Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering

doi:10.3233/978-1-61499-656-9-751

Development of an in situ method to measure the nonlinear shear modulus of soil

Le développement d'un dans la méthode de situ pour mesurer le modulus de cisailles nonlinéaire de sol

Kenneth H. Stokoe, II & Ellen M. Rathje
University of Texas at Austin

Paul J. Axtell
US Army Corps of Engineers

ABSTRACT

One of the key variables in evaluating the response of geotechnical materials during strong earthquake shaking is their nonlinear behavior. This behavior is often expressed in terms of the nonlinear reduction in shear modulus with increasing shearing strain and the corresponding increase in material damping ratio. Currently, there is total dependency on laboratory testing to measure these properties. A generalized test method is under development at the University of Texas at Austin to measure nonlinear soil properties in situ. The method involves applying static and dynamic loads at the surface of a soil deposit using a mobile, electro-hydraulic shaker and measuring the dynamic response of the soil mass beneath the loaded area using embedded instrumentation. The resulting field test is a load-controlled dynamic test that induces nonlinearity in the soil. Initial prototype studies have focused on measuring linear and nonlinear shear moduli over a range in stress states. These studies are discussed, and measurments in a sand deposit are presented. The linearity and nonlinearity measured in situ compare well with laboratory results and empirical trends.

RÉSUMÉ

Une des variables clées dans évaluer la réponse de matériels de geotechnical pendant la secousse de tremblement de terre forte est leur comportement non linéaire. Ce comportement est souvent exprimé dans les termes de la réduction non linéaire dans modulus de cisailles avec augmenter tondant la tension et l'augmentation qui correspond dans le matériel étouffe la proportion. Actuellement, il y a la dépendence totale sur l'essai de laboratoire pour mesurer ces propriétés. Une méthode généralisée de test est sous le développement à l'Université de Texas à Austin pour mesurer des propriétés de sol non linéaire dans situ. La méthode implique la générution des chargements statiques et dynamiques sur la surface d'un dépôt de sol utilisant un mobile, un secoueur et mesurer electro-hydrauliques la réponse dynamique de la masse de sol en dessous du secteur chargé utilisant l'instrumentation enfoncée. Le test résultant de champ est un test dynamique de chargement-contrôlé qui persuade la non-linéarité dans le sol. Les études initiales de prototype ont convergé sur mesurant des moduli de cisailles linéaire et non linéaire par-dessus une gamme dans les états de tension. Ces études sont discutées, et measurments dans un dépôt de sable est présenté. La linéarité et non-linéarité mesurée dans situ compare bien avec les résultats de laboratoire et les tendances empiriques.

1 INTRODUCTION

Evaluation of nonlinear soil properties (i.e., nonlinear reduction in shear modulus with shearing strain and the corresponding nonlinear increase in material damping ratio) is an important part of site characterization in geotechnical earthquake engineering. Currently, there is total dependency on laboratory tests to measure nonlinear soil properties. Typically, laboratory tests are performed with either high-quality intact samples or reconstituted specimens. The nonlinear reduction in shear modulus (G) and nonlinear increase in material damping in shear (D_S) with shearing strain (γ) are evaluated over a wide strain range using one or more types of laboratory equipment. The resulting nonlinear curves are used in site response analyses that model shear wave propagation.

In situ measurements of soil properties offer many advantages over laboratory measurements and, hence, are typically the approach of choice when possible. For instance, V_S is routinely measured in situ using small-strain seismic techniques. However, when the nonlinear variation of G and D_S with γ are required, laboratory testing is conducted and/or empirical relationships are used. Concern always exists about the accuracy with which the in situ properties are represented by laboratory-determined values. Differences are known to occur because of sample disturbance, improper confinement, and nonrepresentative boundary conditions. This concern is heightened if empirical relationships are used due to the lack of site-specific calibration and the potential exclusion of important (and sometimes unknown) parameters. Development of a large-strain in situ test will overcome many of the limitations associated with laboratory testing and empirical relationships. Additionally, in situ measurements of nonlinear soil properties will provide opportunities to: (1) evaluate the accuracy of representing in situ nonlinear properties with laboratory measurements, (2) modify laboratory tests and procedures to improve agreement between the laboratory and field, and (3) improve empirical relationships and develop new relationships where data are very limited (i.e. gravelly soils, cemented soils, etc.).

