Development of a Field Method for Evaluating Nonlinear Properties and Liquefaction Resistance of Near-Surface Soils

Grant Number: OOHQGR0015

Kenneth H. Stokoe, II
Ellen M. Rathje

University of Texas at Austin
Office of Sponsored Projects
P.O. Box 7726
Austin, Texas 78712
Telephone: (512) 471-4929
Fax: (512)471-6548
k.stokoe@mail.utexas.edu

Program Element: PT

Keywords: Liquefaction, Site Effects, Strong Ground Motion

Investigations Undertaken

The goals of this project are to develop field methods that can be used to: 1. evaluate the nonlinear response of soils and 2. evaluate the liquefaction resistance of soils. At this time, the field methods under development are aimed at testing near-surface soils; that is, soils within 0.5 to 3 m of the ground surface.

During this second year of the project, development of a generalized test method to measure nonlinear soil properties has continued. The method involves applying static and dynamic loads at the surface of the soil deposit being tested, and measuring the dynamic response of the soil mass beneath the loaded area using embedded instrumentation. A vibroseis truck is used to apply static and dynamic loads to a large circular footing at the ground surface. A vibroseis truck is an electro-hydraulic shaker used in oil exploration as a seismic source for reflection studies. The instrumentation includes a load cell to measure the loading applied to the footing and embedded velocity transducers (geophones) under the loaded area to measure the response of the soil mass. The result is a load-controlled dynamic field test that induces soil nonlinearity within a predetermined instrumented zone.

The second-year testing presented herein focuses on vertically and horizontally loading the soil, evaluating the magnitude of induced strains, and assessing: 1. the variation of constrained compression wave (P-wave) velocity with vertical stress and vertical strain, and 2. the variation of shear wave (S-wave) velocity with shearing stress and shearing strain. Evaluating in situ material damping was beyond the scope of these tests, but it is certainly an important parameter to be studied in future tests.

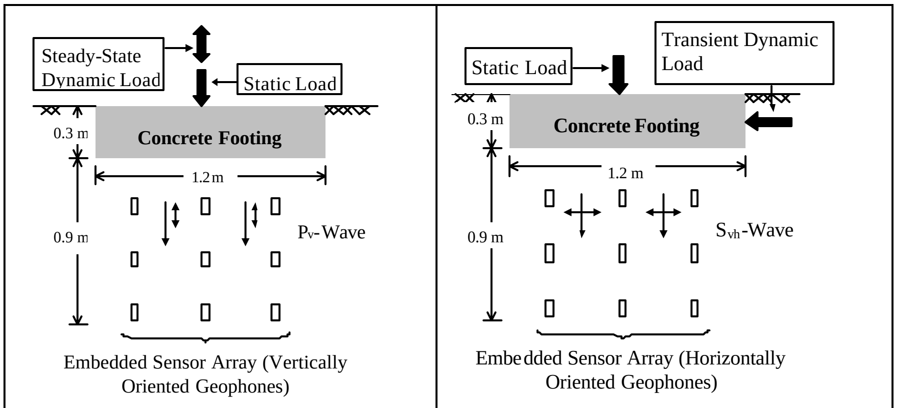
Work also commenced this year on developing a field method to evaluate the liquefaction resistance of soils. In this initial work, the vibroseis truck was located about 4m away from the soil deposit to be liquefied . Embedded instrumentation in the saturated soil was used to monitor motions and pore water pressures. The set-up resulted in the application of controlled cyclic loads to the liquefiable soils by using Rayleigh waves that were generated with the vibroseis truck.

Test Setup in the Field

All field testing was performed at a local soil quarry owned by Capitol Aggregates, in Austin, Texas. For the work involving nonlinear soil measurements, a circular reinforced concrete footing was constructed at the site to transfer load from the hydraulic ram of the vibroseis truck to the surface of the soil mass. The footing was 1.2 m in diameter, 0.3-m thick, and was embedded 0.3 m into the ground. Before the concrete footing was constructed, an array of 30 geophones was embedded at various locations and depths below the ground surface. These geophones were placed as either one-dimensional (1-D) vertical sensors or three-dimensional (3-D) sensors. The basic configuration of the embedded geophone array and the corresponding dynamic loading modes are shown in Figure 1.

To apply vertical, steady-state dynamic loads, the vibroseis truck was placed over the concrete footing and the loading ram from the truck was lowered onto a steel frame that was used to distribute the load across the footing. A load cell was placed between the ram and steel frame to measure the load levels. The vibroseis truck in this position is shown in Figure 2. To apply horizontal loads to the footing, a pendulum mechanism was constructed and a transient dynamic load was applied to the side of the footing as illustrated in Figure 1b.

