Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.III)

4-3-1

LIQUEFACTION OF GRAVELLY SOILS DURING THE 1983 BORAH PEAK, IDAHO EARTHQUAKE

Kenneth H. STOKOE, II[1], Glenn J. RIX[2], Ignacio SANCHEZ-SALINERO[3], Ronald D. ANDRUS[2] and Young-Jin MOK[4]

[1]Brunswick-Abernathy Regents Professor, Department of Civil Engineering, University of Texas at Austin, USA
[2]Graduate Res. Asst., Dept. of Civil Engrg., Univ. of Texas at Austin, USA
[3]Research Engineer, GEOCISA, Madrid, SPAIN
[4]Postdoctoral Research. Associate, Department of Civil Engineering, University of Texas at Austin, USA

SUMMARY

The Spectral-Analysis-of-Surface-Waves (SASW) method was used to characterize the shear wave velocities of gravelly soils which liquefied during the 1983 Borah Peak, Idaho earthquake. Because the SASW method requires no boreholes, it was well-suited for evaluating these hard-to-sample materials. The gravelly soils which liquefied were found to be very loose based upon the results of SASW and penetration tests. Simplified methods of determining the liquefaction potential of sands and silts were applied to the field data and found to predict the observed liquefaction behavior correctly.

INTRODUCTION

On October 28, 1983, a magnitude 7.3 (M_s) earthquake occurred in the Borah Peak area of south-central Idaho. The earthquake created a 37-km long rupture along the Lost River fault. One of the major causes of damage during the earthquake was the liquefaction of saturated granular deposits. Reported liquefaction effects included numerous sand boils and lateral spreading failures (Ref. 1). Many of the soils which liquefied contained significant amounts of gravel.

In July, 1985, the authors conducted in situ seismic tests using the Spectral-Analysis-of-Surface-Waves (SASW) method at the Pence Ranch, located about 10 km from the epicenter of the earthquake. Lateral spreading induced by liquefaction of gravelly soils resulted in damage to a home, roadway, and barn at this location. The primary objective of these tests was to evaluate the in situ stiffness of these hard-to-sample soils in terms of shear wave velocity. The SASW method was selected as the primary measurement technique in the field because no boreholes are required, a significant advantage in these gravelly soils. In addition, disturbed samples were collected with a 127-mm ID auger tube sampler.

A brief description of the SASW method is presented herein. Basic soil properties and the results of the disturbed sampling, cone penetration (CPT), standard penetration (SPT) and SASW tests are discussed. (It was interesting to note that SPT and CPT tests were originally not planned in these gravelly materials. However, once the looseness of the gravelly materials was established, SPT and CPT tests were attempted.) Finally, shear wave velocity, SPT and CPT results are used to evaluate the liquefaction potential of the gravelly materials.

SPECTRAL-ANALYSIS-OF-SURFACE-WAVES METHOD

The dispersive nature of surface wave propagation in a layered, elastic half space forms the basis of the SASW method. Field measurement and calculation of surface wave dispersion followed by inversion of the data permit detailed profiles of shear wave velocity to be determined. One of the principal advantages of the SASW method is that both the source and receivers are located on the ground surface, thereby eliminating the need for boreholes. This makes the SASW method well-suited for determining stiffness profiles of hard-to-sample soils such as these gravelly materials (Ref. 2).

Test Configuration and Equipment The general configuration of the source, receivers, and recording equipment used in SASW testing is shown in Fig. 1. Typically, distances between receivers of 1, 2, 4, 8, 16, and 32 m are used if the soil profile is to be evaluated to a depth of about 20 m. The most common types of sources used are either simple hammers or dropped weights weighing from 200 to 1500 N.

A dual-channel FFT analyzer is used to record and process data. The ability to process data rapidly in the field is an essential part of the SASW method, allowing the operators to immediately assess the quality of the data being collected and, if necessary, modify the arrangement of source and receivers or other test parameters accordingly.