To measure nonlinear soil properties in situ, a new test method is being developed at the University of Texas at Austin (Rathje et al. 2001 and Axtell et al., 2002). The method involves applying static and dynamic loads near the surface of a soil deposit, and measuring the dynamic response of the soil mass beneath the loaded area using embedded instrumentation. Small-strain seismic measurements using the traditional crosshole procedure are employed to further characterize the soil. In these initial prototype studies, varying static loads were vertically applied to a 1.2-m diameter footing. A truck mounted electro-hydraulic shaker used in geophysical exploration (called a Vibroseis) was employed as the reaction mass for the vertical static loads. Transient horizontal loading of the footing was achieved at each static load with a large pendulum hammer that horizontally impacted the footing. Horizontal loads over a wide range in amplitudes were generated with the pendulum hammer. This paper focuses on the linear and nonlinear measurements of V_s and G with this prototype system. Improvements in the loading portion of the system using a new, tri-axial vibroseis are also discussed. Evaluation of in situ material damping will be considered in future work.

2 TEST SETUP IN THE FIELD

Field testing was performed at a local soil quarry owned by Capitol Aggregates in Austin, Texas. A circular, reinforced concrete footing was constructed at the site. The footing was cast-in-place so that it would develop contact with the soil over the complete base of the footing and hence model an end platen in a laboratory test. The footing was then used to transfer the applied static and dynamic loads to the soil beneath the footing. 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 (Axtell (2002) presents additional details on the geophones and placement procedures. The basic configuration of the embedded geophone array and the corresponding loads applied to the footing are illustrated in Figure 1. The horizontal geophones were used to measure vertically propagating and horizontally polarized shear waves, S_{vh}, that were generated by horizontally impacting the footing. The vertical geophones were used in small-strain crosshole seismic testing using horizontally propagating and vertically polarized shear waves, S_{hv}, as discussed below. Two cased boreholes, approximately 0.3 m from the edge of the footing, (not shown in Figure 1) were used as source boreholes in the crosshole tests.

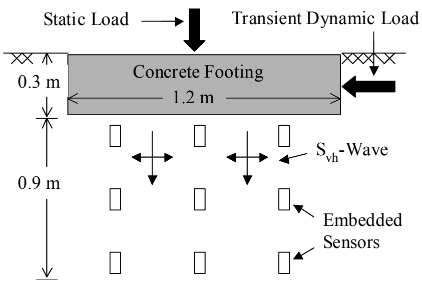


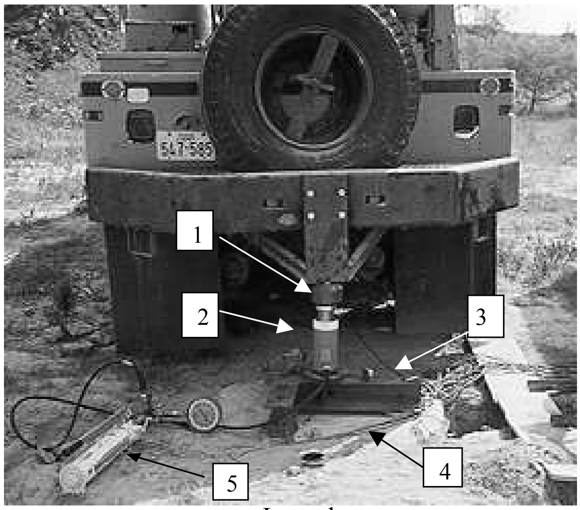
Figure 1. Basic approach for performing nonlinear shear modulus measurements

To apply vertical static loads, the rear bumper of the Vibroseis was positioned over the concrete footing, and a jacking mechanism was inserted between the bumper and a steel loading frame on top of the concrete footing. The steel frame provided a single point of contact where the vertical load could be measured using a load cell. The mass of the Vibroseis (around 22,700 kg) was simply used as a reaction force for the static load. Use of the truck in this manner was not special to the Vibroseis, as any mobile, heavy mass would have provided an adequate load. However, the truck was on site and readily available for these prototype tests and therefore was used. The Vibroseis in this position is shown in Figure 2.

