder, L.F. 1988. Use of penetration tests to determine the liquefaction potential of soils during earthquake shaking. *Ph.D. Dissertation*. Univ. CA, Berkeley.

Robertson, P.K. 1990. Seismic cone penetration testing for evaluating liquefaction potential. *Proc. Symp. on Recent Advances in Earthquake Design Using Laboratory and In Situ Tests*. ConeTec Investigations Ltd., Burnaby, B.C.

Roesset, J.M., D.W. Chang & K.H. Stokoe, II 1991. Comparison of 2-D and 3-D models of analysis of surface wave tests. *Proc. 5th ICSDEE*, Karlsruhe, Germany: 111-126.

Seed, H.B., R.T. Wong, I.M. Idriss & K. Tokimatsu 1985. Moduli and damping factors for dynamic analysis of cohesionless soils. *J. Geotech. Eng. Div.* 112, 11: 1016-1032. ASCE.

Stokoe, K.H., II, R.D. Andrus, G.J. Rix, I Sanchez-Salinero, J.C. Sheu & Y.J. Mok 1988a. Field investigation of gravelly soils which did and did not liquefy during the 1983 Borah Peak, ID, earthquake. *Geotech. Eng. Rpt. GR87-1*. Univ. of TX at Austin.

Stokoe, K.H., II, S. Nazarian, G.J. Rix, I. Sanchez-Salinero, J.C. Sheu & Y.J. Mok 1988b. In situ testing of hard-to-sample soils by surface wave method. In J.L. Von Thun (ed.), *Earthquake Eng. and Soil Dyn. II*, ASCE Geotech. Special Pub. 20: 264-278.

Stokoe, K.H., II, G.J. Rix, I. Sanchez-Salinero, R.D. Andrus & Y.J. Mok 1989a. Liquefaction of gravelly soils during the 1983 Borah Peak, ID earthquake. *Proc. 9th WCEE III*, Tokyo, Japan: 183-188.

Stokoe, K.H., II, J.M. Roesset, J.G. Bierschwale & M. Aouad 1989b. Liquefaction potential of sands from shear wave velocity. *Proc. 9th WCEE III*, Tokyo, Japan: 213-218. Nissei Kogyo Co., Ltd.

Youd, T.L., E.L. Harp, D.K. Keefer & R.C. Wilson 1985. The Borah Peak, ID earthquake of October 28, 1983-liquefaction. *Earthquake Sprectra* 2, 4: 71-89.