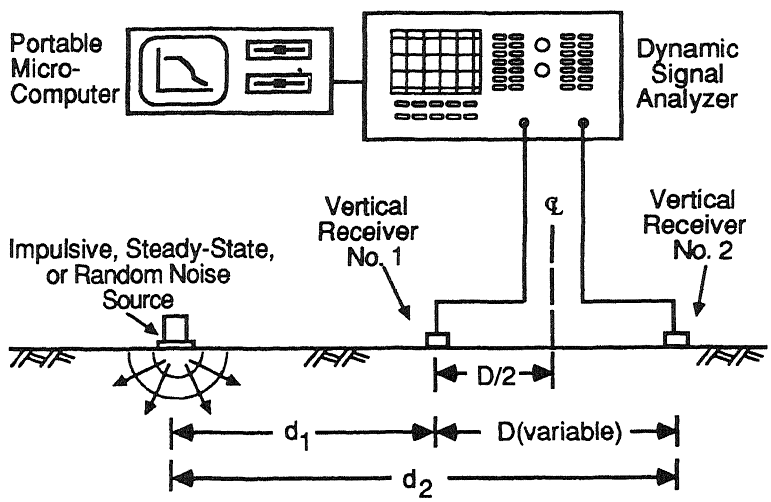

Fig. 1 General Configuration of Source and Receivers Used in SASW Testing

The time delay between receivers as a function of frequency is calculated using the phase of the cross power spectrum between the two receivers. The surface wave phase velocity is determined using the time delay and distance between receivers. The result is a dispersion curve for one receiver spacing. Individual dispersion curves resulting from all of the different receiver spacings used at one test location are assembled together to form the composite dispersion curve for the site as discussed in Reference 2.

Inversion Inversion is the process of obtaining the shear wave velocity profile from the experimental dispersion curve. Inversion is performed using an iterative procedure based on forward modelling. In this procedure, a theoretical dispersion curve calculated using a Haskell-Thomson algorithm (Ref. 2) is matched to the experimental curve measured in the field. The application of inverse theory to engineering surface wave testing has increased the accuracy of resulting wave velocity profiles and has significantly expanded the variety of sites at which the SASW method can be successfully used.

SITE DESCRIPTION AND TEST RESULTS

A map of the key portion of the Pence Ranch is shown in Fig. 2. The locations of gravelly sand boils and fissures which occurred are indicated in the figure. Disturbed sampling and SASW, SPT, and CPT tests were conducted at the test sites shown near the hay yard. Based on the penetration logs and sample borings, the generalized cross section shown in Fig. 3 was constructed. The sediment profile consists of a sand and silt topsoil (unit A) underlain by loose to very dense sand, gravel and cobbles (units B, C, D, and E). The two units of most concern are C and D. Shear wave velocities determined using the SASW procedure ranged from 90 to 105 m/s in unit C. SPT and CPT resistances in unit C ranged from 5 to 10 blows/ft and 4 to 180 kgf/cm^2, respectively. Unit D is characterized by shear wave velocities of 115 to 215 m/s, N-values of 18 to 23, and tip resistances of 100 to 290 kgf/cm^2. Sediments within these units range from clean gravelly sand (SP-GP) to sandy gravel (GP). Grain-size distribution curves of samples collected from unit C using a 127-mm ID auger tube sampler are shown in Fig. 4. The gravels are rounded with low sphericity. Samples collected from unit D contain more gravel size material. The small percentage of sand and silts suggest that gravel particles are supported by each other creating a "clast-supported structure."

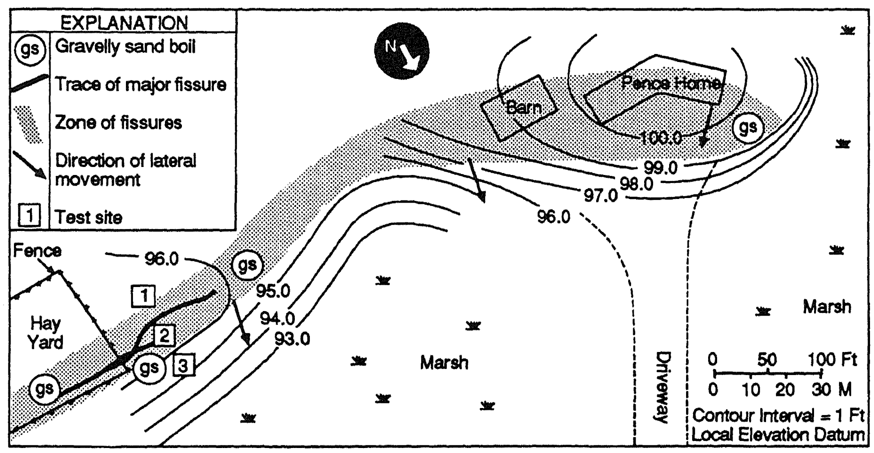

Fig. 2 Map of Pence Ranch Showing Liquefaction Effects and Location of Three Test Sites