Dynamic horizontal loads were applied to the footing using a pendulum hammer. The Vibroseis was not used because it can only apply loads in the vertical direction. The pendulum was used to strike the footing as illustrated in Figure 3. A steel mass that weighed 60 kg was placed on a 1.5-m long arm. Dynamic loads ranging from small to large were generated simply by rotating the pendulum hammer to different heights and releasing.

Small-strain crosshole seismic tests were performed after each static load was applied to the footing. Two boreholes, adjacent to the footing, were used as crosshole source boreholes. Measurements of horizontally propagating compression waves (P_h), and horizontally propagating and vertically polarized shear waves, (S_{hv}), were performed between the embedded instrumentation.

Nonlinear dynamic testing was conducted over a 16-day period, during which increasing static and dynamic loads were applied to the soil mass. A schematic illustration of the staged



Legend:
1. Load Cell
2. Hydraulic Cylinder
3. 3-Point Steel Loading Frame
4. Concrete Footing
5. Manual Hydraulic Pump

Figure 2. Static loading of concrete footing for small- and large-strain tests

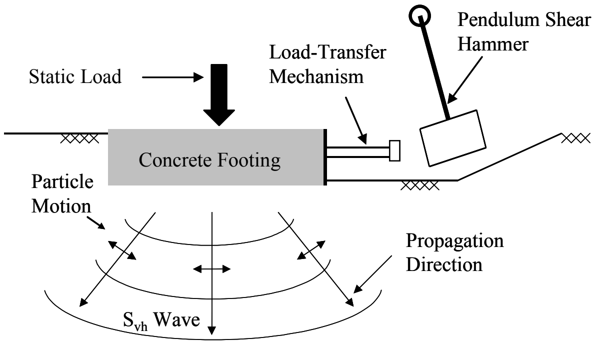


Figure 3. Generation of linear and nonlinear S_{vh} waves using a pendulum hammer

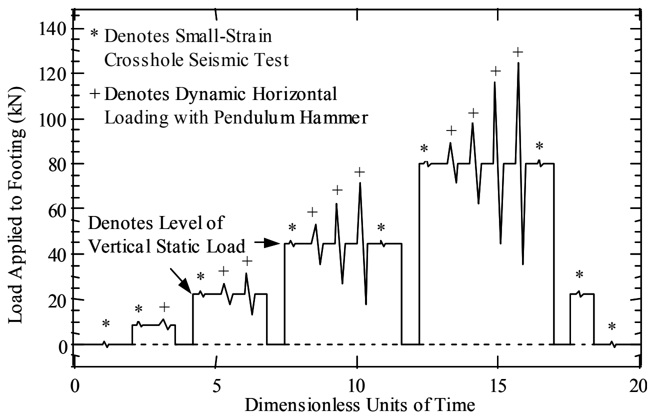


Figure 4. Schematic illustration of the testing sequence used in the small-strain (linear) and large-strain (nonlinear) shear wave measurements

loading sequence is shown in Figure 4. For each dynamic, horizontal loading stage, a vertical static load was first applied to the footing, followed by transient horizontal loading with the pendulum source. Static loads applied to the footing ranged from 0 kN to 80.3 kN. Dynamic loads varied from small to large and were controlled by the release height of the pendulum. The magnitude of the dynamic forces were not measured because they are not required in any subsequent calculations.

3 SOIL AT THE SITE

The test site is located in the flood plain of the Colorado River and is composed of poorly graded sand (SP) with 5% finer than the #200 sieve. The sand 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 sand is heavily overconsolidated due to the removal of at least 8.8 m of overburden (evident from an elevation survey and investigation of historic aerial photographs). In addition, many different layers of sandy soil exist at the site, most of which contain varying degrees of cementation. The nature of the cementation is unknown, but it most likely results from mineralization at particle contacts due to groundwater evaporation. The average water content of the soil around the embedded geophones was 2.7%.