The soil at the site is a poorly graded sand (SP) with 5% finer than the #200 sieve. The soil is tan in color and has occasional rounded, gravel-sized particles that amount to less than 0.5% of the total soil volume. The groundwater table is at a depth of 1.5 m. The soil is heavily overconsolidated due to the removal of at least 8.8 m of overburden. In addition, many different layers of soil exist at the site, most of which contain varying degrees of cementation. The average water content of the soil around the embedded geophones was 2.7%. An intact block sample of the soil gave an in situ density of 16.9 kN/m^3, a degree of saturation, S_R, of 12% and a void ratio, e, of 0.60. Resonant column tests were also performed on relatively undisturbed samples trimmed from the large block sample.



a. Constrained Compression Waves

b. Shear Waves

Figure 1 Basic approach to performing large-strain dynamic tests in situ.

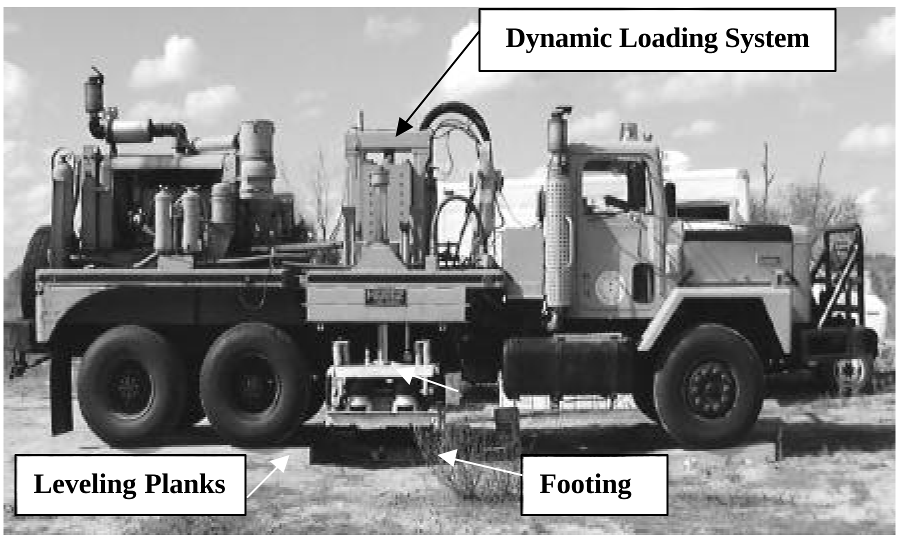


Figure 2 Vibroseis truck in position over footing for vertical loading.

Small-strain crosshole and downhole seismic tests were performed after each static load was applied to the footing. These tests were used to evaluate the small-strain (linear) stiffness of the soil beneath the footing. Measurements of horizontally propagating compression waves, P_h, and horizontally propagating and vertically polarized shear waves, S_{hv}, were performed. Measurements of vertically propagating compression waves, P_V, and vertically propagating and horizontally polarized shear (S_{vh}) waves were also performed. In terms of the work presented herein, the discussion is limited to P_V waves. The work dealing with S_{hv} waves is presented in Axtell et al. [1].

Nonlinear dynamic testing was conducted over a 16-day period, during which increasing static and dynamic loads were applied to the soil mass. For each vertical, steady-state dynamic loading stage, a static load was first applied to the footing, followed by a sinusoidal dynamic load centered about the static load. Static loads applied to the footing ranged from 0 kN to 80.3 kN and dynamic loads varied from 2.2 kN to 44.6 kN. All steady-state testing was performed at a frequency of 40 Hz.

Similar types of nonlinear tests, but performed in shear, were also conducted. These tests are described in Axtell et al. [3]

Results: in situ small-strain measurements

It is a well established fact that increasing the confining pressure causes an increase in the stiffness of the soil as long as large shearing strains are not developed. The state of stress in the soil beneath the footing increased with each increase in static vertical load applied to the footing. The effect of stress state on soil stiffness was evaluated in situ by measuring small-strain wave velocities at each static load. Figure 3 presents typical P_V-wave velocity (V_{Pv}) measurements. These measurements were performed over the depth interval from the base of the footing to the first embedded receiver at a depth of 17.8 cm below the base. (Hence, they are represented by an average depth of 8.9 cm beneath the footing base.) A uniform pressure distribution was assumed at the footing base and a Boussinesq stress distribution was used to obtain profiles of the increase in vertical total stress in the soil at each static load. Total stresses were used in this analysis as opposed to effective stresses because the porewater pressures in the soil, certainly negative, but very small were unknown.