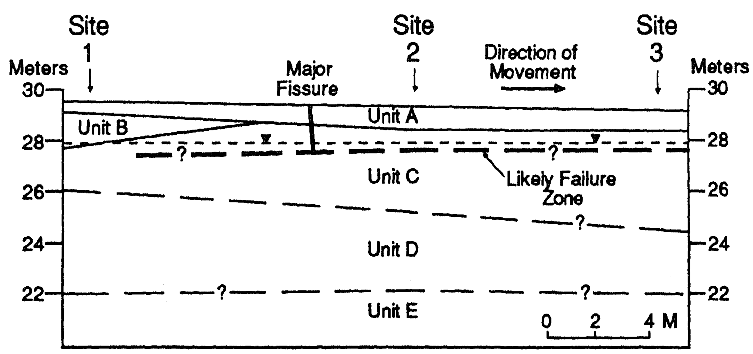

Fig. 3 Generalized Cross-Section of Lateral Spread Near Hay Yard

It is possible to qualitatively estimate the in situ density of the sandy gravel material using the relationship suggested by Seed et al (Ref. 3):

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

where G_{max} is the low-amplitude shear modulus, K_2 is an empirical constant which reflects the density of the material, and $\bar{\sigma}_m$ is the mean effective stress. Since G_{max} is directly related to shear wave velocity, shear wave velocity can be used to qualitatively estimate the in situ density. Values of K_2 of gravels which are less than 40 have been suggested to be representative of loose materials (Ref. 3). In fact, K_2 values in unit C of less than 22 were calculated based upon the shear wave velocities determined from the SASW test (Ref. 2). These very low values of K_2 indicate that unit C is composed of very loose material. It is interesting to note that the authors have also determined similar low values of K_2 for many of the loose sands in the Imperial Valley of California which liquefied in the 1979 earthquake.

Based on the low shear wave velocities and low penetration resistances, liquefaction and shear deformation are assumed to have occurred in unit C. Further evidence supporting this assumption is the geometric relationship of the sediment layers to the lateral spread. Cone penetration soundings clearly suggest a thinning of unit C from about 4 m at Site 3 to 1.5 m at Site 1 (Fig. 3). This change in facies provides an explanation for the location of the southern extent of the fissures. High porewater pressures may have also developed in layer D. However, since unit D is present in all the drill holes and only part of the site experienced lateral spreading, the pore pressures within layer D do not appear to have controlled sliding.

LIQUEFACTION POTENTIAL

Generally accepted guidelines have not been developed for evaluating the liquefaction potential of soils which contain significant amounts of gravel. Therefore, simplified procedures developed for sands and silts were used to assess the liquefaction potential of the gravelly sediments.

Stress-Based Approach The most widely used approach for evaluating the liquefaction potential of sands and silts is a procedure developed by Seed and his colleagues based upon SPT and CPT test results (Refs. 4 and 5). To use this method, the in situ cyclic stress ratio is calculated and plotted versus the modified penetration resistance. The cyclic stress ratio is calculated using the following expression:

$$\tau_{av}/\bar{\sigma}_o = 0.65\,a_{max}/g\,(\sigma_o/\bar{\sigma}_o)\,r_d \qquad (2)$$

in which a_{max} is the maximum ground acceleration at the ground surface, σ_o is the total overburden stress at the depth under consideration, $\bar{\sigma}_o$ is the effective overburden stress at the same depth, and r_d is a stress reduction coefficient. The maximum ground acceleration on a stiff site was estimated to be 0.35g based on the surface wave magnitude (M_s = 7.3) of the earthquake. The maximum ground acceleration on top of the sites which liquefied is estimated to be reduced slightly to about 0.28g (Ref. 6). Modified penetration resistances were determined using procedures described in Refs. 4 and 5.