Negative pore water pressures and slight cementation in the sand allowed an intact block sample to be hand carved in the field and transported in a large sampling device. The block sample was then used to determine 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, and results from these tests can be found in Stokoe et al. (2005).

4 RESULTS OF IN SITU SMALL-STRAIN MEASUREMENTS

Both compression and shear waves were used to evaluate the small-strain (linear) stiffness of the sand beneath the footing at each loading stage. Small-strain testing was performed before and after large-strain testing. Crosshole tests were used to measure horizontally propagating compression (P_h) waves and horizontally propagating and vertically polarized shear (S_{hv}) waves. Downhole tests were used to measure vertically propagating compression (P_v) waves and vertically propagating and horizontally polarized shear (S_{vh}) waves. In terms of the work presented herein, the discussion is limited to S_{hv} waves. However, it is important to note that S_{hv} and S_{vh} waves exhibited the same small-strain behavior in these tests (Axtell et al., 2002).

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 (Hardin and Drnevich, 1972). The state of stress in the soil beneath the footing increased with each increase in the 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 under each static load. Figure 5 presents typical measurements of S_{hv}-wave velocity versus stress state. These measurements were performed at a depth of 18 cm beneath the base of the footing A uniform pressure distribution was assumed at the footing base, and a Boussinesq stress distribution was used to obtain 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 porewater pressures in the soil, certainly somewhat negative, were unknown. However, because the soil was only 12% saturated, it is reasonable to assume that the change in total vertical stress, $\Delta\sigma_V$, was equal to the change in effective vertical stress $\Delta\sigma_V'$. Horizontal total stresses for the overconsolidated sand were estimated using Mayne and Kulhawy's (1982) equation. Complete details of the calculations are given in Axtell et al. (2002).

As seen in Figure 5, V_{Shv} increases with increasing σ_V and σ_h, as expected (Stokoe et al., 1994). A trend line is placed through the V_{Shv} data that has the form:

$$V_{Shv} = C_{hv} (\sigma_v \sigma_h)^m \qquad (1)$$

where C_{hv} is a constant and m is an exponent indicating the influence of the stress state. The constant, C_{hv}, and the exponent, m, would change somewhat if effective stresses were used. However, it is felt that the values reported in Figure 5 are still reasonably close to those that would be found if effective stresses were known. The large values of S_{hv}-wave velocities and the small value of m (0.066) indicate that the sand at this depth is cemented. If it were uncemented, the values of V_{Shv} would be reduced by more than 25%, and the value of m would be in the range of 0.18 to 0.23 (Stokoe et al., 1994).

Small-strain waves velocities were measured during each stage of high-amplitude testing to: (1) confirm that the soil is responding in general accordance with expectations, (2) investigate the small-strain, site-specific characteristics, and (3) determine if the high-amplitude loading at any stage permanently changed the soil. The relationship shown by the trend line in Figure 5 fulfills items (1) and (2). However, V_{Shv} exhibited a substantial decrease after high-amplitude testing at a static load level of 44.5 kN as shown in Figure 5 by the open square. A similar reduction was also identifiable in the other wave velocities. The high-amplitude loading most likely caused the cementation bonds in the soil to break. Therefore, a different material was essentially being tested after this stage. As a result, testing was discontinued.

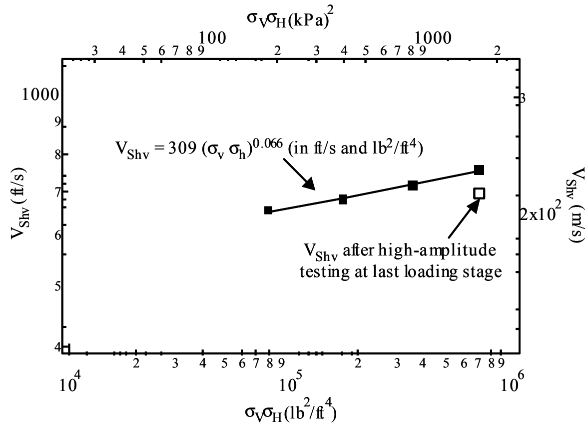


Figure 5. Effect of increasing total static vertical and horizontal stresses on S_{hv}-wave velocities at a depth of 18 cm beneath the footing