A substantial decrease in V_{Pv} after high-amplitude, steady-state testing at a static load level of 44.6 kN is shown in Figure 3 by the open square. The high-amplitude loading at this static load level resulted in the breakage of cementation bonds in the soil. Therefore, a different material was essentially being tested after dynamic testing at a static load of 44.6 kN. This reduction in wave velocity was not unique to V_{Pv}. The reduction was easily identifiable in all wave velocities, indicating a change had occurred in the dynamic soil properties beneath the footing.

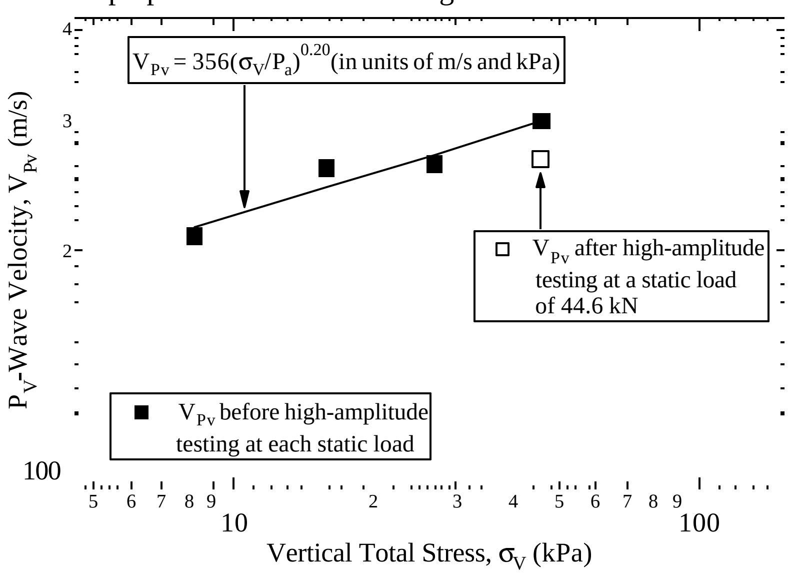


Figure 3 In situ variation of P_v-wave velocity with change in vertical total stress at an average depth of 8.9 cm beneath the footing.

Results: in-situ nonlinear measurements

The primary goal in this study was to generate nonlinear behavior in the soil by inducing large axial strains during dynamic vertical loading. During these tests, the largest axial strains induced in the soil were 0.035%. However, larger strains most likely could have been generated with larger static and dynamic loads. Unfortunately, due to our inexperience in this new experimental endeavor and the time required to reduce the data and calculate axial strains, the exact strain levels were not known in the field. Consequently, loads were not adjusted in the field to generate strains larger than 0.035%. This shortcoming will be overcome in future projects.

Both constrained moduli and axial strains were calculated from the steady-state dynamic loading using the vertical geophones embedded beneath the center of the footing. The constrained moduli at different axial strain levels and different geophone depths were calculated from the measured body wave velocities. The axial strain level in the soil was evaluated at each geophone location and was calculated assuming plane wave propagation by using the peak particle velocity and the wave propagation velocity.

The nonlinear constrained moduli collected at each static load level during steady-state dynamic loading were normalized by the maximum constrained modulus ($M_{v,max}$) evaluated with the small-strain downhole tests. The constrained moduli were normalized by dividing each constrained modulus (M_v) by the maximum constrained modulus ($M_{v,max}$) for that static load level. The variations of normalized moduli with peak axial strain ($M_v/M_{v,max}$ – log ε_P) evaluated at an average depth of 33 cm beneath the footing are shown in Figure 4. Normalized moduli at three different static load levels are shown. Also, the upper and lower bounds and average generic shear modulus reduction curves for sand from Seed et al. [2] have been added to the figure for comparison purposes. Although the curves presented by Seed et al [2] were developed for normalized shear modulus (G/G_{max}), as opposed to normalized constrained modulus (M/M_{max}), the generalized relationship with strain is still valid because the sand tested in this study was only 12% saturated and M and G can be related by Poisson's ratio, ν, even though ν may vary nonlinearly with strains.

The first obvious result in Figure 4 is that nonlinear behavior was generated in the sand. The largest axial strain was only about 0.015% at this test depth. However, the value of M_v was reduced to about 70% of $M_{v,max}$ at this strain level. The second point is that the in-situ $M_v/M_{v,max}$ – log ε_P relationship shows excellent agreement with the mean sand curve presented by Seed et al. [2]. This close agreement seems to indicate that, at least in this case, nonlinear measurements in the laboratory represent the nonlinear field behavior very well.

Two points of concern that require more investigation, both experimentally and analytically, are the plane wave approximation and the assumption of constrained compression (no lateral strain).

Additional information, including nonlinear measurements in shear, can be found in Axtell et al. [1].