Cyclic stress ratios are plotted versus average modified SPT and CPT penetration resistances in Figs. 5 and 6, respectively. Also shown in these figures is the liquefaction potential curve for materials with less than 5 percent fines. By applying the criteria of Seed and his colleagues directly to units C and D at Pence Ranch, unit C is predicted to liquefy and has significant shear deformation potential. This assessment is in agreement with field observations. Unit D is, however, also shown to be rather close to possible liquefaction in Fig. 5. It is difficult to clearly assess the liquefaction potential of unit D using q_c and Fig. 6 because the average q_c/N_{60} value for unit D is greater than 7 which is well beyond the limit of 5.3 presented in this plot. A set of potential curves has been presented for ratios less than 5.3 (Ref. 5). This suggests that additional curves could be drawn for ratios of q_c/N_{60} greater than 5.3 and closer to unit D.

Strain-Based Approach Another method of evaluating the liquefaction potential of sediments is based upon measured shear wave velocities and maximum ground acceleration (Refs. 6 and 7). The liquefaction potential is estimated by plotting the shear wave velocity versus the maximum ground acceleration estimated for a stiff site at the candidate-site location. (This method has evolved from the strain approach to liquefaction proposed by Dobry and his colleagues [Ref. 8].) Shear wave velocities determined in units C and D are shown in Fig. 7. Unit C lies within the zone where liquefaction is predicted to occur which agrees with the field performance. Unit D has a lower potential than unit C. However, unit D is shown to lie in the region of likely liquefaction. It is difficult to properly assess the liquefaction potential of unit D because Fig. 7 was generated assuming no drainage and drainage may have occurred in unit D.

CONCLUSIONS

This study demonstrates that gravelly soils which were previously believed to be too well drained to liquefy may, in fact, liquefy if the soils are very loose, sufficiently large ground accelerations occur and a relatively impermeable cap overlies the gravelly materials. Additional evidence has also been presented which suggests that simplified methods of evaluating liquefaction susceptibility of sands can be applied to very loose gravelly soils. Finally, the usefulness of the SASW method for making in situ measurements of shear wave velocity in hard-to-sample soils is shown.

ACKNOWLEDGEMENTS

The authors wish to extend their gratitude to the U.S. Geological Survey for supporting this work. Dr. T. Leslie Youd of Brigham Young University graciously provided information about the performance of the sites during the 1983 earthquake.

REFERENCES

1. Youd, T.L., Harp, E.L., Keefer, D.K., and Wilson, R.C., (1985), The Borah Peak, Idaho Earthquake of October 28, 1983 - Liquefaction," Earthquake Spectra, Vol.2, No. 4, pp.71-89.
2. Stokoe, K.H., II, Nazarian, S., Rix, G.J., Sánchez-Salinero, S., Sheu, J.C., and Mok, Y.J., (1988), "In Situ Seismic Testing of Hard-To-Sample Soils by Surface Wave Method," in Earthquake Engineering and Soil Dynamics II, Geotechnical Special Publication No. 20, ASCE, pp. 264-278.
3. Seed, H.B., Wong, R.T., Idriss, I.M., and Tokimatsu, K., (1986), "Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils," Journal of the Geotechnical Engineering Division, ASCE, Vol. 112, No. 11, 17 pp 1016-1032.
4. Seed, H.B., Tokimatsu, K., Harder, L.F., and Chung, R.M., (1985), "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," Journal of the Geotechnical Engineering Division, ASCE, Vol. 111, No. GT12, pp. 1425-1445.
5. Seed, H.B., and de Alba, P., (1986), "Use of SPT and CPT Tests for Evaluating the Liquefaction Resistance of Sands," Use of In Situ Tests in Geotechnical Engineering, Geotechnical Special Publication No. 6, ASCE, pp. 281-302.
6. Bierschwale, J.G., and Stokoe, K.H., II, (1984), "Analytical Evaluation of Liquefaction Potential of Sands Subjected to the 1981 Westmorland Earthquake," Geotechnical Engrg. Rpt. No. GR84-15, The University of Texas at Austin, 231 pp.
7. Roesset, J.M., Bierschwale, J.G., Aouad, M. and Stokoe, K.H., II, (1988), "Liquefaction Potential of Sands from Shear Wave Velocity," submitted to 9WCEE, Tokyo, Japan.
8. Dobry, R., Ladd, R.S., Yokel, F.Y., Chung, R.M., and Powell, D., (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.