5 RESULTS OF IN SITU NONLINEAR MEASUREMENTS

The primary goal in this study was to generate nonlinear behavior in the soil by inducing large shearing strains during dynamic loading with the pendulum hammer. During these tests, the largest shearing strains were only about 0.01%. However, larger strains could have been generated with larger dynamic loads. Unfortunately, due to our inexperience in this new experimental endeavor and the time required to reduce the data and calculate the strains, the exact strain levels were not known in the field. Consequently, load levels were not adjusted in the field to generate larger strains. Additionally, this work was combined with a companion investigation using vertical dynamic loading and nonlinear measurements in axial compression (Axtell et al., 2002). The nonlinear axial measurements were carried to larger strains (0.035%) which damaged the cemented sand as detected by changes in the small-strain wave velocities (see Figure 5). Therefore, testing was discontinued in both the compressional and shear modes after the changed conditions were measured.

Both shear moduli and shearing strains were calculated from the dynamic horizontal loading using the horizontal geophones embedded beneath the footing. The shear moduli at different shearing strains and different geophone depths were calculated from measured velocities of downward propagating shear waves. The equation used to calculate shear modulus from shear wave velocity is:

$$G_{vh} = \rho\,(V_{Shv})^2 \qquad (2)$$

where: G_{vh} = shear modulus in the vertical plane, and ρ = total mass density of soil. Each value of wave velocity used in Equation 2 is associated with some incremental travel path; i.e., between two horizontal geophones at different depths. The shearing strain level was evaluated at each geophone location and was calculated assuming plane wave propagation by:

$$\gamma_{vh} = U_{PPV} / V_{Svh} \quad (3)$$

where: γ_{vh} = peak shear strain, and U_{PPV} = peak particle velocity measured with the horizontal geophone. It is important to note that U_{PPV} is measure at a geophone and V_{Svh} is measured between geophones. When computing shearing strain with Equation 3, an average U_{PPV} was used to represent the strain level at the midpoint between geophones. Because the shear wave velocity was measured between the same geophones, the value of shearing strain best represents the strain level associated with the measured shear modulus/shear wave velocity.

The in-situ shear modulus reduction curves (G-log γ) measured at four different vertical static loads are shown in Figure 6a. These measurements are at an average depth of 11 cm beneath the footing. It can be seen that: (1) nonlinear moduli were measured in the strain range of 0.001 to 0.01%, and (2) the nonlinear moduli at a given γ increase with the increasing static load. The nonlinear shear moduli collected at each static load were normalized by the maximum shear modulus (G_{max}) at that load level. The variations of normalized moduli with shearing strain (G/G_{max} - log γ) are shown Figure 6b. Upper, lower and average generic curves for sand from Seed et al. (1986) are also shown in Figure 6b for comparison. It is clear that the in-situ G/G_{max} – log γ relationships compare well with the Seed et al, (1986) empirical curves. Furthermore, the G/G_{max} – log γ relationships show an increase in linearity with increasing confining pressure, which agrees with recent trends in the literature (Stokoe et al., 1994). These results and comparisons lead to the conclusion that the general test method, with its limitations, is a viable field method.

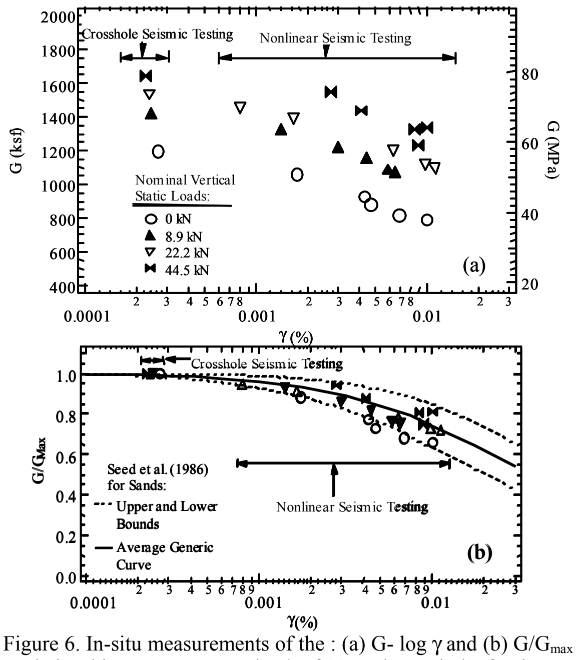


Figure 6. In-situ measurements of the : (a) G- log γ and (b) G/G_{max} – log γ relationships at an average depth of 11 cm beneath the footing