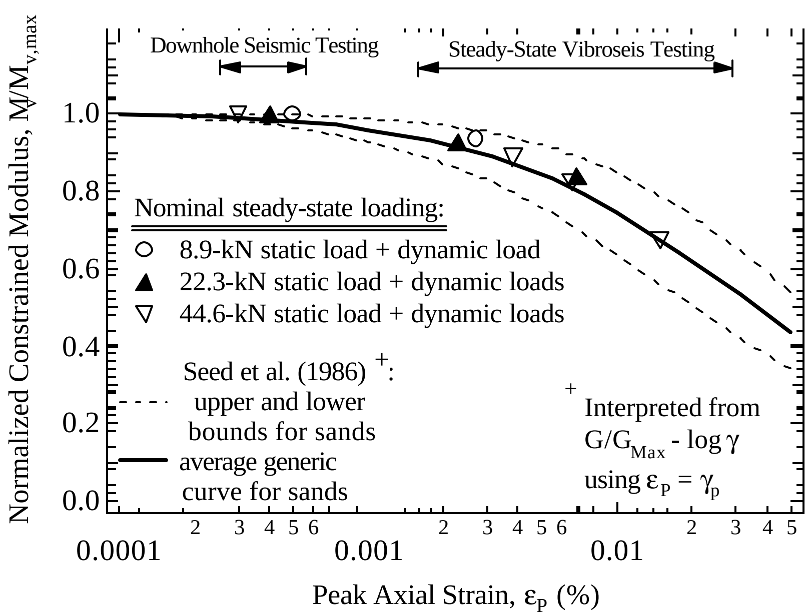


Figure 4 Variation in normalized vertical constrained modulus with vertical axial strain from in situ measurement of sand at an average depth of 33 cm beneath the footing base.

Conclusions

The testing procedures and methods of data analysis developed in this work allow in situ measurements of nonlinear soil properties. Large-strain (nonlinear) compression-wave tests were performed using a vibroseis truck and large-strain shear-wave tests were performed using a pendulum hammer. Results from the linear and nonlinear tests allowed in situ constrained moduli and shear moduli reduction curves to be developed. With further improvements, it should be possible to measure more material properties, such as material damping in shear and compression, and draw conclusions about dynamic soil behavior and in situ states of stress for coarse-grained soils. Upon refinement of the testing method, generation of pore water pressures for the purpose of in situ liquefaction evaluation will be possible, as preliminary testing has shown. Data from test involving the generation of pore water pressure will be extremely useful in understanding liquefaction and refining liquefaction evaluation techniques.

References

[1] Axtell, P.J., Stokoe, II, K.H., Rathje, E.M., and Valle, C., (2001), "In-Situ Measurements of Linear and Nonlinear Properties of a Near-Surface, Poorly Graded Sand,". United State Geological Survey Final Report, (in progress).

[2] 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 Soil Mechanics and Foundations Division*, ASCE, Vol. 112, No. 11, pp. 1016-1032.

Non-Technical Summary

Evaluation of the earthquake response of soil sites requires knowledge of the stiffness and damping characteristics of the soil. At this time, there is total dependency on laboratory testing with small specimens to evaluate these characteristics. One goal of this project is to develop a field method to evaluate the stiffness and damping characteristics. Field testing has focused on applying vertical and horizontal dynamic loads to a rigid footing on the ground surface. The soil behavior beneath the footing during these loads was measured with embedded instrumentation. This work shows that nonlinear soil behavior can be successfully measured.

Reports Published

Phillips, R.D. and Rathje, E.M., (2000), "Initial Design and Implementation of an In Situ Test for Measurement of Nonlinear Soil Properties," Geotechnical Engineering Report, GT00-1, Civil Engineering Department, University of Texas at Austin, Austin, Texas.

Axtell, P.J., Stokoe, II, K.H., Rathje, E.M., and Valle, C., (2001), "In-Situ Measurements of Linear and Nonlinear Properties of a Near-Surface, Poorly Graded Sand," United State Geological Survey Final Report, (in progress).

Papers Published

Rathje, E.M., Phillips, R.D., Chang, W.-J. and Stokoe, K.H., II, (2001), "Evaluation Nonlinear Soil Response in Situ," Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, March 26-31.

Stokoe, K.H., II, Axtell, PJ., and Rathje, E.M., (2001), "Development of an In Situ Method to Measure Nonlinear Soil Behavior," Third International Conference on Earthquake Resistant Engineering Structures, ERES2001, September 4-6, 2001, Malaga, Spain..

Availability of Processed Data

All processed data are available in Axtell et al. (2001) in graphical and tabular forms. The contact person is Professor Kenneth H. Stokoe, II. He can be reached at 512-232-3683 and by e-mail at k.stokoe@mail.utexas.edu.