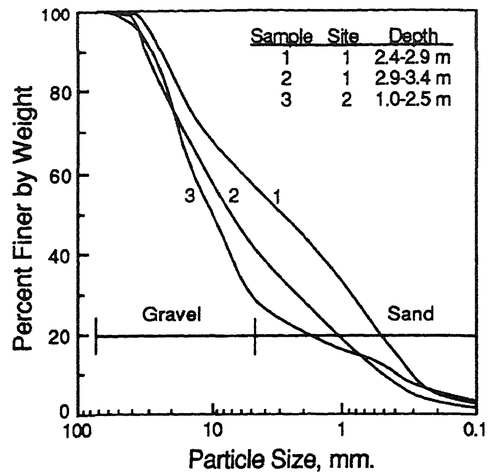

Fig. 4 Grain-Size Distribution Curves for Samples Taken from Unit C

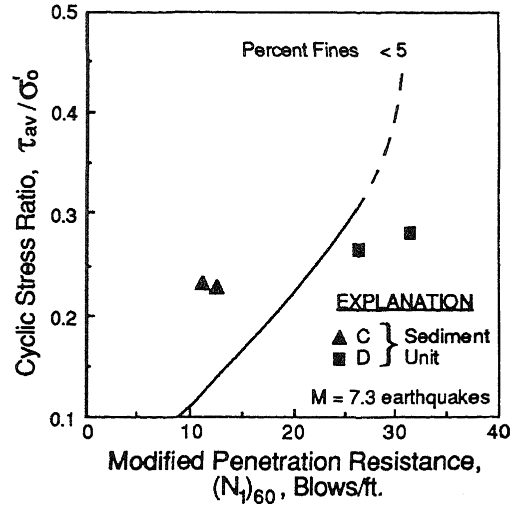

Fig. 5 Liquefaction Potential Chart Based on Modified N-Values (Ref. 7) with Results from Test Sites

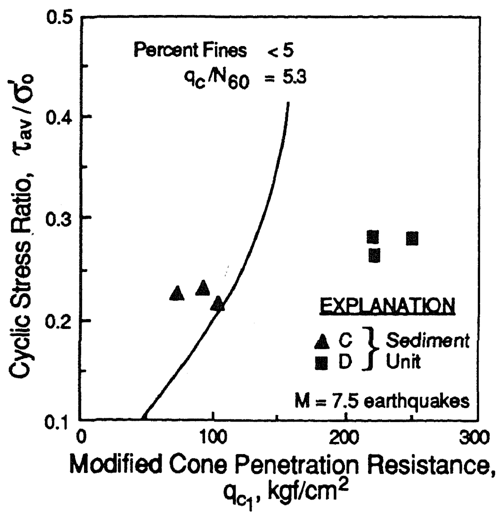

Fig. 6 Liquefaction Potential Chart Based on Modified Tip Resistance (Ref. 8) with Results from Test Sites

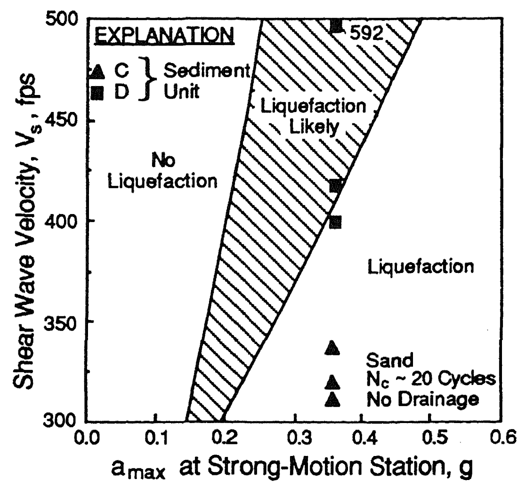

Fig. 7 Liquefaction Potential Chart Based on Shear Wave Velocity with Results from Test Sites