6 NEXT-GENERATION DYNAMIC SOURCE

Two shortcomings of the prototype source shown in Figure 2 are the low force levels that can be generated by the pendulum hammer and the lack of automation. These shortcomings are overcome with a new dynamic source that has been developed as part of the George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES), funded by the U.S. National

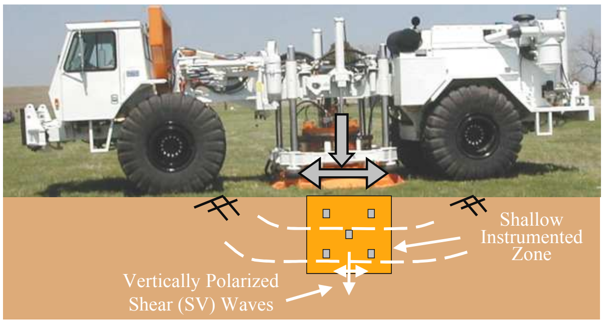


Figure 7 Illustration of new dynamic source (T-Rex) loading an instrumented site

Science Foundation. The source, called "T-Rex," is housed at the NEES equipment site at the University of Texas at Austin. T-Rex is capable of generating large dynamic forces in any of three directions. An illustration of T-Rex applying a static vertical force and a sinusoidal horizontal force to the top of an instrumented site is shown in Figure 7.

7 CONCLUSIONS

A large-strain seismic test in which nonlinear V_S measurements are performed in near-surface soil deposits appears to be a viable method for directly obtaining in-situ nonlinear soil properties. The nonlinear variation in G/G_{max} with γ evaluated at a sand quarry agrees with trends from laboratory measurements and a well-known empirical relationship (Seed et al., 1986). Initial testing was performed using a crude prototype source. Future testing will be conducted with a mobile, state-of-the-art, triaxial shaker called T-Rex.

ACKNOWLEDGEMENTS

Financial support was provided by the United States Geological Survey under FY 2000 NEHRP Grant 00HGGR0015. Capitol Aggregates provided the testing facilities and concrete for the field experiment. This support is gratefully acknowledged.

REFERENCES

Axtell, P.J., Stokoe, II, K.H., Rathje, E.M., and Chang, W. J. 2002. Development of methods to measure nonlinear properties and liquefaction characteristics of near-surface soils. U.S. Geological Survey Final Report.

Hardin, B.O. and Drnevich, V.P. 1972. Shear modulus and damping in soils: design equations and curves. *Journal of Soil Mechanics and Foundations Division,* ASCE, Vol. 98, No. 7, pp. 667-692

Mayne, P.W and Kulhawy, F.H. 1982. K_O-OCR relationships in soil. *Journal of the Soil Mechanics and Foundations Division,* American Society of Civil Engineers, Vol. 108, No. 6, 851-872 pp.

Rathje, E.M., Phillips, R.D., Chang, W.J. and Stokoe, II, K.H. 2001. Evaluating Nonlinear soil response in situ. 4th Inter. Conf. on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA.

Stokoe, II, K.H., Axtell, P.J., Valle, C. and Rathje, E.M. 2005. In Situ measurements of changes in small-strain soil stiffness beneath a footing with varying static and dynamic loads. GeoFrontiers 2005, Special Session Honoring Richard D. Woods, Austin, TX.

Stokoe, II, K.H., Hwang, S.K., Lee, J.N., and Andrus, R.D. 1994. Effects of various parameters on the stiffness and damping of soils at small to medium strains. Invited Keynote Lecture, Int. Symposium on Pre-failure Deformation Characteristics of Geomaterials, Shibuya, Mitachi and Miura, eds., Sapporo, Japan, pp. 785-816.

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.